Part V Reference Technical Data for Part II

Chapter 1 Observations, Examinations and Tests

1 General

1.1 Fundamentals

(1) When designing, constructing, or maintaining facilities, it is necessary to exhaustively extract design conditions and others, which are to be set, and properly set necessary natural and other conditions. To properly set these conditions, adequate observations, examinations, and tests are necessary for quality assurance required for setting objective conditions.

(2) When planning efficient observations, examinations, tests, and so on, the method of reflecting their results to design, construction and other activities must be established adequately in advance.

1.2 Characterization and Structure of this Chapter

1.2.1 Characterization of this Chapter

A variety of technical information to properly and effectively observe, examine, test, and conduct other activities when setting natural and other conditions is summarized in this chapter. Such technical information is related to assuring quality and data control, points to consider in survey planning, utilization of numerical analysis, and other topics. Since the technical information described in this chapter is edited based on information at the time this book was prepared, it is necessary to examine the plan for actual observations, examinations, tests, and other activities based on current information during planning.

1.2.2 Structure of this Chapter

This chapter comprises the following sections (1) and (2).

(1) Items Common to Observations, Examinations and Tests **[1 General]**

Items commonly considered in observations, examinations and tests are summarized in General.

① Quality and reliability of the examination technology **[1 General 1.3]**

Items that impact quality and reliability of examination and test results are extracted and organized. A summary overview, with noteworthy points and other information to secure quality and reliability, will be provided.

② Utilization of ICT (Information and Communication Technology) **[1 General 1.4]**

Utilization of ICT (Information and Communication Technology) and 3-dimensional data are desirable to improve productivity across the entire construction production system. This system is a series of processes from examinations and surveys to maintenance. This section describes the fundamental approach to using ICT (Information and Communication Technology) and 3-dimensional data in examinations.

③ Total system of port facilities' examination technology **[1 General 1.5]**

Typical books and other materials for reference during planning, examining, designing, constructing, and maintaining ports, as well as for examining ports during disaster recovery, are shown as a list for systematic viewing.

(2) Observations, Examinations and Tests of Each Item **[2 to 6]**

Points to note when planning examinations and tests include test methods, data evaluation, and numerical analysis. The important points are summarized for items shown in **Table 1.2.1**.

Table 1.2.1 Observations, Examinations and Tests Related to Port Facilities Described in Referenced Technical Documents

Classification	Items to Observe, Examine and Test
2 Observations and examinations concerning weather and sea states	Weather (**2.2**) Tide level (**2.3**) Ocean waves (**2.4**) Tsunami (**2.5**) Bathymetric survey (**2.6**) Swath sounding (**2.7**) Water flow and others (**2.8**) Littoral drift (**2.9**) Hydraulic model experiment (**2.10**) Numerical analysis concerning performance verification of structures (**2.11**)
3 Examinations and tests concerning ground	Plan for ground examination (**3.2**) Examination of existing documents and site surveys (**3.3**) Boring (**3.4**) In-situ test (**3.5**) Sampling (**3.6**) Laboratory soil test (**3.7**) Geophysical exploration (**3.8**) Subgrade and sub base course tests (**3.9**) Pile loading test (**3.10**) Dynamic observation (**3.11**)
4 Observations for setting up ground motions for performance verification	Observing microtremors at the construction site and in its vicinity (**4.2**) Evaluating the site amplification factor based on in-situ earthquake observations (**4.3**) Evaluating the site amplification factor based on microtremor measurements (**4.4**) Zoning the site amplification factor using microtremor measurements (**4.5**) Applying the results of microtremor measurements to port planning (**4.6**)
5 General items concerning examination and test of structures and materials	Steel members (**5.2**) Concrete (**5.3**) Asphalt materials (**5.4**) Stone (**5.5**) Timber (**5.6**) Fender (**5.7**)
6 Observations and examinations concerning environment	Water quality (**6.2**) Sediment (**6.3**) Living organism (**6.4**)

1.3 Quality and Reliability of Examination Technology

1.3.1 Fundamentals

When examining ports, it is necessary to pay full attention to all items and contents which influence the quality and reliability of the results of examinations and tests listed in **Table 1.3.1**. Moreover, the examination results must be traceable, so it is desirable to keep data concerning items and contents which influence quality and reliability at each stage, from examination to design.

Table 1.3.1 Points to Note on Quality and Reliability of Examination Technology (Overview)

Item	Details	Influence to results of examination and others		Points to note, etc.
		Quality	Reliability	
Plan for examination	Timing, period, location, depth, and others of examination		○	• Characteristics of areas to be examined, scale of structures • Time-spatial scale of examination events
Equipment of examination, testing and analysis	Measurement performance	○		• Equipment certification and calibration
	Measurement specifications (data sampling interval, etc.)	○		• Time scale of examination events
Methods of examination, testing, and analysis	Examination, test and analysis in general	○	○	• Criteria, standards, technical manuals
	Identification of organism species		○	• Allocate professional or qualified examiners • Verify data based on existing examination results and others
	Introduction of new technologies	○	○	• Demonstrate experiments when introducing new technologies
Numerical analysis	Numerical analysis model		○	• Phenomenon to be analyzed and analysis accuracy • Characteristics of area subject to analysis • Setting of conditions using field survey results
	Range of calculation, computational grid interval	○		• Time-spatial scale of phenomenon to be analyzed
Interpretation of the examination data	Outlier and variation of data	○	○	• Clarify the data handling process (checking against existing examination results, standards for disposal of outliers, etc.) • Allocate professional or qualified examiners

Note: Quality: quantitatively evaluable set values or examination results by checking against values of standards, criteria, etc.

Reliability: Not quantitatively evaluable set values or examination results (Contents evaluated based on existing examination results or common knowledge)

1.3.2 Fundamental Points to Note

(1) Examination Plan

The reliability of the natural condition examination results improves by considering the examination objectives, the characteristics (topography, geology, etc.) of the area to be examined, the scale of the structure to be designed, the time-spatial scale of the examination event, and by properly setting the examination timing (through the year, four seasons, in stormy weather, etc.), the examination period (if tidal current examination, 15 days and nights or one day and night), the examination location, and the examination depth. Examination quantity (mainly, the number of survey points and survey depth) influences the results' reliability and the examination's cost, so it is desirable to balance the examination specifications with accuracy required in fulfilling the examination's objectives. If the examination reliability results are not subjected to the examination timing, a stepwise examination method may be adopted.

(2) Measuring and Analysis Equipment

Proper equipment needs to be selected by understanding equipment performance (measurement range, resolution, measurement error, etc.) and considering the examination's objectives. Moreover, measurement capability may change over time; ensure equipment is appropriately examined and calibrated by methods and during periods prescribed in specifications, manuals, etc. If a wave gauge, a current meter, or other equipment allows selection of measurement conditions, such as data sampling intervals, it needs to be properly set considering the time scale of examination events and other examination factors.

(3) Methods of Examination, Test and Analysis

The methods for examination, testing, and analysis described here indicate a series of processes, from examination planning to acquiring and analyzing examination data, conforming to standards, criteria, technical manual and others according to the examination objectives are essential to achieving accountability for examination, test, and analytical results. On the other hand, it is desirable to aggressively introduce new technologies may improve the quality of examination results and reduce examination cost. When introducing new technologies, it is necessary to prove their reliability by way of in-situ field demonstration experiments, etc.

For examination items, such as identification of organism species, whose reliability of examination result is significantly influenced by the examiner’s skill, it is desirable to allocate professional or qualified examiners and verify the examination data by using existing examination results and so on.

(4) Numerical Analysis

Numerical analysis results are commonly used to set the design load concerning ocean waves, storm surges, and tsunamis as well as to evaluate the environmental impact. The numerical analysis model needs to be properly selected considering the phenomenon to be analyzed, required analytical accuracy (spatial resolution, etc.) and the characteristics (water depth or water mass characteristics) of the area to be analyzed. Moreover, it is necessary to set the calculation conditions using in-situ field examination results, if needed.

The range of calculation and the computational grid interval of the numerical analysis model need to be properly set from the time-spatial scale of phenomenon to be analyzed and the required analytical accuracy.

(5) Interpretation of the Examination Data

It is desirable to secure examination data traceability by clarifying the data handling process (checking against existing examination results, standards for disposal of outliers, etc.) and allocate professional or qualified examiners, since the disposal of outliers and the evaluation of variations in examination, test and analysis data may be influenced by the examiner’s and others’ knowledge and skill.

1.4 Utilization of ICT (Information and Communication Technology)

Fig. 1.4.1 shows the directionality of ICT use for ports. To use ICT, it is important to look at overall optimization and productivity improvement in terms of the entire construction production system, throughout the series of processes, from examination and survey to maintenance. Specifically, it becomes important to define the way and range of ICT use in the early business stages. It becomes important to determine whether the improvement of quality and safety in construction and efficiency in work can be achieved to the subsequent design, construction, and maintenance stages, by adequately transferring 3-dimensional data acquired in the examination and survey conducted in the early business stages. To this end, it is important to arrange an environment common to ordering and ordered parties where ICT can be used. Preparation of standards (e.g., application to dredging work [1]), standardization of data, fostering of human resources, and other activities need to be done at the same time.

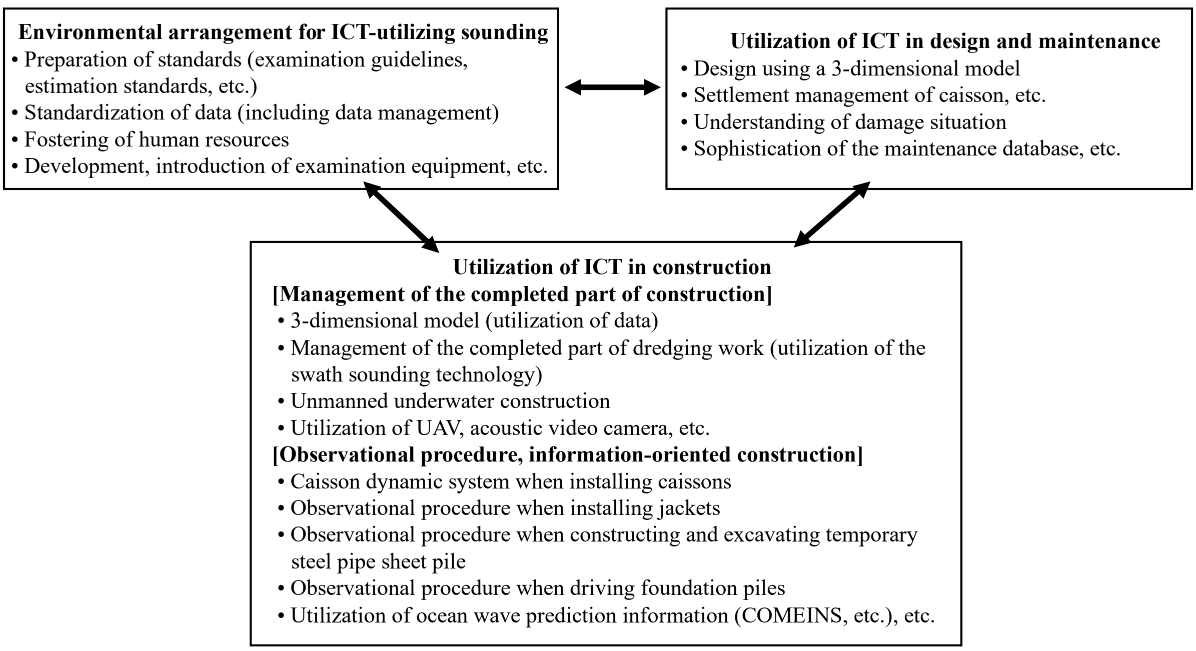


Fig. 1.4.1 Image of Directionality of ICT Utilization in the Field of Ports

1.5 Overall System of Examination Technology Concerning Port Facilities

(English translation of this section from Japanese version is currently being prepared.)

[Reference]

1) "Ports and Harbours Bureau, Ministry of Land, Infrastructure, Transport and Tourism:Implementation policy and new standards of Utilization of ICT (Information and Communication Technology) in dredging work (in Japanese) (http://www.mlit.go.jp/kowan/kowan_fr5_000061.html) "

2 Observations and Examinations Concerning Weather and Sea States

2.1 General

2.1.1 Overview

Observations and examinations concerning weather and sea states are conducted to acquire the required data in the planning, design, construction, maintenance, and disaster recovery stages.

The observation and examination items described in this section can be classified as follows according to their purposes:

① Calculation of external forces acting on port facilities: meteorology, ocean waves, tsunami, and water flow

② Acquisition of information in the height direction (datum level, water depth, etc.) required for the design, maintenance, disaster recovery of port facilities, etc.: tide level and bathymetric survey

③ Resolution of complicated phenomenon caused by various factors: littoral drift

④ Acquisition of information that is difficult to acquire with field measurement by using observed data in designing port facilities: hydraulic model test, numerical analysis

Table 2.1.1 shows these items and observation equipment, and **Table 2.1.2** shows these observation items and analysis methods.

Table 2.1.1 Observation Equipment Concerning Weather and Sea States

Item	Typical observation equipment
Weather (wind, atmospheric pressure, accumulated snow, and others)	Anemometer, thermometer, etc.
Tide level (tide)	Tide indicator (Fuess type, air-launching type), etc.
Ocean waves	Maritime weather meter, GPS wave gauge, etc.
Tsunami	Tide indicator, GPS wave gauge, etc.
Bathymetric survey (including swath survey)	Echo sounder, narrow multibeam sounder, etc.
Water flow (flow condition)	Electromagnetic current meter, ultrasonic current meter, etc.

Table 2.1.2 Analysis Method Concerning Weather and Sea States

Item	Analysis method
Littoral drift	Estimation of the trend and amount of littoral drift utilizing beach topographic map and sounding map Collection and analysis of sea bed materials
Hydraulic model experiment	Model experiment using a plane water tank
Numerical analysis (simulation)	Simulation using analysis software

The items that must be commonly kept in mind as external forces when designing and performing other activities of port facilities, i.e., observation of weather, tide level, ocean waves, tsunami, bathymetric survey, and water flow among the items listed in **Table 2.1.1**, are described hereinafter. Specifically, each item can be used as a reference.

2.1.2 Points to Note in Observation

(1) Selection of an Observation Point

Although it is desirable to perform observations at a point that can represent the phenomenon to be observed, it may be practically impossible to do so for various reasons. Nonetheless, select the most appropriate point among the realistically possible points.

(2) Selection of Observation Equipment

During observation, suitable observation equipment must be selected for the observation objectives. Items that should be considered when selecting observation equipment are as follows:

① Observation items (e.g., When observing ocean waves, is “wave height” the only item to observe? Is “Wave direction” also required?)

② Observation period (e.g., several months or one year or longer)

③ Method to for the observation data (e.g., use the collected data in real time or after observing for a certain period)

④ Maintainability

(3) Quality Assurance

Ensuring the quality of observation data is important even in the observation of natural phenomenon. This point is particularly important for weather and sea state observation equipment, which is always subjected to severe natural environments (outside weather, ocean waves, high pressure at deep water, etc.).

Weather observation equipment is subject to the qualification system stipulated in the Meteorological Service Act, as stated in **Reference (Part II), Chapter 1, 2.2.4 Maintenance of Meteorological Instruments**. On the contrary, the quality of sea state observation equipment needs to be secured with the planned maintenance by the installer.

(4) Safety Assurance

When installing or maintaining observation equipment, the following operations are indispensable:

① High-place operation
② Offshore operation
③ Diving operation

The various laws and ordinances related to these operations must be observed.

Keep in mind that **the Ordinance on Safety and Health of Work under High Pressure** was amended and is effective for strengthening the responsibility of operators. The amendment restricts diving by using air (nitrogen density: approximately 78%) to 40 m of water depth. When diving deeper than 40 m, helium mixture gas or other gases must be used.

(5) Utilization of Existing Data

The seacoast of Japan is observed carefully monitored by the Ports and Harbours Bureau of the Ministry of Land, Infrastructure, Transport and Tourism; the Japan Meteorological Agency; the Japan Coast Guard; the Geospatial Information Authority of Japan; local authorities; and other institutions using a system such as Nationwide Ocean Wave information network for Ports and HarbourS (hereinafter “NOWPHAS”).[1] Most of the observation data are publicized. Moreover, many observation data are available in real time via the Internet.

When planning and implementing observations, utilize these existing data, and complement insufficient data with the individual observations. This approach also makes it possible to confirm the reliability of observation data.

2.1.3 Maintenance of Observation Equipment

Observation equipment, particularly its sensor portion, is sometimes installed on the sea bottom or in the observation tower on the water surface. This may make it impossible to repair immediately when a failure occurs, and data may be lost for a long time. Therefore, scheduled maintenance is necessary. Refer to the section for each type of equipment in this chapter.

2.1.4 Organization and Summarization of Observation Data

Observation data observed with observation equipment are analyzed (statistical processing of observation data for a certain period) according to each observation objective, except for the actual data to be directly utilized. Considering that the organization and summarization of observation data are related to the selection of observation location and observation equipment described in **2.1.2 Points to Note in Observation**, the policy for “organization and summarization of observation data” should be determined when planning an observation before conducting the observation.

2.2 Meteorological Observations and Examinations

2.2.1 Overview

Meteorological observation data related to the development of ports are widely utilized. **Table 2.2.1** shows the manner in which such data are utilized.

Table 2.2.1 Major Meteorological Factors and Methods of Utilization

Classification	Specific examples	Related meteorological factors
Development of port plans	① Examination of breakwater, quaywall face line plans ② Estimation of difficulty in ship maneuvering ③ Estimation of cargo handling operation rate ④ Calculation of wind wave and wind pressure	Wind, visual range
Planning of environmental maintenance measures	① Environmental assessment of air quality ② Examination of countermeasures against smoke damage, pollution, wind-blown sand	Wind, weather, temperature, humidity
Design of structures	① Determination of design wave, design wind velocity ② Examination of stability of quaywall attached facilities and buildings, such as a lighthouse ③ Design of bridges	Wind, depth of snow, temperature
Planning of construction work Supervision of construction work	① Estimation of the number of days when work can be performed ② Securement of work safety	Wind, weather, temperature, humidity, visual range

Meteorological observation data in the port planning stage is used for the planning of the breakwater or quaywall face line, the determination of the utilization form of various port facilities, and the examination of safety and others of navigating ships. Moreover, the elaboration of the meteorological observation and accurate understanding of the air quality condition are also essential for the estimation of changes in environmental influence due to port development, planning of environmental maintenance measures, etc.

The Meteorological Service Act (Act No. 41 of 2017) Article 6 stipulates that in case any governmental institution other than the Japan Meteorological Agency (JMA) or any local government performs meteorological observations, it shall do so in compliance with the technical standards specified by Ordinance of the Ministry of Land, Infrastructure, Transport and Tourism.

Therefore, meteorological observation related to the port development must comply with these criteria (**Ordinance for Enforcement of the Meteorological Service Act Article 1, Paragraph 3**). Considering that **the Meteorological Service Act Article 9** stipulates that certain instruments intended for observational use must not be used unless they passed the verification test conducted by a person who has obtained registration by the Director-General of the JMA, they need to be properly maintained according to this (see **Reference [Part II], Chapter 1, 2.2.4** for details).

The definition of major meteorological factors and the way they are utilized in ports are presented as follows.

(1) Wind

Wind is the relative movement of air to the ground surface, and is expressed as a vector composed of direction (wind direction) and the amount (wind velocity).

Wind direction is the direction from which the wind blows, whereas wind velocity is the distance (wind run) traveled by the air per unit time. Considering that the wind direction and the wind velocity constantly change, instantaneous and mean values are observed.

The instantaneous wind direction and velocity are the wind direction and velocity at a certain moment, and the maximum instantaneous wind velocity in a day (0:00–24:00 in a day; the same applies hereinafter) is called the maximum daily instantaneous wind velocity.

The mean wind direction and velocity are the wind direction and velocity averaged within a specific duration. In the surface meteorological observation, the mean value for 10 minutes before the observation time is defined as the mean value at that time. The maximum value of the 10-minute mean wind velocity in a day is called the maximum daily wind velocity. The wind run in a day divided by 24 hours is the mean daily wind velocity.

The wind direction is expressed in 16 or 36 orientations by dividing the entire perimeter into 16 or 36 portions in a clockwise direction starting at the true north. The 16 orientations increases as 1(NNE), 2(NE), and so on from north to east (clockwise), and the north is defined as 16(N). The true north is defined as the center of north.

Wind is a particularly important meteorological factor. The wind condition chart (wind rose) and others ranged by wind direction and velocity class frequency are always required as a fundamental background of a port.

It is a factor necessary for several examinations such as the fundamental external force for the prediction of ocean waves, operation condition for the cargo handling operation rate, external force for ship hull motion calculation, diffusion of air quality, design of bridges, and construction conditions.

(2) Temperature and Humidity

Temperature of the air is simply called air temperature. According to the World Meteorological Organization, the standard height of temperature measurement is 1.2- 2.0m above the ground level for surface weather observation. In Japan, it is 1.5m.

Temperature is expressed in °C (Celsius) with the value rounded down to one decimal place.

Relative humidity is the ratio of actual water vapor pressure to the saturation water vapor pressure corresponding to its temperature, and it is expressed in %.

Temperature is a factor for quality control in concrete placement. An extraordinarily high or low temperature will become a problem. Temperature and humidity can be used to understand and control the work environment, such as in preventing heatstroke.

(3) Atmospheric Pressure

Atmospheric pressure is defined as the pressure of the static air. Therefore, the atmospheric pressure on the ground is equal to the weight of a vertical column of air above the unit area, i.e., its mass multiplied by the acceleration of gravity.

Atmospheric pressure is expressed in hPa (hecto Pascal) with the value rounded down to one decimal place.

Atmospheric pressure is a fundamental factor that contributes to various weather phenomena. Pressure-gradient force causes wind, and the movement and direction of high- and low-pressure systems contribute to the change in climate. Wind direction and velocity may be estimated from a weather map (contour) when determining offshore wind distribution.

The pressure change tendency provides one of the criteria for disaster prevention activities in a site by monitoring the approach and pass of typhoons and significant low-pressure systems and/or confirming the pass of cold fronts.

(4) Visual Range

The visual range expresses the degree of atmospheric turbidity near the ground surface in distance. The daytime visual range is the maximum distance when a dark target can be seen (the shape of the target can be identified) by the naked eye with the sky as a background. If it differs in direction, the minimum distance becomes the visual range.

Fog is the condition wherein the vapor near the ground surface is cooled and condensed to float fine water particles and wherein the visual range becomes less than 1 km. The condition wherein the visual range is 1 km or more but 10 km or less is called mist.

Factors other than fog that shorten the visual range include strong precipitation phenomenon and, in rarer cases, smog and yellow sand.

The visual range not only influences the navigation of ships but can also be a factor to prevent works that depend on eyesight, such as a survey.

(5) Precipitation and Depth of Snow

The phenomenon in which vapor condenses in the atmosphere as rain, snow, big hail, small hail, or others; sublimed water droplets or ice chips drop; and sublimed, frozen, and melted ice chips or water droplets drop, and the dropped objects are called precipitation.

The amount of precipitation indicates the amount of falling objects that reached the horizontal plane (or horizontal projection plane) of the ground surface within a certain period and is expressed in the depth of water. If the water originated from dew, frost, splash, drifting snow, and others and if the fallen water cannot be distinguished, the amount of the former is also included in the amount of precipitation.

Precipitation is expressed in mm with the value rounded down to one decimal place. However, if the first decimal place of the observed value is less than 0.5, it is expressed as 0; if it is 0.5 or more and less than 1.0, it is expressed as 0.5.

The depth of snow indicates the depth of naturally accumulated snow and is expressed in cm. The depth of snow accumulated in a certain period (normally 1 hour) is called the snowfall depth but is different from its total.

Rain becomes a factor in the supervision of construction work, work planning of concrete placement, and other works. The depth of snow is treated as a part of loaded weight in cold regions.

(6) Weather

Weather is the overall condition of atmosphere focusing on the atmospheric phenomena and the cloudage (apparent ratio of clouded part to the whole sky). The JMA uses 15 weather names in its weather information such as “sunny,” “rainy,” “thunder,” etc. **Table 2.2.2** shows the major names. When several types of weather can be selected at the same time, the lowermost name in the table shall be selected.

Table 2.2.2 Major Weather Names and Their Definitions

Weather name	Definition	Remarks
Sunny	Cloudage is eight or less.	Clear and sunny (cloudage is one or less) or slightly cloudy (cloudage is nine or more and cirrus, cirrocumulus, and cirrostratus are prevailing) are included.
Cloudy	Cloudage is nine or more (excluding slightly cloudy).	
Fumy	The condition in which dry fine particles lessen the visual range to less than 10 km.	Health hazard may be possible owing to yellow sand and/or soot and smoke.
Foggy	The condition in which fog lessens the visual range to less than 1 km.	
Rainy	The condition in which water droplets of 0.5 mm or more in diameter are falling.	Drizzle if the diameter of the water droplet is less than 0.5 mm
Sleet	The condition in which rain and snow are falling at the same time.	
Snowy	The condition in which crystalized freezing water droplets are falling.	Hail is included.
Big hail	The condition in which frozen water droplets of 5 mm or more in diameter are falling.	Small hail if the diameter of the frozen water droplet is less than 5 mm.
Thunder	The condition in which thunderbolt falls or thunder is heard.	

Weather may influence the operation of port construction work and/or work efficiency. Moreover, thunder occurring under developed cumulonimbus may accompany gusts (including tornados) and severe precipitation, in addition to lightning, thus causing significant damage in a short time. When information about these phenomena is brought to the site of port construction and other locations, immediate action must be taken such as evacuation to a safer location.

2.2.2 Making Observations

(1) Wind

Due to wind affects port construction, cargo handling works, and others, the need for real-time observation is high, and permanent installation of observation equipment should be considered. Given that wind is easily affected by geography and terrestrial objects, a short-term field observation may be conducted to judge if the existing data acquired from the nearby meteorological offices can be directly utilized.

Wind measuring instruments shall normally be installed 10 m above the ground by selecting a flat and open place and erecting an independent tower or a supporting post (Guideline of Meteorological Observation, JMA[3)]). This condition is actually often difficult to satisfy; in the case of a permanent installation, an anemometer tower or an anemometer stand is placed on the roof of a governmental building, and the instrument is installed on the tower or the stand. The instrument shall be installed at least 2 m above the tower to avoid the effect of wind turbulence by the anemometer tower.

Even if an ideal installation of instruments is impossible or if the short-time observation is conducted, an installation location as close to the condition (10 m above the ground at a flat and open place) as possible shall be selected.

(2) Temperature and Humidity

The temperature instruments must be installed considering the representativeness of the temperatures near the observation point. Therefore, the sensor should not be affected by the solar radiation or reflection or radiation from the ground surface or surrounding buildings. Furthermore, it is also necessary to ventilate the area around the instrument well.

Given that humidity and temperature refers to the atmosphere at the same altitude, humidity shall be observed at the same condition as the temperature.

(3) Atmospheric Pressure

Considering that atmospheric pressure has little localness, the values obtained from the nearby meteorological offices can normally be used as a reference. When understanding the change in weather and wind caused by the approach and pass of significant low-pressure system or typhoon and the pass of the cold front on site, the atmospheric pressure from hour to hour shall be graphically represented in real-time, and its changing trend shall be monitored. The atmospheric pressure shall be observed in a room where temperature is well stabilized.

(4) Visual Range

Observe as appropriate, e.g., when dense fog has occurred

Normally, visual observation shall be made at a place where the objective can be well overlooked, e.g., on the roof.

(5) Precipitation and Depth of Snow

Tall trees or buildings in the vicinity disturb the wind and unbalance the falling pattern of rain or snow, thus influencing the measurement of precipitation. Therefore, the instruments should be installed away from nearby tall trees and buildings by two to four times or more of their height (if impossible, 10 m or more) and at a place where air flows as horizontally as possible.

Given that the wind-blown upward rain or snow due to buildings may affect the measurements on the roof of a building or its vicinity, the instruments should not be installed on the roof of a building whenever possible.

2.2.3 Types of Meteorological Instruments[3)]

(1) Wind

① **Cup anemometer**

This is a type of anemometer equipped with three or four semispherical cups as sensors. Three-cup anemometers are commonly used, which observe the mean wind velocity. The JMA used the cup anemometer as an official instrument until 1970.

Currently used cup anemometers can be produced in relatively small sizes, and these anemometers are hand-held types for mobile observation or installed with tripod for convenient environmental measurement at construction sites and other places.

② **Windmill anemometer (Windmill aerovane)**

This is an aerovane that integrates an anemoscope and an anemometer, which can record the change in wind direction and velocity and also observe the instantaneous wind velocity. Given that the distance constant (responsiveness index: the value of SVm where Ss is the time required for the wind velocity to become 0.63 Vm/s when the wind velocity changes instantaneously from 0 m/s to V m/s; a smaller SVm value leads to better responsiveness) is smaller than the cup anemometer, many meteorological observatories currently use this instrument. This instrument comprises a stream-line-type body, a wind direction sensing tail unit perpendicular to the body, four wind velocity sensing propellers, and other parts.

The number of rotations of the propeller per unit time is almost proportional to the wind velocity. This is converted to induced power and output. The body rotates freely with respect to the vertically fixed axis. The signal corresponding to 360° or 256 orientations is the output of the encoder equipped below the body rotation axis or other equipment. The digital recording part is prevailing instead of the autographic-recording part. It samples the signal output from the aerovane at regular intervals (e.g., 0.25 s), processes data, and obtains instantaneous wind velocity, direction and time of occurrence, maximum instantaneous wind velocity, direction and time of occurrence, wind velocity and direction, maximum wind velocity, direction and time of occurrence, etc.

The maximum daily instantaneous wind velocity is obtained from the maximum wind velocity by sampling time. The wind direction at that time is the wind direction of the maximum instantaneous wind velocity. The wind velocity (mean in 10 minutes) is obtained from the wind run for the previous 10 minutes. The wind direction (mean in 10 minutes) is the vector mean value for the previous 10 minutes. The wind direction and velocity at each top of the hour is the wind direction and velocity (mean in 10 minutes) at the corresponding time. The meteorological offices obtain mean wind velocity every 10 minutes from the top of the hour (e.g., 6:20). The maximum mean velocity in 10 minutes becomes the maximum daily wind velocity and the wind direction at the time when the wind velocity reaches the maximum value will become the wind direction of the maximum wind velocity. The meteorological offices choose from among the maximum values from 0:10 to 24:00.

③ **Ultrasonic anemometer**

This instrument measures the wind direction and velocity by utilizing the property of sound wave, i.e., velocity changes according to the wind velocity when it propagates in the air. This instrument uses the ultrasonic wave to distinguish it from surrounding noise.

Where sound velocity is Vs and wind velocity is Vw, the sound wave propagates down the wind at the velocity of Vs + Vw and toward the wind at Vs − Vw. The wind velocity can be measured by measuring the propagation time of ultrasonic between the transmitter and receiver that are placed face to face. Wind can be observed as the vectors of wind direction and velocity by performing measurements per element by using two sets of transmitters and receivers with mutually orthogonal measurement sections, such as east–west or north–south. Moreover, observation errors can be suppressed by arranging the integrated receiver and transmitter at both ends of the measurement section and by measuring both round trip times to offset the change in sound velocity caused by the change in temperature and atmospheric pressure.

This instrument is often installed in secluded areas where frequent inspection is difficult to perform because it has no distance constant in principle, has good weather resistance owing to the nonexposure of working mechanical parts to the environment, and is easily equipped with an anti-ice system.

Ultrasonic anemometers include a 3D ultrasonic aerovane, which enables the measurement of vertical elements, which may be necessary for designing of bridges and others. This anemometer has two sets of ultrasonic transceivers that are located slantwise in three directions at a 120° angular interval. Horizontal velocity and vertical wind velocity can be measured by computing the difference in propagation times obtained from the three slanted sets of sensors.

(2) Temperature and Humidity

① **Aspirated psychrometer**

Aspirated psychrometers comprise two double-pipe mercury-in-glass thermometers (a dry bulb and a wet bulb), a fan to vent each bulb, a ventilation pipe, etc.

② **Electric thermometer (Platinum resistance type)**

Electric thermometers apply the principle that the electric resistance value of conductors, such as metal, changes by temperature. It indicates the temperature by measuring the resistance value of the platinum needle as a sensor.

③ **Electric hygrometer (Electrostatic capacitance type)**

This instrument is used to observe the relative humidity, the dew point temperature, and the vapor pressure. The sensor of the hygrometer is a capacitor structure with macromolecular film insulation and performs measurements by utilizing the hygroscopic property of the macromolecular film and converting the change in electrostatic capacitance owing to the change in the relative humidity to electric signals. The hygrometer is used for observation by placing it in the same vent sleeve as the thermometer. There is also an electric-resistance-type sensor that utilizes the change in conductivity caused by the water absorption of a humidity sensitive material.

(3) Atmospheric Pressure

① **Liquid mercury barometer (Fortin-type mercury barometer)**

This barometer uses mercury as a liquid column and determines the atmospheric pressure by measuring the height of the mercury column, which balances the atmosphere pressure. This instrument is used to correct other barometers.

② **Electric barometer (Electrostatic capacitance type)**

The electrostatic-capacitance-type sensor has a vacuum part on the silicon substrate. It detects the change in electrostatic capacitance in electric signals owing to the displacement between electrodes above and below the vacuum part caused by the change in atmospheric pressure. Given that the pressure change tendency can be confirmed on the autographic-recording paper by processing and showing the observation values graphically, the approach and pass of significant low-pressure systems or typhoons, pass of cold fronts, and other phenomena can be identified. This instrument is useful on site.

③ **Aneroid barometer**

This instrument measures the atmospheric pressure by balancing between the atmospheric pressure, which tries to make a dent in an almost vacuumized disk-shaped or cylindrical metal airtight container, and the repulsion force of a built-in spring in the mechanism. This is less accurate than the Fortin-type mercury barometer, but is compact and easy to handle and measure.

(4) Precipitation

① **Tipping-bucket-type rain gauge**

The tipping-bucket-type rain gauge comprises a receiver with a 20 cm inner diameter receiving port, a water filter, a tipping bucket, a reed switch for pulse generation, and the others inside of it. The receiver is equipped with a double-wire gauze to remove refuse. The receiving port is approximately 50 cm higher than the ground surface including a mount. In heavy-snow regions, the receiving port is raised in reference to the deepest snow in successive years. The rainwater caught by the receiving port pours into one side of the tipping bucket through a funnel and the water filter. The collected rainwater in the tipping bucket moves the gravity center of the tipping bucket to the opposite direction of the supporting point. When the amount of precipitation reaches 0.5 mm, the tipping bucket tips and drains the rainwater. The tipping instantly actuates the reed switch to generate a pulse in the connected electric circuit. Thereafter, continually falling rainwater is collected in another tipping bucket. During the rain, the buckets tip alternately by 0.5 mm of precipitation and generate as many pulses.

For observation in cold regions, warm-water and overflow tipping-bucket-type rain gauges are used.

The warm-water tipping-bucket-type rain gauge has a double outer cylinder in the receiver, anti-freezing fluid encapsulated between the double outer cylinder, and a heater inside the double outer cylinder to keep the water at a constant temperature (5 ± 1°C) with a thermostat. The snow that falls in the receiving port is heated, converted to water droplets, and enters the tipping bucket through the water filter.

The overflow tipping-bucket-type rain gauge keeps the anti-freezing fluid encapsulated between the double structure outer cylinders at 10 ± 1°C with a thermostat and an electric heater to heat the water and the oil in the receiver. The snow that falls in the receiving port is changed to water droplets with heated oil and drops on the water surface through the oil layer. Therefore, the water flows out of a water overflow port by the amount that heightened the water surface and enters the tipping bucket through the water filter.

② **Weather radar**

The weather radar observes rain and/or snow existing in a wide region within some 100 kilometer radius by emitting a radio wave (microwave) from a rotating antenna. The distance to rain or snow is measured by the elapsed time from the emission to reception of the radio wave. The intensity of rain or snow is observed by the strength of the returned radio wave (radar echo). Moreover, the movement of rain or snow, i.e., wind in a precipitation area, can be observed by utilizing the frequency drift (Doppler effect) in the returned radio wave.

Precipitation intensity distribution observation in every 5 minute tells the approach or pass of the precipitation area or the change in quantity and intensity of rainfall.

The meteorological observation data from the government or autonomous bodies can be utilized on their websites.[4), 5)] C band radars can be used to observe a rain area in a wide region and the X band radar to monitor local heavy rain. The latter can observe a heavy rain area in more detail, but the wave largely attenuates in the background of the heavy rain area leading to significantly poor accuracy.

③ **Snow scale**

The snow scale is a white column with a cm scale. Its length is determined by referring to the deepest snow in successive years. The snow scale is erected vertically with the lower portion buried in the ground. In places wherein snow is less likely to accumulate or accumulated snow melts in a short time, a normal ruler vertically erected on the ground may be used for measurement.

④ **Snow gauge**

The ultrasonic- and photoelectric-type snow gauges are available. They measure the time difference or the phase difference from emission of the ultrasonic or light from the sensor fixed on the observation pole to reception at the sensor of the ultrasonic or light reflected on the snow surface and calculate the distance from the sensor to the snow covered surface on the basis of the observation value.

2.2.4 Maintenance of Meteorological Instruments

The Meteorological Service Act Article 9 mandates the use of meteorological instruments that have passed the required qualifications. **The Rules for Qualification of Meteorological Instruments Article 15** stipulates the effective period of the instrument qualifications used for surface weather observation.

Table 2.2.3 Effective Period of the Qualification of Meteorological Instruments

Type of meteorological instruments	Effective period of qualification
Electric barometer	10 years
Liquid mercury barometer (Fortin-type mercury barometer)	5 years
Aneroid barometer	
Cup anemometer	
Windmill anemometer (windmill aerovane) Ultrasonic anemometer	
Electric actinometer	
Autographic-recording water-storing-type rain gauge	
Tipping-bucket-type rain gauge	

Other than the above table, the effective period is stipulated as one year only for radiosonde instruments that are used for upper-air observations. The errors allowed at qualification are as follows.

(1) Wind

The distance constants and acceptable errors in the wind velocity are specified by the diameter of windmills for the windmill type and by the diameter of cups for the cup type. The sensitive anemometers are able to accurately observe the wind velocity of approximately 10 m/sec or less and are utilized for judging the atmospheric stability used for atmospheric environment evaluation.

Table 2.2.4 Acceptable Errors for Windmill Anemometers
(1) Distance Constant

Type		Distance constant
Windmill anemometer (except windmill sensitive anemometer)	① When the diameter of a windmill exceeds 15 cm	8 m or less
	② When the diameter of a windmill is 15 cm or less	9 m or less
③ Windmill sensitive anemometer		6.5 m or less

(2) Wind Velocity

Type	Qualification range	Acceptable errors
①	The wind velocity is 10 m/s or less Range where the wind velocity exceeds 10 m/s	0.5 m/s 5% of the wind velocity
②	The wind velocity is 10 m/s or less Range where the wind velocity exceeds 10 m/s	1 m/s 10 % of the wind velocity
③	The wind velocity is 6 m/s or less Range where the wind velocity exceeds 6 m/s	0.3 m/s 5% of the wind velocity

The type numbers in circle correspond to the (**distance constant**) table.

Table 2.2.5 Acceptable Errors for Cup Anemometers
(1) Distance Constant

Type		Distance constant
Cup anemometer (except cup sensitive anemometer)	① When the diameter of a cup exceeds 5 cm	12 m or less
	② When the diameter of a cup is 5 cm or less	13 m or less
③ Cup sensitive anemometer		6.5 m or less

(2) Wind Velocity

Type	Qualification range	Acceptable errors
①	The wind velocity is 10 m/s or less Range where the wind velocity exceeds 10 m/s	0.5 m/s 5% of the wind velocity
②	The wind velocity is 10 m/s or less Range where the wind velocity exceeds 10 m/s	1 m/s 10 % of the wind velocity
③	The wind velocity is 6 m/s or less Range where the wind velocity exceeds 6 m/s	0.3 m/s 5% of the wind velocity

The type numbers in circle correspond to the (**distance constant**) table.

For the wind direction, it is stipulated that “when the wind direction changes to 90° at the wind velocity of 10 m/s, the propellers or disk wheels that are directed toward the wind shall become immobilized within 5 s for the changed wind direction.”

The acceptable errors for the ultrasonic anemometers are as follows.

Table 2.2.6 Acceptable Errors for Ultrasonic Anemometers

Qualification range	Acceptable errors
The wind velocity is 6 m/s or less Range where the wind velocity exceeds 6 m/s	0.3 m/s 5% of the wind velocity

(2) Temperature and Humidity

The acceptable error for the electric thermometers is 0.5°C within the qualification range of -50 to +50°C.

The acceptable error for the electric hygrometers is 5%.

(3) Atmospheric Pressure

The acceptable error is 0.7 hPa regardless of the type (Fortin, aneroid, and electric).

(4) Precipitation

The acceptable errors for the tipping-bucket-type rain gauge are as follows.

Table 2.2.7 Acceptable Errors for Tipping-Bucket-Type Rain Gauge

Type	Qualification range	Acceptable errors
The tipping quantity of rain is 0.5 mm or less	The quantity of rain is 20 mm or less Range where the quantity of rain exceeds 20 mm	0.5 mm 3% of the quantity of rain
The tipping quantity of rain exceeds 0.5 mm	The quantity of rain is 40 mm or less Range where the quantity of rain exceeds 40 mm	1.0 mm 3% of the quantity of rain

2.2.5 Organization and Summarization of Observation Data

(1) Wind

There is a form for wind direction and velocity to organize the mean wind direction and velocity at every hour on the hour, the maximum daily mean wind velocity and its wind direction, the maximum daily instantaneous wind velocity, and its wind direction every month.

When the observation period exceeds one year, organize the frequency of occurrence per wind velocity class by wind direction. As the occurrence property changes by the season, tabularize each hour value, the maximum daily wind, and the maximum daily instantaneous wind through the year, by season and by month. Furthermore, draw wind roses.

The wind velocity is classified as follows:

$U < 0.3$ m/s Calm

0.3 m/s $\leq U <$ 5.0 m/s

5.0 m/s $\leq U <$ 10.0 m/s

10.0 m/s $\leq U$ Strong wind

The organization of the data observed with the sensitive anemometer for the atmospheric environment classifies the wind velocity into the following five ranks:

$U < 2$ m/s

2 m/s $\leq U <$ 3 m/s

3 m/s $\leq U <$ 4 m/s

4 m/s $\leq U <$ 6 m/s

6 m/s $\leq U$

(2) Others

No organization form other than for wind has been specified. The following has been prepared in reference to the organization method adopted by the meteorological offices. Organizable content differs according to the observation items or the data processing method in meteorological instruments. Items for nonorganizable content may be left empty.

Month	Day	Daily precipitation	Daily maximum hourly precipitation	Daily mean wind velocity	Daily maximum wind	Daily maximum instantaneous wind	Daily mean temperature	Daily maximum temperature	Daily minimum temperature	Daily minimum humidity	Special notes
											Maximum snow depth, minimum visual range, atmospheric phenomenon (e.g., fog, lightning), typhoon, alarm, accident, etc.

2.2.6 Meteorological Observation Data of the JMA and Utilization of Grid Point Value (GPV) Data

(1) Utilization of Meteorological Observation Data and Points to Note

Japan has nearly 70 meteorological offices (meteorological observatories and meteorological stations), which conduct surface observations all over the country, and the number of AMeDAS observatories is 1,300.

Among the meteorological factors concerning port administration and other tasks, meteorological offices provide the wind direction, velocity, atmospheric pressure, temperature, humidity, precipitation, depth of snow, and visual range. Approximately 850 AMeDAS observatories provide the wind direction, velocity, precipitation, and temperature (approximately 270 among them also observe the depth of snow), and there are approximately 460 observatories that observe precipitation only.

① **Wind direction and velocity**

The wind direction and velocity are prone to effects by the surrounding geography and structures, the condition of the ground surface, and so on, and the wind velocity in an inland area is generally weaker than that in a coastal area. The wind direction and velocity are quite localized, and it is difficult to correct the effect of geography in a simple manner. Therefore, it is necessary to fully scrutinize whether the data represent the nearby sea area by selecting a place of similar geography near the shore rather than selecting the nearest observatory.

The wind direction and velocity are observed at the standard height of 10 m above the ground level, but the actual height is often set according to the location considering the effect of the surrounding terrestrial objects and the building itself to establish. Moreover, the standard height of the AMeDAS was 6.5 m above the ground level, but recently it has been changed to 10 m in many locations. Therefore, it is necessary to be careful when connecting statistics (usually wind data are treated separately before and after the relocation of the AMedas facility).

When the wind velocity at a height different from that of the observation point on the sea is needed, correction is necessary because the wind velocity is influenced by the height from the ground level. As a correction method of height, a logarithmic law is induced from the equation of motion under the condition that the atmospheric stability is neutral in the flat topography of uniform roughness. However, as a simplified method, a power law that assumes that the velocity distribution in the height direction is proportional to the power of height from the ground level is generally used.

The aerovanes of the meteorological offices were changed from the cup anemometers to the windmill aerovanes in 1975. Given that the observation data before and after the change have systematic differences (the windmill type shows approximately 10% less velocity), only the data after 1975 shall be used if a long-term statistic is needed.

② **Precipitation**

Precipitation is a phenomenon that is prone to the influence of topography and has strong locality. Specifically, a heavier rain corresponds to a stronger locality. Considering that the precipitation in every year also varies a lot, long-term observation results need to be utilized.

③ **Temperature and humidity**

Temperature and humidity are strongly influenced by the sea. It is cooler in summer and warmer in winter compared with the inland temperature. Moreover, the annual variance is smaller than the precipitation and others. Given that the saturation vapor pressure differs by temperature, the relative humidity value at the same location as the temperature shall be used.

④ **Atmospheric pressure**

Given that the sea-level pressure has fewer localities, there is no specific problem in referring to the data of the nearby meteorological office.

⑤ **Others**

Attention shall be paid to the significant secular changes due to global warming and the heat-island phenomenon. For example, the number of foggy days that relate to the visual range has decreased a lot in recent years. Confirm secular changes and utilize the data for a proper period.

(2) Utilization of the GPV Data and Points to Note

The JMA collects various domestic and overseas observation data from meteorological satellites and the AMeDAS, expresses the continuous volume of the atmospheric state by the values in discrete lattices, and forecasts the future atmospheric state by using the supercomputers. Moreover, these information and data are publicized and the GPV about forecast data including the analytical values that become initial values for numerical forecasting can be obtained from the Japan Meteorological Business Support Center, which is a general foundational juridical organization.[6)]

A mesoscale model GPV (MSM) is used as the forecast data in the 3D lattices for targeting the atmosphere of Japan and its surrounding area for the future condition of temperature, wind, water vapor content, and others by using supercomputers. The horizontal resolution is approximately 5 km, and the forecast up to 39 hours later is announced every 3 hours. However, it is necessary to note that the data is recorded in Universal Time Coordinated (UTC) and not in Japan Standard Time (JST). JST is 9 hours ahead of UTC, e.g., 18:00 of the 10th day in UTC corresponds to 3:00 of the 11th day in JST.

By comparing the GPV data obtained on the spot or on adjacent waters and the data at the same time obtained by the nearby meteorological offices or the AMeDAS, it is possible to compare which meteorological office has high correlativity by element and justify the setting of a representative point.

Considering that homogenous GPV data for 10 years has already been collected, it is also possible to use the statistic of the neighboring data. However, data at an arbitrary time, such as the maximum daily value, cannot be obtained.

2.3 Observation and Examination of Tide Level

2.3.1 Overview

Tide observation is often conducted to continuously measure the tide level. Tracing and other checks may also be conducted, but tide observation is the main topic in this chapter.

(1) Definition of Tide Observation

Tide observation refers to the continuous observation of ground and sea levels. Specifically, this type of observation measures the difference in the level of the port B.M. installed onshore and the nearby sea level. Tide observation may also mean the observation of the Tokyo Peil (T.P.) or the sea level on the basis of a spheroid. It measures the sea level by eliminating relatively short-period variations, such as ocean waves, instead of the instantaneous sea level.

(2) Tide-Level Criteria

The datum level should be kept constant when observing the tide level. The tide level is recorded as the level from level 0 of gauge (hereinafter the observation datum level [O.D.L.]). Considering that the O.D.L. used by a tide gauge is a virtual datum level, the relation between the ground level and the sea level cannot be understood unless the relation to the level of the port B.M. is confirmed while observing the tide level. Moreover, the difference between the O.D.L. and the port B.M. may change when measured only once. The difference in height between the port B.M. and the fixed point near the tide gauge do not fluctuate significantly in a short time unless there is an earthquake. However, it is desirable to frequently check the difference between the O.D.L. and the fixed point near the tide gauge because the difference between the fixed point and the O.D.L. often goes unnoticed owing to a failure of the tide gauge or a manual operation, such as inspection.

When investigating the long-term sea level variation or calculating the datum level for port administration, it is necessary to periodically confirm that the O.D.L. has not changed because a deviation from the quality standard specified by the authority concerned may cause negative effects even if the O.D.L. has not actually changed.

(3) Purposes of Tide-Level Observation

The following shows the main purposes of tide-level observation. The tide level may temporarily be observed at a specified location and in a specified period according to the purpose, but the records obtained at an existing permanent tide station are often diverted to other purposes.

① **Understanding Tide Characteristics**

The sea level generally repeats in half-day- and day-based periodic changes mainly because of the phenomenon caused by the attraction of the moon and the sun, which is called tide. The tide phenomenon can be quantitatively understood by organizing and analyzing the tide observation record of the target sea area. The results enable the prediction of tide at an arbitrary day and time at an observation point.

② **Determination of the Datum Level for Port Administration and Validation**

A unique datum level for port administration is set for each port. However, as time goes by, the change in sea level and ground deformation may deviate from their proper ranges in the existing datum level for port administration. Thus, confirm that the datum level is adequate or not by observing the tide level.

③ **Mean Sea Level Monitoring**

Given that the possibility that the mean sea level (M.S.L.) fluctuates has been noted in recent years, it is necessary to continue monitoring and checking the ground deformation and the interannual variability of the M.S.L. as part of the environmental monitoring.

④ **Understanding the Tsunami, Storm Surge, and Long Period Wave**

When seaside structures are damaged, the first step in the investigation of the cause and development of a recovery plan is to understand the oceanographical conditions, including the tide-level records. Therefore, the tide-level records at the time of disaster are important.

⑤ **Supervision of Construction Works**

When measuring the depth from the reference sea level to the sea bottom in dredging or bathymetric survey and when converting to the water depth, the tide level at the construction time is necessary and is considered the tide-level correction amount.

(4) Explanation of Terms Concerning the Tide-Level Observation

Considering that various institutions are observing the tide level for different purposes, the terms used are not necessarily standardized. In this part, the terms mainly used in the port field are explained below. For the sake of convenience, the terms defined in **[Action] Chapter 2, 3.1 Astronomical Tide** are also explained.

① **Associated Bench Mark**

The Geographical Survey Institute Bench Mark (G.S.B.M.) is used to associate the difference in height with the T.P. Given that the mean result of a bench mark (B.M.) (land elevation) is reviewed every few years, the fiscal year of publication needs to be recorded without fail.

② **Port B.M.**

The port B.M. is a stone marker placed in the port area to determine the levels of tide observatories and port facilities.

Considering that this indicates a relative level to the G.S.B.M., the G.S.B.M. number, fiscal year history in which the mean result was published, leveling date, land elevation difference, and others must be specified. Hydrographic and Oceanographic Department of the Japan Coast Guard calls this the “Hydrographic Department B.M.”

③ **Standard Mark**

The standard mark is a brass mark that measures or indicates the levels installed in the tide observation well frame or on the floor of a tide observation booth.

④ **Fixed point and Datum Constant**

The fixed point is a base point installed to maintain the continuity of the O.D.L. in tide level observation and is an indication part of a scale of the steel scale suspended on the well sea level. The set value of the height from the fixed point to the O.D.L. is called the datum constant. The original set datum constant is normally invariable.

The datum constant measurement confirms the presence of abnormalities, such as a movement or an inclination of the tide gauge. Given that this system may undergo a slight change, such as the movement of equipment or a recording paper and expansion and contraction of a wire, this is considered one of the most important inspection processes for ensuring accuracy.

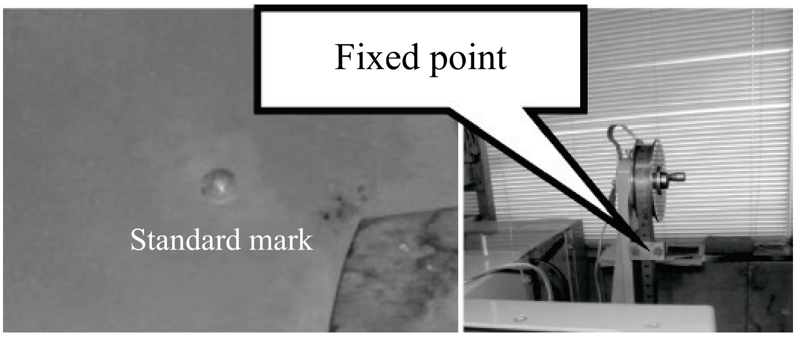


Fig. 2.3.1 Standard Mark and Fixed point

⑤ **Observation Datum Level: O.D.L. (0 of Gauge)**

The level zero surface on a recording paper for the tide-level observation is set with sufficient margin below the minimum water surface so that the tide level does not fall below this surface even when it significantly falls. This is publicized on the "List of Tide Observatories in Japan (Coastal Movements Data Center [17])." The difference in the land elevation from the fixed point shall be set as the Datum constant by the tide station.

⑥ **Relationship Diagram Describing Each Base Point in Tide Observatories** [5)]

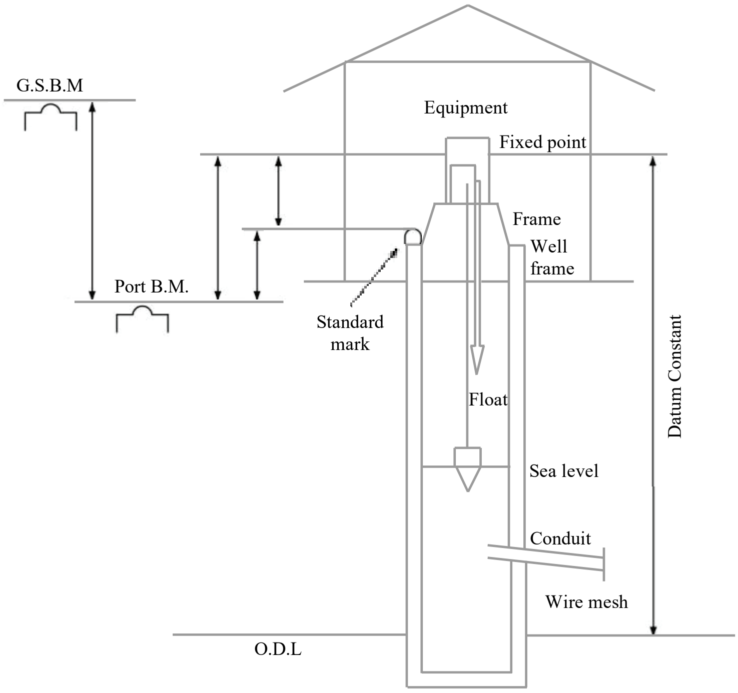


Fig. 2.3.2 Relationship Diagram between Base Points in Tide Observatories

⑦ **Datum Level for Port Administration: Chart Datum Level**

The **public notice for technical reference details of port facilities** aligns the nautical chart and the port facilities by setting the datum level for port administration as the lowest water level (L.W.L.; zero level surface of the nautical chart indication water depth). The chart datum level (C.D.L.) is normally used as the abbreviation for the datum level for port administration. Previously, it was called the datum level for construction work.

⑧ **Tokyo Peil (T.P.)**

The M.S.L. of Tokyo Bay, which has a geoid surface and was adopted as the reference (zero level) of land elevation in Japan. The Japanese Vertical Datum in Miyakezaka, Tokyo, was designated as a land elevation of 24.500 m (above T.P.) on the basis of the tidal level observation at Reiganjima, Tokyo, for six years (from 1873 until 1879) in 1891. After the Great Kanto Earthquake in 1923, the vertical datum was amended to a land elevation of 24.4140 m (above T.P.). The vertical datum sank by 24 mm after the Great East Japan Earthquake occurred in March 2011, and the datum land elevation was amended to T.P. + 24.3900 m.

⑨ **L.W.L. (Design Level)** [4)]

This is the zero level in the water depth indication on a nautical chart and the rise of tide on a tide table. The height differs depending on the port. It was called the datum plane before the amendment of the **Act on Services Related to Waterways** on June 2001. The L.W.L. is equal to the mean sea level minus the sum of the semi-ranges of the four principal tidal constituents. It is listed on the list of the mean sea level, highest water level (H.W.L.), and L.W.L. made public by the Japan Coast Guard. The L.W.L. (design level [D.L.]) and the datum level for port administration (C.D.L.) shall have the same value, but they do not necessarily coincide with the observed value. When the difference becomes significant, it shall be amended by discussions with the Japan Coast Guard (see **Reference [Part II], Chapter 1, 2.3.5 [4]** ③).

⑩ **Sampling Period of the Tide-Level Data**

This is the interval between tide-level data observations. The observed object is a periodic phenomenon of several minute intervals that consider harbor resonance or tsunami. Therefore, the sampling period may be longer than the ocean waves, but the tide observatories of the Ports and Harbours Bureau of the Ministry of Land, Infrastructure, Transport and Tourism adopt a value of 0.5 seconds. This period was selected because it is convenient to have the same interval as the ocean waves and so that the ocean wave data in NOWPHAS can be integrated (see **Reference [Part II], Chapter 1, 2.4.5 [4]**).

In the case of recordings drawn with a pen. We are often drawing with a smoothing line added. This is like observing a one-hour sampling period by eliminating short-period sea level oscillations. Even in this case, the high and low tide times shall be read in minutes.

⑪ **Tide tables published by the Japanese government**

predicted tide levels (astronomical tide levels) are made public by the Japan Meteorological Agency and the Japan Coast Guard, and both levels are generally treated the same. However, the Japan Coast Guard uses the notation in **"Chouseki-Hyou"**, whereas the Japan Meteorological Agency uses the **"Choui-Hyou"**. There are two tide levels of the Japan. Japan Coast Guard: All tide level based on C.D.L. Meteorological Agency: a tide level based on C.D.L and tide level based on T.P. Care needs to be taken because these indicated values are different from the "rise of tide" values in the tide table of the Japan Coast Guard.

⑫ **Astronomical Tide (Tidal Movement)**

Harmonic decomposition is the process of obtaining a harmonic constant for the 60 tidal constituents by using the tide-level observation data of the actual mean value every hour for at least 1 year and by calculating the astronomical tide. The data for approximately the last five years are averaged to reduce the deviation by year.

The harmonic decomposition shall be performed with the 60 tidal constituents from among the data for one year when analyzing a long-term variation of the M.S.L. Check the continuity (there is no change in the standard mark level and of the O.D.L.) when using a harmonic constant for five years or more in the analysis process.

⑬ **Long-Term Variation Analysis of the M.S.L.**

This analyzes the long-term variation of the M.S.L. due to global warming. This obtains the tide level with the ground deformation eliminated by subtracting the vertical deformation of the ground at the associated B.M., the difference in the height of the associated B.M. and the standard mark, and the difference in the height of the standard mark and the O.D.L. from the tide-level data with the short-term variation processed and eliminated. This process also analyzes the long-term trend of the M.S.L. value.

⑭ **Continuously Operating Reference Station (CORS)**

This refers to the continuous observation points using the Global Navigation Satellite System (GNSS), which was installed by the Geospatial Information Authority of Japan. The continuous observation points receive positional signal from the GNSS satellite for 24 hours and continuously observe the ground deformation. The observation values at permanent GPS monuments. have two meanings: one is for the change in the crust, and the other is for the B.M. This manual uses the observation data to calculate the change in the crust of the tide observatories. There are 1,300 CORSs. in Japan as of 2014.

⑮ **GNSS**

This system positions (surveys) with high accuracy by utilizing the satellite positioning system. When permanently installed in tide observatories, this becomes a static observation system.

⑯ **Ellipsoid and Height of an Ellipsoid**

Constants such as the shape of the Earth, gravitational constant, and angular velocity have been defined to assume the Earth as an ellipsoid and express the most similar shape in an equation. The GNSS survey makes it possible to obtain geometric positions (latitude, longitude, and ellipsoidal height). The land elevation is obtained by subtracting the "geoid level" from the ellipsoidal height. Survey instruments have a built-in calculation system. The geodetic datum of the positional coordinates uses the world geodetic system.

⑰ **Geoid Level**

This is the "equipotential surface of gravity" and is also called the "level surface" in the survey field. The height of the global M.S.L. is obtained from the Earth's center of gravity. Related institutions are addressing the improvement of the absolute accuracy of the geoid model to improve the accuracy in understanding the long-term vertical movement of the ground and the land elevation of the reference system (i.e., the height from the M.S.L.).

⑱ **M.S.L.** [12)]

The M.S.L. within a certain period is called the mean sea level of the period. For the practical tide, the mean sea level is the mean tidal level for over one years.

⑲ **Mean Monthly H.W.L.**

The average of the highest sea levels of the months is observed within two days before and four days after the new moon or full moon.

⑳ **Mean Monthly L.W.L.**

The average of the lowest sea levels of the months is observed within two days before and four days after the new moon or full moon.

㉑ **Nearly Highest High H.W.L.** [12)]

This is the water level of the mean sea level plus the sum of the amplitude of the four principal tidal constituents (M2, S2, K1, O1). This may be called the N.H.H.W.L.

The four principal tidal constituents are the four principal constituents of the tide: M2 tide (principal lunar semidiurnal tide, period = 12.421 hours), S2 tide (principal solar semidiurnal tide, period = 12.00 hours), K1 tide (lunisolar diurnal tide, period = 23.934 hours), and O1 tide (principal lunar diurnal tide, period = 25.819 hours).

㉒ **High Water of Ordinary Spring Tides**

A water level above the mean sea level plus the sum of the semi-range of the tidal constituents M2 and S2. The level of the high water of ordinary spring tides (H.W.O.S.T.) measured from the L.W.L. is called the tidal rise at ordinary spring tides (Spring rise).

㉓ **Low Water of Ordinary Spring Tides**

A water level below the mean sea level plus the sum of the semi-range of the tidal constituents M2 and S2. The level of the low water of ordinary spring tides (L.W.O.S.T.) measured from the L.W.L. is called the tidal rise at ordinary neap tides (Spring rise).

㉔ **Control Survey**

This measures the mutual difference in height between the datum level posts, standard marks, fixed points of tide observatories, B.Ms. of the Geospatial Information Authority of Japan, and others.

2.3.2 Tide-Level Observation Process

(1) Types of Tide Observatories

The tide stations are divided into permanent tide stations and temporary tide stations.

① **Permanent Tide Stations**

The permanent tide stations are facilities that assume a permanent observation. They are often installed in the tide observation well. The float-type or the radio wave-type tide gauges are used for observation in many cases. The records are controlled in the tide observatories or offices, but some of them are schematized in real time and published on the Internet.

② **Temporary Tide Observatories**

The temporary tide stations are facilities that were installed to observe the tide level temporarily to understand the tide-level properties, confirm the amount of tide-level correction for dredging and bathymetric survey, promptly set the datum level for port administration, confirm the aptitude of the previously defined datum level for port administration (L.W.L.), and for other purposes. The hydraulic tide gauges or the ultrasonic tide gauges are often adopted for observation.

(2) Observation Procedure of the Tide Level

Fig. 2.3.3 shows the schematic procedure for conducting a tide observation. Some observations use the tide observation well, whereas others do not. Many permanent tide observatories observe by using the tide observation well, but temporary tide observatories often do not use it. In both cases, in addition to the observation with the tide gauge, the difference in height between the port B.M. and the O.D.L. (0 of gauge) must be confirmed.

Given that the difference in height may be altered during the observation period, it is desirable to confirm this at least two times, namely, when starting and finishing the observation, and periodically if the observation takes long.

For the confirmation, install a fixed point near the tide gauge, and perform measurements between the port B.M. and the fixed point by using the leveling instrument or others and between the fixed point and the O.D.L. by using the other method. The measurement method between the fixed point and the O.D.L. differs depending on whether the tide observation well is used. If the tide observation well is used, perform measurements using the "confirmation of the Datum constant;" otherwise, perform measurements using "simultaneous tide observation."

If the tide observation well is used, it is necessary to periodically check if the well has been clogged, in addition to the above mentioned work.

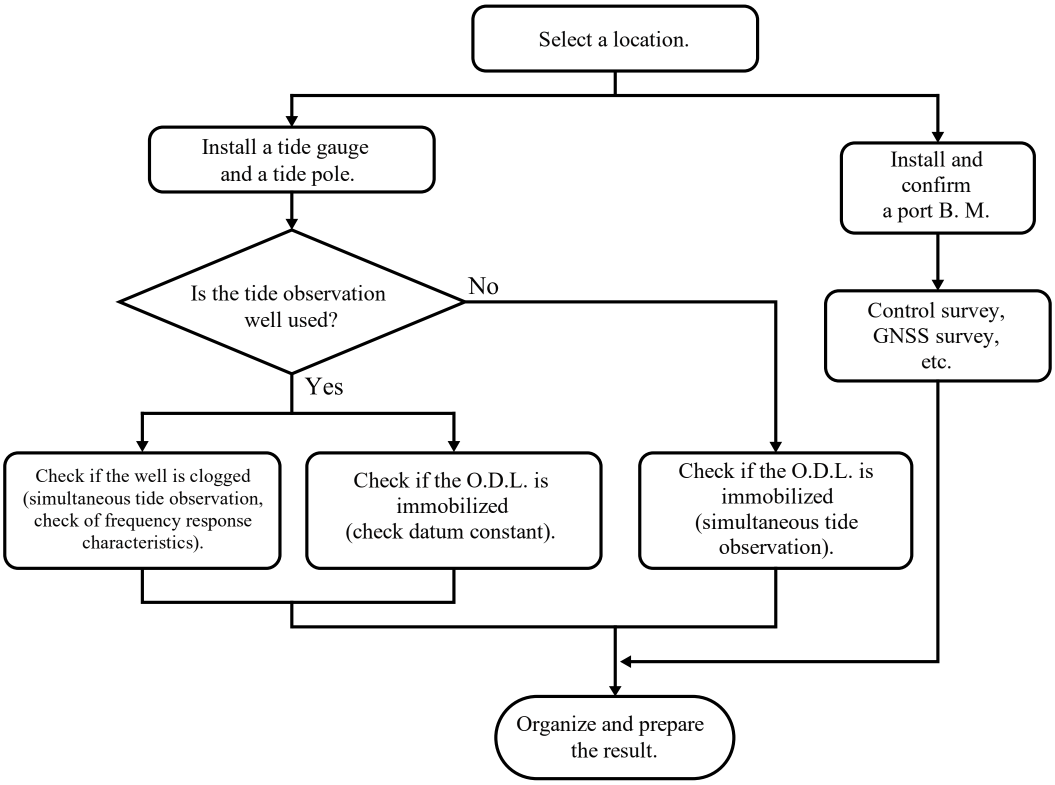


Fig. 2.3.3 Observation Procedure for the Tide Level

① **Select a location.**

There are common conditions for building a tide station. For both the permanent and temporary tide observatories, install a tide pole (water gauge) nearby, and measure the tide level with the tide gauge. These selection conditions are listed below:

<Conditions to build a tide station>

- The observation purpose is properly accomplished.
- Observation is possible even in low tide and high tide.
- Geography allows the easy building of a tide station, and the ground is firm.
- Long-term observation can be maintained.
- Ocean waves, littoral drift, garbage, and others do not interfere with the observation.

<Conditions to install a tide pole>

- The location is near the tide station.
- Observation is possible even in low tide and high tide.
- Waves are calm.

- The control survey with the port B.M. is possible (detached breakwater, sea-crossing level, and lower accuracy)

② **Install a tide gauge and a tide pole.**

The tide level shall usually be observed within one minute from the scheduled time. The accuracy of the start and end times of recording with the tide gauge, the start and end times of maintenance observation, and other activities shall be confirmed.

After installing a tide gauge and a tide pole, the difference in height from the O.D.L. of the tide gauge and 0 of pole to the port B.M. shall be measured.

③ **Check if the O.D.L. is immobilized.**

When installing a tide gauge in a tide observation well and measuring the tide level in the well, the process called “check of the datum constant” shall be performed. This process checks the difference in height between the fixed point with a steel measuring tape called the fixed point installed on the well and the O.D.L. (0 of gauge). This process makes it possible to associate tide-level records to the fixed point. This process shall be performed periodically because the O.D.L. may fluctuate owing to the abrasion or degradation of parts of the tide gauge, as well as earthquakes, manual operation, etc.

If a tide observation well is not used but a tide gauge is installed directly in the sea, install a tide pole nearby, and repeat the work called the “simultaneous tide observation,” which measures the tide gauge and the tide pole to associate the tide-level record with 0 of pole.

The simultaneous tide observation may be performed for a different purpose, namely, to check the clogging condition of the well when using the well (see **Reference [Part II], Chapter 1, 2.3.4 [2] ②**).

④ **Check if the well is clogged.**

Given that the use of a tide observation well may clog the well, check if the well is clogged by simultaneous tide observation, assessment of frequency response characteristics, or other means and remove the sand and silt in and around the conduit if necessary (see **Reference [Part II], Chapter 1, 2.3.4 [3] Check if the well is clogged**).

⑤ **Control survey, GNSS survey, etc.** [7)]

The control survey measures the difference in height from the fixed point to the port B.M. via the standard mark by leveling when a tide observation well is used, from 0 of pole to the port B.M. via the fixed point near the tide gauge when a tide observation well is not used. These differences in height do not fluctuate suddenly and largely unless a large earthquake occurs, but they may fluctuate slowly owing to differential settlements or other phenomena. Therefore, it is desirable to perform measurements periodically (biannually or so) in the case of permanent tide observatories.

In addition to the above measurements, a leveling from the port B.M. to the Geospatial Information Authority of Japan’s B.M. enables the measurement of the difference in height between the land elevation (T.P.) and the mean sea level of a tide station. Furthermore, a leveling to a permanent GPS monument or a GNSS survey that links to the port B.M. makes it possible to grasp the fluctuation of sea level in reference to the ellipsoid. These surveys may be included in the control survey. The difference in height between the port B.M. and the mean sea level (M.S.L.) is influenced by the ground deformation and the change in sea level. By contrast, the T.P. or ellipsoid is not influenced by the ground deformation. Therefore, by setting them as a reference of the tide level enables the confirmation of a long-term change in sea level by eliminating the influence of ground deformation. **Fig. 2.3.4** shows a conceptual scheme of the height management of an ellipsoid in a standard mark by the GNSS survey.

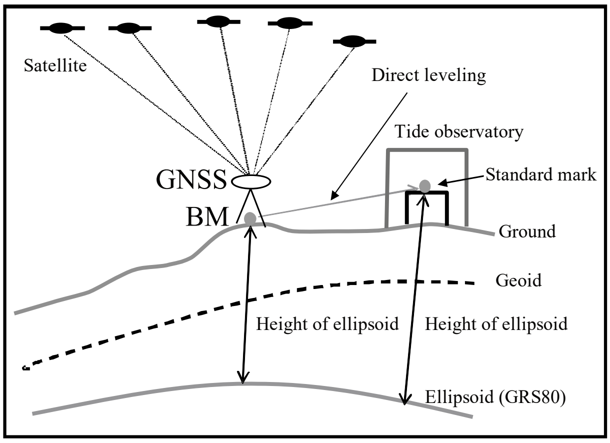


Fig. 2.3.4 Conceptual Scheme of the Height Management of a Tide Station by the GNSS Survey [8)]

2.3.3 Tide-Level Observation Equipment

Major observation equipment is shown below. The recording method has gradually changed from manual reading to the analog recording and then digital recording. Nowadays, most tide gauges adopt the digital recording method.

(1) Tide Pole

A tide pole is a leveling rod with a scale erected at a quaywall and other locations to observe the tide level. The method to read the water level by the scale of the water gauge is the starting point of tide observation. The difference in height between the water gauge and the B.M. shall be measured when erecting the water gauge. The water level shall be read at a certain interval within one hour or at a high tide and low tide together with their times. A water gauge installed near the tide gauge and used for comparative observation may also be called a tide pole.

(2) Float-Type Tide Gauge

The float-Type tide gauge grasps the vertical movement of a float on the sea surface. This is usually installed in the tide observation well but some are installed at a quaywall and other locations to measure the vertical movement of the float in the buoy tube. This is advantageous in ensuring accuracy thanks to its simple measurement principle. The most typical float-Type tide gauge is the Fuess-type tide gauge.

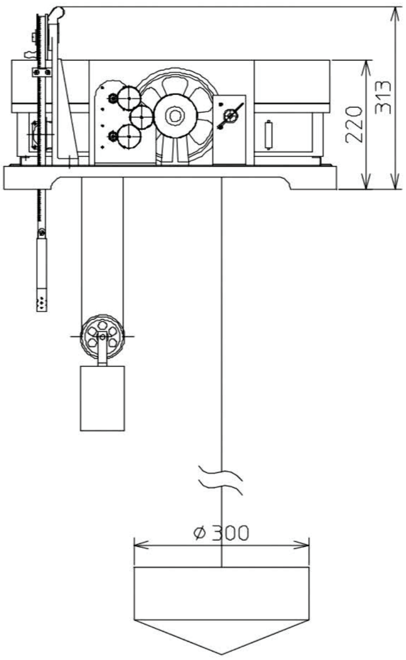


Fig. 2.3.5 Example of a Fuess-Type Tide Gauge (DFT-3; source: manufacturer’s catalog)

(3) Water Pressure-Type Tide gauge

The water pressure-type tide gauge is installed in the sea and obtains the change in pressure from the change in water level. Some water pressure–type tide gauges measure the absolute pressure (water pressure + atmospheric pressure), and others measure the relative pressure at the sea level and the sensor portion. The type to measure the absolute pressure also needs the atmospheric pressure data when converting to the tide level. In either case, a simultaneous tide observation is needed to check the reduction ratio coefficient when converting the water pressure to the tide level and the difference in height between the O.D.L. and the B.M..

(4) Air Launch-Type Tide gauge

Air launch-type tide gauges have two measuring methods: the radio wave method and the ultrasonic method (**Fig. 2.3.6 [a]** and **[b]**). The tide gauges of the radio wave method are often installed in the tide observation well and can characteristically measure the stable tide level. The tide gauges of the ultrasonic method are often installed on the dedicated supporting column and provide easier maintenance compared with the float-type tide gauges via the tide observation well.

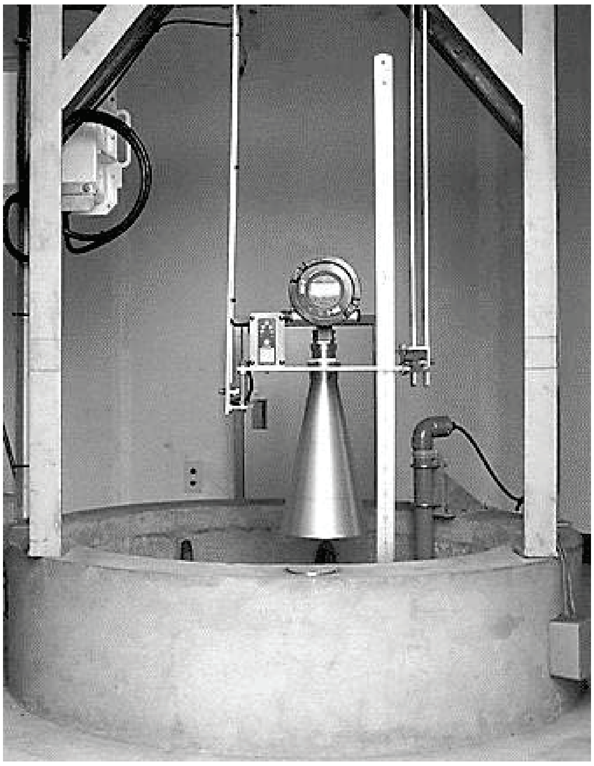


Fig. 2.3.6 (a) Installation Example of a Radio Wave Tide Gauge (Source: Website of the Japan Meteorological Agency)

Fig. 2.3.6 (b) Installation Example of an Ultrasonic Tide Gauge (Nemuro Port)

(5) GPS Wave Gauge

Although the main purpose of the NOWPHAS GPS wave gauge is to check offshore ocean waves and to grasp tsunamis before their arrival, it can grasp the change in offshore water level by obtaining the mean vertical height of a buoy in a one-minute period. Observation from satellite associates the height of the O.D.L. with an ellipsoid not influenced by the ground deformation. A real-time mean sea level measured with a wave gauge can be checked in the NOWPHAS website. For the details of the GPS wave gauges, see **Reference [Part II], Chapter 1, 2.4.3 (4) ⑦**.

(6) Directional Wave Meter on the Sea Bottom

NOWPHAS installs oceanographic observation equipment on the offshore sea bottom and conducts oceanographic observation primarily for observing offshore ocean waves. Among the observation with a directional wave meter, the change in the mean value of an approximately one-minute-long wave height observation records by using the ultrasonic and the water pressure represents the change in the mean sea level at the observation point. Even when a tide station is damaged and loses its observation capability, the multifunction directional wave meter installed on the sea bottom may keep recording. For the details of the directional wave meter, see **Reference [Part II], Chapter 1, 2.4.3 (4) ②**.

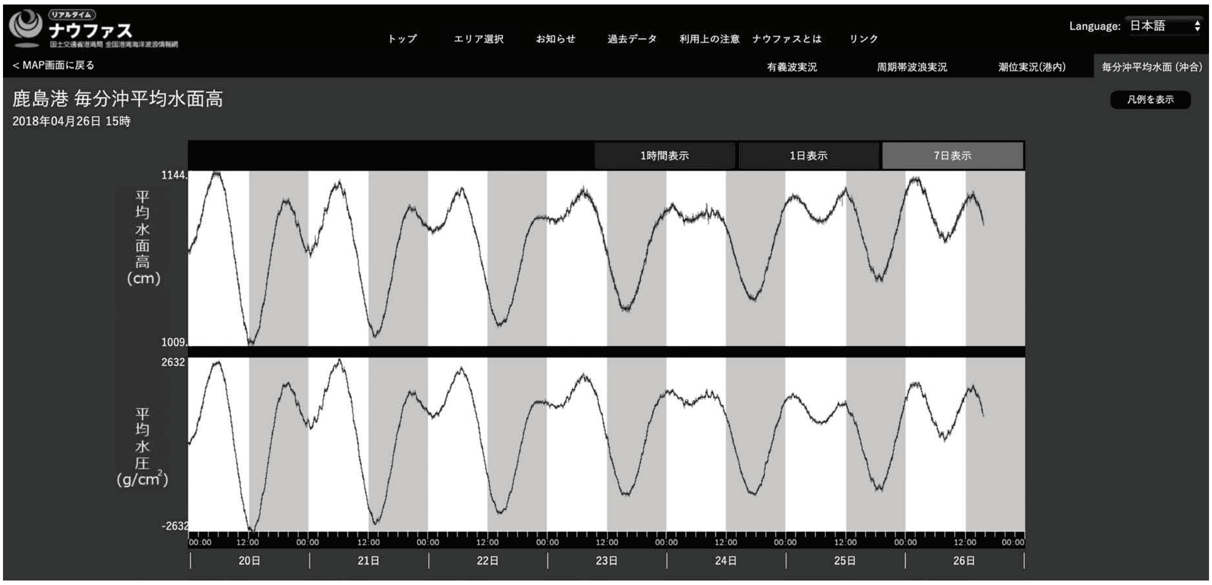


Fig. 2.3.7 Example of the Mean Sea Level Observation Record with a Directional Wave Meter Installed on the Sea Bottom (Source: NOWPHAS Website)

2.3.4 Maintenance of Tide-Level Observation Equipment

There are various types of tide gauges, and their management method differs by type. Here, the management processes common to every type are described.

(1) Checking of Operational Status

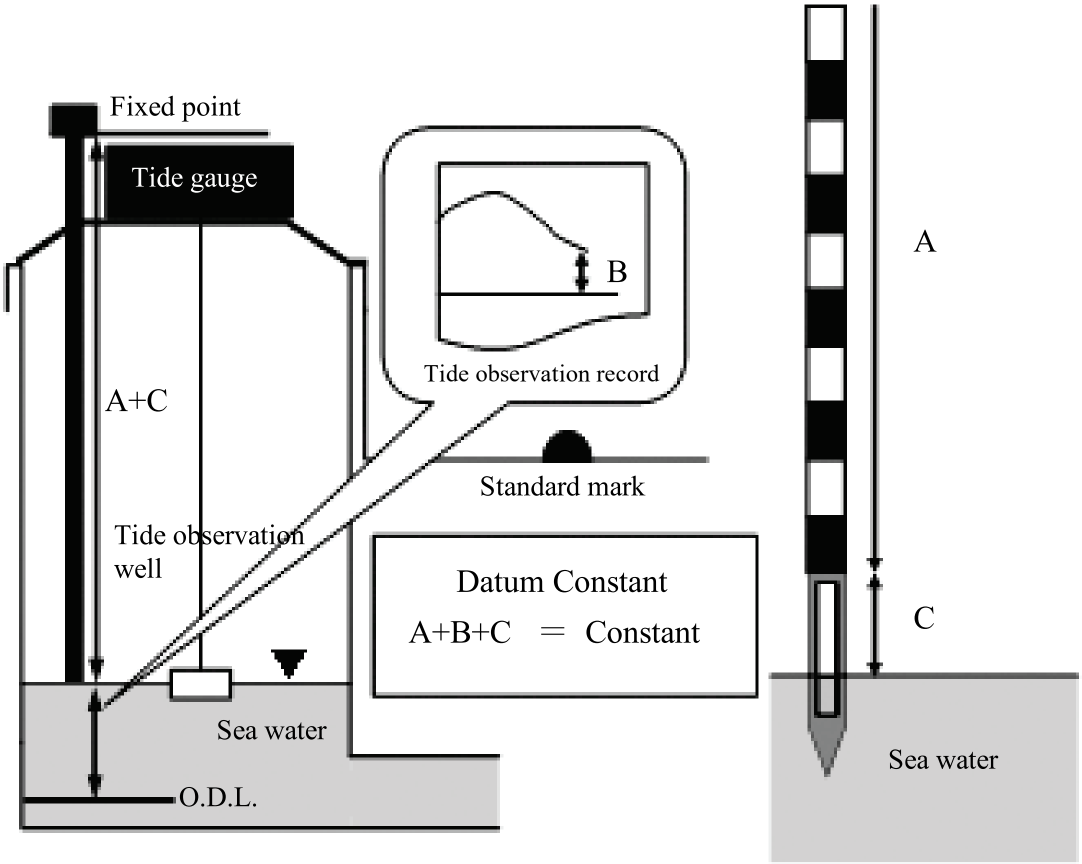
Whether the equipment is largely operating well can be checked from the tide-level data or the time-sequence diagram of the tide-level deviation (actual value - prediction value) even at a distant location provided that the real-time data can be checked. When the ongoing observation record can be checked only on site, it is necessary to visit the site and inspect it. The record shall be checked when possible. For built-in recording type tide gauges that need to be under operation before records can be collected, confirm that the tide gauge is operating and that the installed location has not been changed. Furthermore, check the measurement and acquisition state after collection.

(2) Check height difference between O.D.L. and B.M.

Although the observation operation seems to proceed without problems, the checking of the operation condition may actually reveal that the O.D.L. has moved. The O.D.L. needs to be periodically monitored because its displacement affects the reliability of the tide-level observation. The O.D.L. cannot be measured with a leveling instrument. Therefore, the level shall be checked with a unique method. Different installation methods are described below for cases wherein the tide observation well is used or not used.

① Observation Using a Tide Observation Well (Check of the Datum Constant)

When a tide observation well is used, the water surface in the tide observation well is not affected by the ocean waves, and the water level slowly changes. However, the water surface and its vicinity are dark, and the water level is difficult to check. Instead, a fixed point with a measure called a fixed point is installed on the tide observation well. The difference in height between the fixed point and the O.D.L. shall be measured by measuring the difference in height to the water surface with a measure that has a weight at its tip and associated with the fixed point. At the same time, read the value of the tide gauge down to the millimeter. The sum of both values is equal to the difference in height between the fixed point and the O.D.L. Considering that the water surface always fluctuates even in the well, it is important to measure both at the same time. This shall be measured five times. If the difference between the mean value and the default value (the datum constant) is 10 mm or less, the O.D.L. is judged to have been unchanged. This check method is called the "check of the datum constant"." When the difference is 10 mm or more, report it to the manager and investigate a processing method. **Fig. 2.3.8** shows a schema concerning the check method of the datum constant and an organized chart of the measurement results.



Datum Measurement Record

Mikawa Port tide observatory	Measurement date: Feb. 14, 2015				
Default value (m) 7.743					
Round Time	(1) 13:42	(2) 13:47	(3) 13:52	(4) 13:57	(5) 14:02
Reading of the lead sounding scale	5.090	5.110	5.120	5.140	5.150
Reading of the tide gauge	2.501	2.506	2.505	2.507	2.507
Reading of the flooded paper	0.154	0.126	0.118	0.096	0.085
Total	7.745	7.742	7.743	7.743	7.742
Remarks The tide gauge reads the “A/D converter value.”				Mean (= measurement value)	7.743
				Default value - mean value	0.000

Fig. 2.3.8 Checking Method of the Lead Sounding Constant and an Example of Organized Measurement Record

② **Observation without Using a Tide Observation Well (Simultaneous Tide Observation)**

When a tide observation well is not used, the difference in height between 0 of gauge (O.D.L.) and the B.M. shall be determined by the simultaneous comparative observation of the measurement values of the tide gauge and the tide pole installed nearby. This process is called the “simultaneous tide observation.” A water pressure-type tide gauge is often used for observation without using the well. This also implies that the reduction ratio, which is obtained when the water pressure is converted to the water level, shall be coordinated. The simultaneous tide observation is required at least once to associate the tide-level record to the fixed point. However, the simultaneous tide observations should be conducted two times (at or around the beginning and end of the observation); if the observation period exceeds one month, the simultaneous tide observations should be conducted once a month to confirm that the O.D.L. has not moved during the tide-level observation period.

The organization of results shall be started by obtaining the reduction ratio (a) and the difference in height (b) by the following linear regression calculation.

$$Y=aX+b \qquad \textbf{(2.3.1)}$$

where

Y : Reading of the tide pole

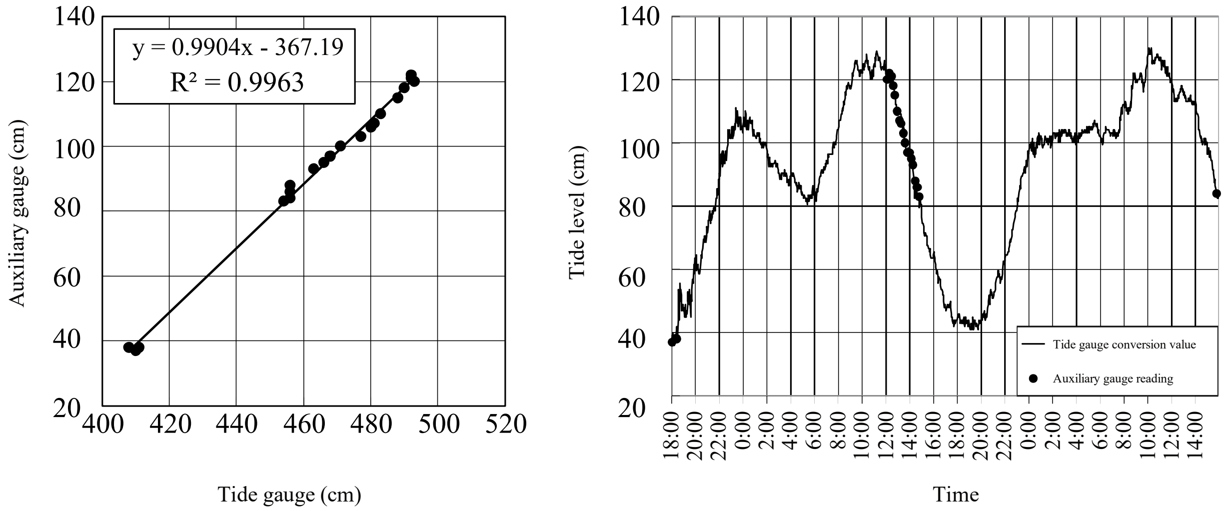
X : Tide-level record of the tide gauge

a : Reduction ratio of the tide gauge

b : Height of 0 of tide gauge on 0 of pole

Thereafter, convert the tide-level records of all tide gauges to the tide level on level 0 of pole by using a and b calculated from **equation (2.3.1)**. The relation between level 0 of pole and the level of the port B.M. can be obtained from the control survey. By coordinating these results, the converted tide level can be associated with the port B.M.

The observation error when confirming a datum constant in a tide observation well is a few millimeters. On the contrary, the observation error when one simultaneous tide observation is performed may amount to a few centimeters if influenced by the ocean waves. Therefore, it is necessary to perform the simultaneous tide observations as many times as possible in a calm environment to ensure accuracy. In the case of water pressure-type tide gauges, given that the simultaneous tide observation corrects the reduction ratio at the same time, observe the high and low tides when the tidal variation is high. **Fig. 2.3.9** shows an example of the organized results of the simultaneous tide observation.



Linear regression calculation of the tidal level data

Time-series comparison of the auxiliary gauge readings and the tide gauge conversion values

Fig. 2.3.9 Example of Organized Results of the Simultaneous Tide Observation

(3) Check if the well is clogged

① **Simultaneous Tide Observation**

The clogging of a conduit of the tide observation well delays the tidal hour in the well and reduces the amplitude. To confirm this, an observation shall be performed using a tide pole. This observation is performed with the same method as that in the simultaneous tide observation described in the previous section, and this work is also called the simultaneous tide observation. However, its purpose and evaluation method are quite different. In an observation that uses the tide pole, a tide pole rod is erected outside the tide observation well, and the readings of the tide pole and the tide gauge are recorded at fixed intervals. This task is performed in time zones with rising tides, falling tides, high water, and low water. The performance of the well shall be judged by comparing the tide gauge data and the tide pole observation data obtained during the work. In other words, confirm that the wind wave component has been eliminated, the conduit is not clogged, and the tide-level component has not been eliminated (**Fig. 2.3.10**.)

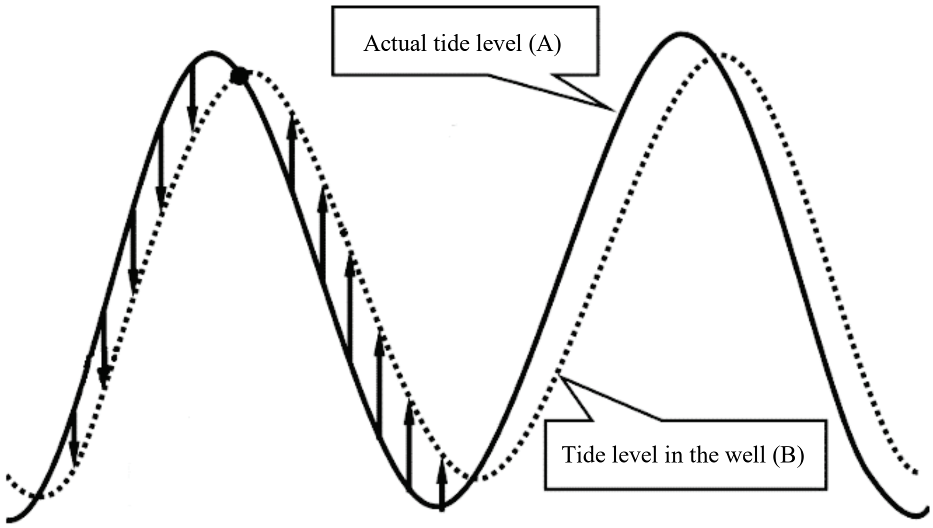


Fig. 2.3.10 Tide-Level Data Obtained from the Well Clogging Check (Simultaneous Tide Observation)

These judgments require professional knowledge and longtime observation between high water and low water. Moreover, there is a problem that the tide pole observation is just a qualitative four-evaluation method and provides no frequency response characteristics required to reproduce the tsunami wave form.

② **Step Response Experiment of the Tide Observation Well** [10)]

The degree of the well clogging is quantitatively checked from the frequency response characteristics by clogging a conduit to manually produce a certain water level difference inside and outside the well, opening the conduit, and measuring the time until the water level returns. Check for well clogging via simultaneous tide observation for a half day or more, including a rising tide and a falling tide. This method can be performed speedily in as short as 30 minutes, and the quantitative result is easy to evaluate. The height of a tsunami of several minute to several 10-minute intervals may be underestimated from the tide observation record. However, the knowledge of the frequency response characteristics may help correct the height and the arrival time of a tsunami. **Fig. 2.3.11** shows an example to obtain the delay time characteristics before and after cleaning from the step response measurement values.

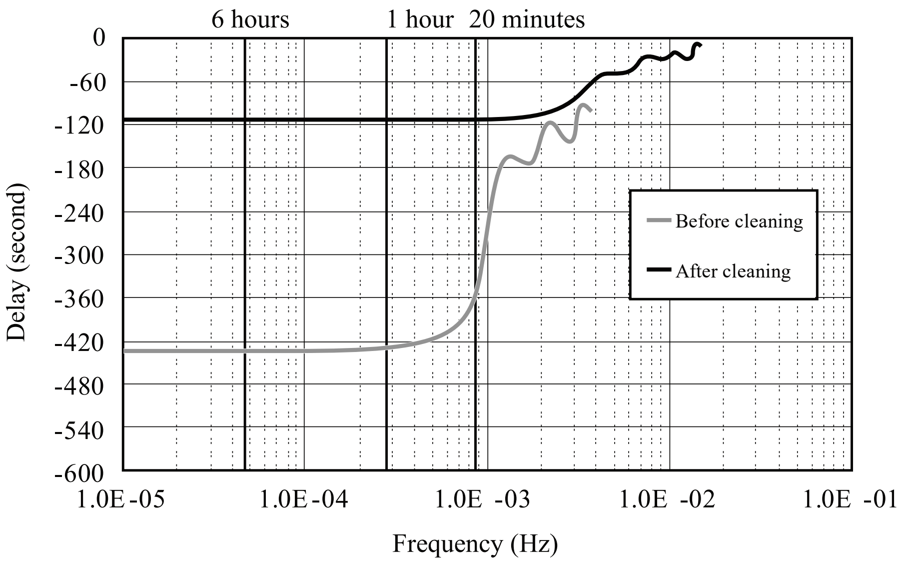


Fig. 2.3.11 Example of Obtaining Delay Time Characteristics Before and After Cleaning from the Step Response Measurement Values

(4) Control Survey and GNSS Survey

The control survey measures the difference in heights between fixed points, such as the stone markers of the Geospatial Information Authority of Japan, the port B.M. standard marks, and the fixed points. This survey obtains the difference in heights between the port B.M., which is the reference stone marker for tide observation and the fixed points. To associate with T.P., it is necessary to measure the difference in height from the stone markers of the Geospatial Information Authority of Japan. To properly understand the effect of ground deformation, it is important to conduct the control survey every other year.

Measuring the difference in height from the nearby permanent GPS monuments B.M. or performing a GNSS survey to complement the control survey enables the confirmation of the changes in port B.M. or the height of an ellipsoid of the standard mark. Therefore, the ground deformation and the change in sea level in a long-term variability analysis of the M.S.L. can be separated.

2.3.5 Organization and Summary of the Tide-Level Observation Data

Currently, the tide-level observation data are saved as a digital record in most cases. Analog data shall be digitized, organized, and saved as necessary by reading the numerical values on the recording paper.

(1) Check if the O.D.L. Is Immobilized

① Problems in the Control Survey and the Confirmation of the Datum Constant

The tide levels are recorded as values on the O.D.L. which is level 0 of gauge. To associate these tide levels to the port B.M., it is necessary to separately check the difference in height between the O.D.L. and the port B.M. The difference in height between the O.D.L. and the port B.M. may vary by natural phenomena, such as earthquakes and differential settlement of grounds; failure; degradation; inspection; repair; replacement of the tide gauge; or human activities, such as relocation of the tide station and the port B.M. Considering that the change in the datum level disables the accurate understanding of the tide level, it is necessary to confirm that it has not moved or confirm the degree of its movement.

This shall be checked by the control survey and the datum constant. Frequent checking enables the confirmation of the change in the datum level. Given that these documents ensure the quality of the tide-level data, it is important to check periodically and save the results with the tide-level records.

The datum level used for the administration of each tide station should be named to avoid confusion. For example, when digital and analog records are used in combination, both records may be called the O.D.L. Furthermore, even when digital records are transmitted from an administration office to another administration facility and when the O.D.L. is changed in this office, the changed datum level may also be called the O.D.L.

The O.D.L. is a name given to level 0 of the utilized tide-level records, whereas the datum constant indicates the difference in height between level zero of the utilized tide-level records and the fixed point. When users of the data postulate this, calling the deviations from this definition O.D.L. or a datum constant may be misleading. Adequate care needs to be taken to not obtain a wrong O.D.L. because a wrong level may lead to significant misunderstanding compared with past water levels or differences in height between the sea level and the port B.M., T.P., etc.

② **History Information on Each Datum Level and Update Management of the Current Value**

When organizing the records of the control survey and the datum constant and trying to understand the long-term change circumstances of the tide gauge O.D.L., insufficient data or insufficient organization of the time often hinders such efforts.

To resolve such conditions, the confirmation of the control survey, datum constant, and others should be organized in the forms of **Fig. 2.3.12** and **Table 2.3.1**. Whenever the control survey and datum constant are checked, add the date, difference in height, and datum constant in the history of the form below and update the land elevation column. If some sections were measured, enter the difference in heights only for such sections, and leave the unmeasured sections blank. If the "confirmation of the datum constant" was performed, enter the date on which the confirmation was performed and the default value. If the land elevation of the associated B.M. was updated, update the elevations of land of ① associated B.M., ② Hydrographic Department Bench Mark, ③ standard mark, ④ fixed point, and ⑤ O.D.L. without fail. Information to which the measurement results can be referred to, such as the name of the investigation and the source, shall be entered into the survey and inspection column. When at least one datum level has been changed, the chart shall be updated. At that time, it is important to leave not only the updated result but also the record of the previous result. It is important to update all records measured at every opportunity, including the inspection work of the tide station. A higher number of measurements lead to the higher reliability of tide observation records.

When updating, if the mean sea level on the O.D.L. (even past data or approximation may be referenced) is known, the mean sea level should be compared with T.P. Although the relation between the mean sea level and T.P. differs by location, it does not change significantly in the same sea area in several years. Therefore, it can be considered a realistic result.

Table 2.3.1 shows an example of organized level administration by the GNSS survey of the standard mark in the tide station. The GNSS survey measures the level of the ellipsoid in the port B.M. Thereafter, the difference in height between the port B.M. and the standard mark is directly leveled to calculate the level of the ellipsoid of the standard mark. If it is combined with the difference in height between the Hydrographic Department Bench Mark and the O.D.L., the level of the ellipsoid of the O.D.L. can be calculated. The accuracy of the level of the ellipsoid of the standard mark shall be checked by comparing the specified value with the result of direct leveling, adding the result to the previous survey data, recording it, and saving the survey history.

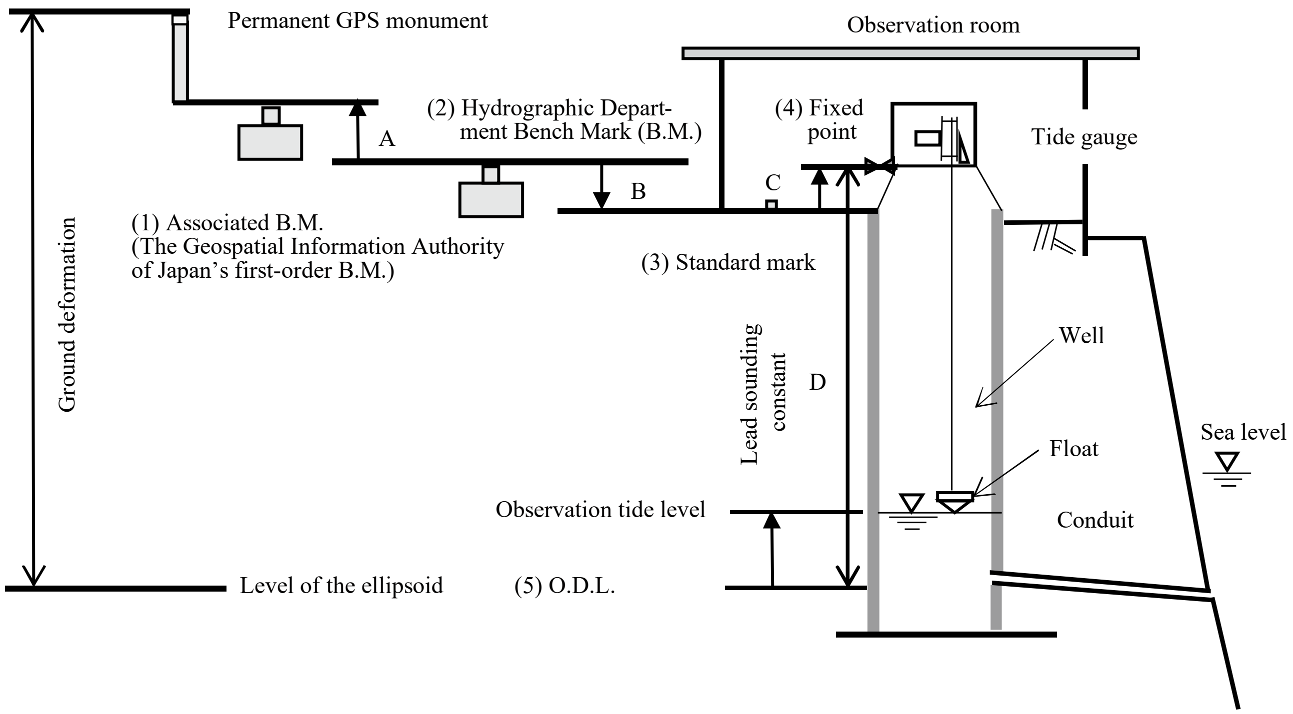


Fig. 2.3.12 Relationship Diagram of the Difference in Height from the Associated B.M. to the O.D.L. 7)

Table 2.3.1 Example of a History Entry Form of the Tide Station Datum Level 7)

Unit: m

Date of measurement	Associated B.M. First-order B.M. No. X ①	Difference in height A ① − ②	Hydrographic Department B.M. ②	Difference in height B ② − ③	Standard mark ③	Difference in height C ③ − ④	Lead sounding base point ④	Difference in height D ④ − ⑤	O.D.L. ⑤	Measurement item	Survey and inspection
MM, YYYY	17.467									①	FY XX mean result
MM, YYYY		−14.575	2.892							A	Xth grade leveling
MM, YYYY				1.334	4.226	1.239	5.465	−7.743	−2.278	B, C, D	Maintenance and inspection
								−7.743	−2.278	D	Maintenance and inspection

* The difference in height between the associated B.M. and the O.D.L. is an example of descriptions shown on the basis of the survey results at each tide station and the measurement records at the maintenance and inspection procedures.
* The ground deformation in the figure is generally expressed in the land elevation.
* The land elevation of the associated B.M. ① shall be recorded by obtaining the data published by the Geospatial Information Authority of Japan from its website or others.
* When the land elevation of the associated B.M. is revised or the referring associated B.M. is changed, the date when the new B.M. was surveyed, its land elevation, and the referent of the survey results shall be recorded.
* The information of the measurement results of A, B, C and D in the figure shall be recorded together with the measurement dates and names of the referred reports and others, and ② to ⑤ shall be calculated.
* When measurement values are added, add rows to the form, and record the same as above.
* Record the O.D.L. ⑤ on each measurement day, save its history information, and utilize it for the analysis of the tide observation records.
*Reflect the results to the description of the investigation report or the observation register on the basis of the organized results and the department in charge of the tide-level observation controls uniformly.

Table 2.3.2 Example of an Organized History of the GNSS Survey Results and the Direct Leveling Results

History of the GNSS survey results and comparison with the direct leveling results

Unit: m

Observation year and month	Electronic B.M.		This quarter	B.M. Level of the ellipsoid	Relative elevation of B.M. - standard mark	Levels of standard mark and ellipsoid	Geoid level		Land elevation	Result of direct leveling	Difference	Remarks
	Mark name	Kind of results	Original quarter				Level	Version				
Jan. 20, 2015	XX	2011	This quarter	41.5921	2.015	43.6071	39.3115	2011	4.296			SemiDyna2014
			Original quarter	41.5039		43.5189			4.207	4.214	-0.007	

(2) Organization of the Tide-Level Data

Fig. 2.3.13 shows the flow to organize the tide-level data. Hourly value, daily mean, monthly mean, and annual mean shall be summarized on the basis of the raw data recorded by the tide gauge. Data corresponding to each time scale shall be used when examining various phenomena.

The observation state of some tide observatories is disclosed in real-time and can be viewed on the web. When organizing the tide-level data, always make the used datum level known. The difference in height between the used datum level and the port B.M. should also be made known.

The tide-level data are usually organized using values from the O.D.L., but it is often converted to values on another datum level according to the purpose of its use. Care should be taken because the organized results may be indicated in values on the datum level for port administration (C.D.L.), T.P., or mean sea level.

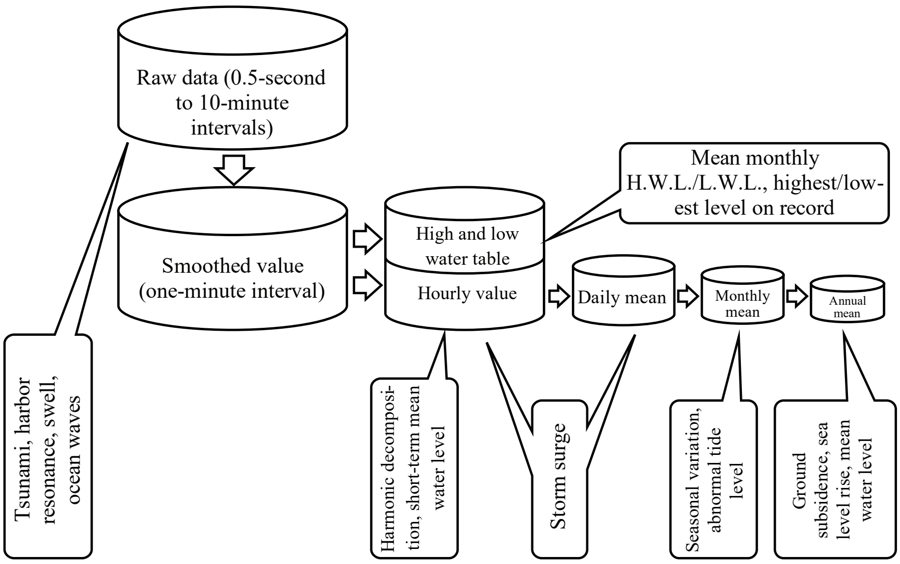


Fig. 2.3.13 Flow to Organize the Tide-Level Data

① **Real-Time Data**

Real-time data are useful for immediately checking the current state, such as checking the operation status of the tide gauge and the latest status in disaster, and correcting the tide level of dredging work or bathymetric survey. The values indicated at this time are the so called preliminary figures obtained from the automatic processing of the measurement values and may be different from the definitive values subjected to the quality management.

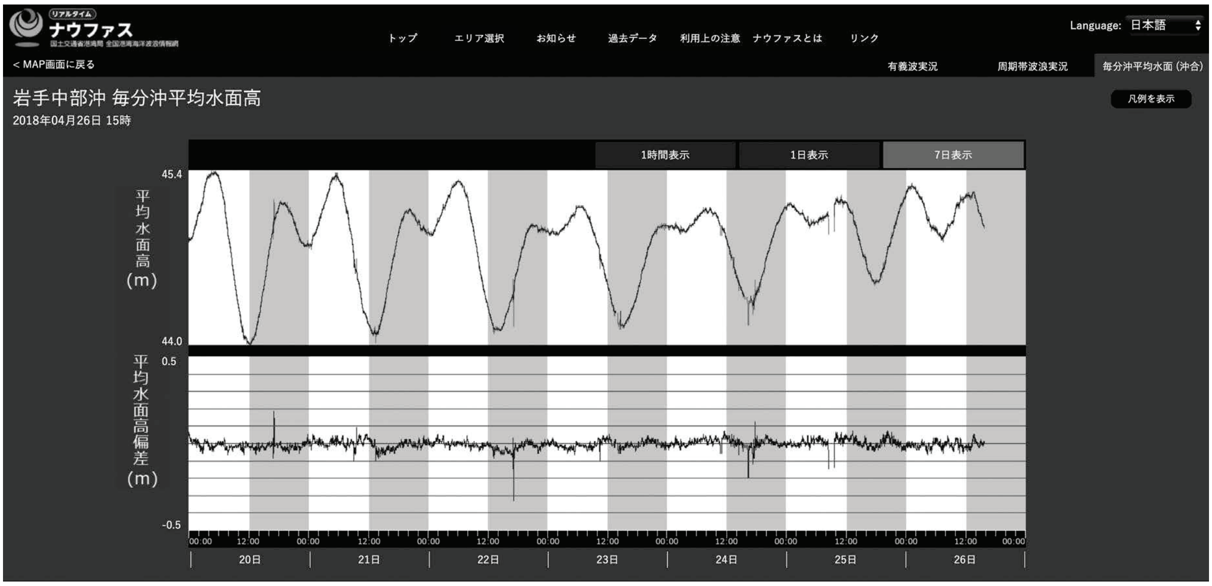


Fig. 2.3.14 Example of Displayed Real-Time Tide-Level Data (Source: NOWPHAS Website)

② **Raw Data**

The source data recorded by the tide gauge is called the raw data. In many cases, the tide level is measured every 0.5 seconds, and the whole data are recorded directly by tide observatories, such as those registered in NOWHAS. By contrast, the mean value in several seconds to 60 seconds may be recorded in some cases. Observations using simplified tide gauges often record the mean values or instantaneous values in the interval of 1 minute to 20 minutes. Vast and unprocessed raw data are difficult to use. However, given that it is the fundamental data on which everything is based, the record shall be saved in its original form. **Fig. 2.3.15** shows an example of the raw data measured by NOWPHAS. Fluctuations of approximately two-minute intervals can be seen in this data.

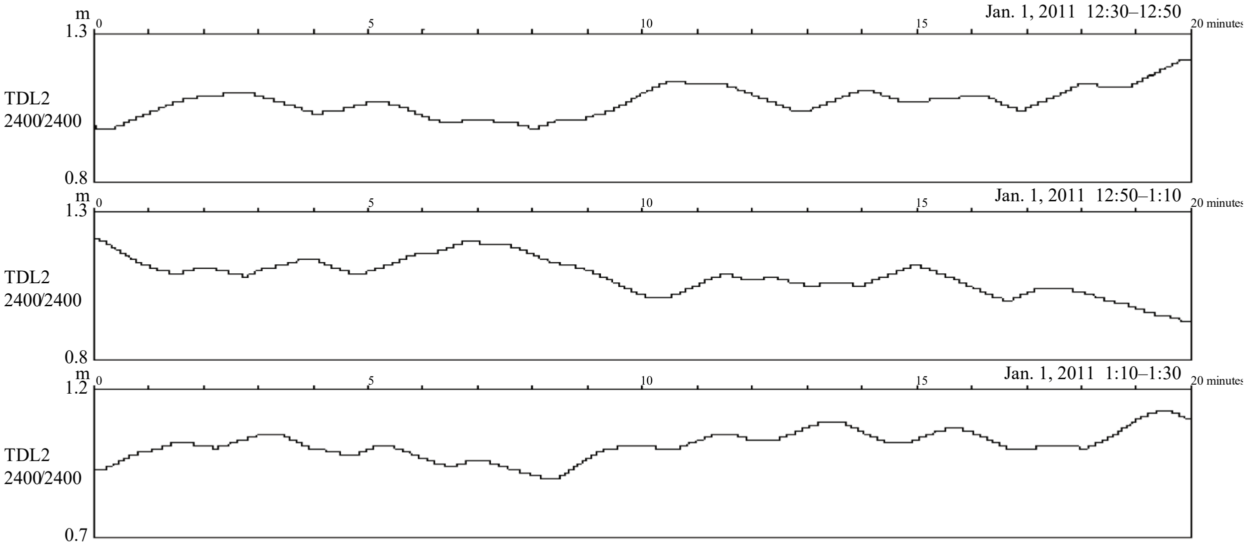
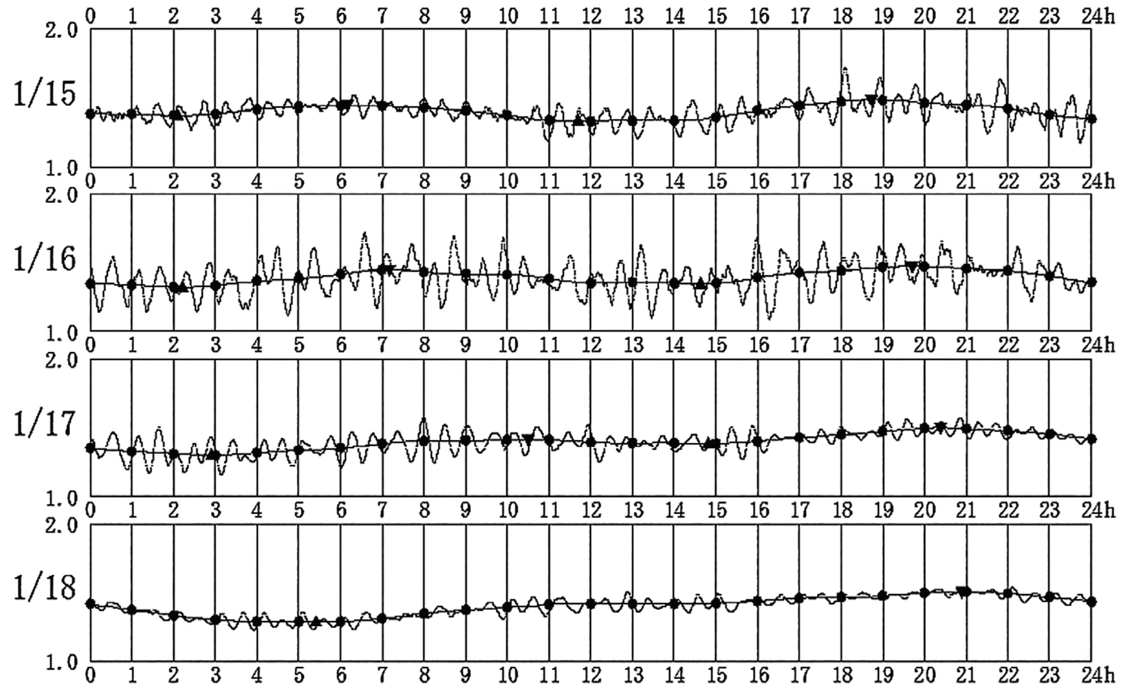


Fig. 2.3.15 Example of Raw Data Measured by NOWPHAS

③ Fifteen-Second Value, 30-Second Value, and 1-Minute Value

The 15-second value, 30-second value, or 1-minute value shall be created from the tide-level data of the measurement, and the acquisition interval of which is 1 minute or less. Although such values are basically obtained by averaging the tide levels measured within the interval, apparent abnormal values such as values outside of the specified range, same values repeating longer than the specified period, and values that are not consistent with previous or subsequent values shall be eliminated and calculated. If an abnormal value appears every hour, check the records at the same time to determine the cause.

This data is often used for examining harbor resonances or tsunamis with several minute or longer frequencies. Considering that some recording time intervals may make it difficult to understand short-frequency fluctuations, it is desirable to set the recording time interval on the basis of the frequency of the objective phenomenon on site. **Fig. 2.3.16** shows an example of 15-second interval data that show the mean value in the vertical axis.



(Frequency fluctuation curve of approximately 30 minutes shows 15-second values, smooth curve shows smoothed values, ● shows the hourly value, and ▲▼ shows high/low tide.)

Fig. 2.3.16 Example Graphs of 15-Second Interval Data

④ **Monthly Report of Tide Level (Every Hour Value)**

Calculate one-minute interval smoothed values by eliminating components with roughly three-hour or lower frequencies on the basis of the observation values created in the previous section. Make a monthly report of tidal data by setting the top of the hour smoothed value as an hourly value, the highest level on record as a high tide, the lowest level on record as a low tide, the mean value of hourly values in a day as a daily mean value, and the mean of daily mean value for one month as a monthly mean. **Table 2.3.3** shows an example of the monthly tide table.

Table 2.3.3 Example of the Monthly Tide Table

Monthly Tidal Motion Table Kurihama Jan. 2013

	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	Total	Mean	High tide		Low tide		High tide		Low tide		Moon phase
																									cm	cm	h m	cm	h m	cm	h m	cm	h m	cm	
1	114	109	119	141	169	197	221	234	235	226	211	193	179	173	177	190	207	224	235	235	224	203	177	150	4543	189. 2	7:38	236	0:48	109	18:31	236	13:06	173	
2	129	118	121	136	160	187	212	229	236	232	220	202	184	172	170	176	190	206	219	225	222	211	191	168	4516	188. 1	8:09	236	1:18	117	19:13	225	13:48	169	
3	146	130	125	132	148	172	196	216	227	228	219	203	184	169	161	161	168	180	193	203	207	204	195	179	4346	181. 0	8:36	229	1:55	125	20:10	207	14:31	160	
4	161	144	134	134	143	162	184	206	221	227	224	213	197	181	168	162	163	170	180	191	201	206	204	196	4372	182. 1	9:12	227	2:34	133	21:15	206	15:23	162	
5	185	172	161	155	158	168	184	202	218	228	229	222	209	193	178	166	160	159	164	172	182	193	199	201	4458	185. 7	9:40	230	3:13	155	22:51	201	16:36	159	
6	198	192	184	178	176	179	188	201	215	228	235	235	228	215	199	183	170	160	156	157	164	174	186	197	4598	191. 5	10:30	236	3:52	176	*****	***	18:14	156	
7	204	207	206	201	197	194	195	200	208	217	227	232	232	225	213	196	176	158	144	136	136	144	158	176	4582	190. 9	1:13	207	5:15	194	11:25	233	19:32	135	
8	193	206	215	217	214	208	203	201	203	209	218	227	232	233	228	215	195	172	148	128	118	118	128	147	4576	190. 6	2:55	217	6:59	201	12:41	233	20:30	116	
9	170	193	211	223	228	225	218	209	203	203	209	218	229	238	241	236	221	198	169	140	118	107	108	123	4638	193. 2	4:05	228	8:31	202	13:55	241	21:24	106	
10	146	175	202	224	237	239	233	221	208	199	198	203	215	230	242	246	239	219	189	155	123	99	89	94	4625	192. 7	4:47	240	9:43	197	14:56	246	22:12	89	
11	112	140	173	203	226	239	239	229	212	196	185	184	193	209	227	241	244	233	210	177	139	107	86	79	4483	186. 7	5:32	241	10:38	183	15:45	244	22:57	79	
12	88	111	144	179	210	232	241	237	222	203	186	178	182	197	217	237	250	251	237	210	174	134	102	85	4507	187. 7	6:10	241	11:08	178	16:34	252	23:37	82	
13	83	97	124	158	192	220	236	240	230	211	190	173	168	176	195	219	240	251	248	230	200	164	129	103	4477	186. 5	6:45	240	11:55	168	17:17	251	*****	***	
14	92	98	117	148	183	215	239	249	246	232	211	190	178	179	194	216	239	256	263	258	240	210	176	145	4774	198. 9	7:15	250	0:10	92	18:08	263	12:25	177	
15	123	117	126	148	177	208	233	248	249	238	219	196	178	170	175	190	212	232	246	248	239	219	192	165	4748	197. 8	7:36	250	0:55	117	18:43	249	13:08	170	
16	142	131	133	148	172	199	225	243	249	243	228	206	184	169	166	174	190	210	227	236	235	225	206	185	4726	196. 9	8:01	249	1:21	130	19:26	237	13:46	165	
17	164	151	148	156	172	194	218	236	246	245	234	215	194	177	167	167	176	190	205	216	222	220	211	197	4721	196. 7	8:25	247	1:45	148	20:18	222	14:28	166	
18	181	167	161	163	174	190	210	228	240	242	236	222	204	185	171	164	164	171	182	194	204	208	206	199	4666	194. 4	8:49	243	2:14	161	21:10	208	15:26	163	
19	188	179	173	172	177	187	201	216	227	233	232	224	210	194	178	167	161	162	166	173	182	189	193	194	4578	190. 7	9:23	234	2:38	172	22:40	194	16:28	161	
20	191	187	184	181	182	187	195	205	215	222	225	222	213	200	185	171	158	150	146	147	152	161	170	177	4426	184. 4	10:01	225	3:19	181	*****	***	18:16	146	
21	183	186	186	186	185	186	189	194	201	208	212	214	212	205	196	183	169	155	145	140	140	146	156	169	4346	181. 0	2:18	186	3:54	185	10:54	214	19:30	139	
22	181	192	199	203	204	204	204	206	209	215	222	227	229	228	224	214	200	183	165	150	141	141	148	159	4648	193. 6	12:19	229	20:31	140	*****	***	*****	***	
23	175	191	203	210	213	211	208	204	201	201	204	210	216	222	224	220	209	192	172	151	136	129	133	145	4580	190. 8	4:09	213	8:25	201	13:56	224	21:11	129	
24	164	185	205	219	227	228	223	216	208	204	204	209	218	228	235	238	231	216	194	169	146	131	128	136	4762	198. 4	4:36	228	9:22	203	14:48	238	21:48	128	
25	154	177	202	223	237	241	237	228	217	208	204	206	214	225	238	246	245	233	211	184	154	130	118	119	4851	202. 1	5:02	241	10:10	204	15:26	246	22:24	117	
26	133	156	183	208	227	237	237	228	214	200	190	188	194	208	225	241	248	243	225	197	165	134	113	105	4699	195. 7	5:30	238	10:48	187	16:08	248	22:59	105	
27	113	134	163	192	217	232	235	228	213	195	180	173	176	188	206	226	241	244	234	211	180	147	121	107	4556	189. 8	5:47	236	11:14	173	16:45	245	23:24	106	
28	109	125	152	184	213	234	243	238	224	204	185	171	169	178	195	217	236	246	242	225	196	163	132	110	4591	191. 2	6:10	243	11:43	168	17:15	246	23:58	103	
29	103	112	133	164	195	220	235	237	226	207	185	167	158	161	176	197	220	237	243	234	213	184	151	124	4482	186. 7	6:38	238	12:16	158	17:54	243	*****	***	
30	108	109	125	152	184	213	232	239	232	214	191	169	154	150	159	177	200	221	233	232	218	195	165	136	4408	183. 6	6:58	239	0:28	106	18:26	234	12:47	150	
31	115	107	114	134	162	190	213	225	224	211	190	166	146	136	138	151	172	195	212	221	218	204	182	157	4183	174. 2	7:23	226	1:02	107	19:15	221	13:20	135	

The asterisk (*) indicates missing data. Monthly mean tide level 190.1 cm (Measurement acquisition ratio 100.0%)

Calculating a tide-level deviation (actual value - prediction value) for the same period, drawing a graph, and checking the fluctuation state to validate the data are helpful in the preparation of a monthly table. Considering that the tide-level deviation does not contain a tide phenomenon, which is the primary component of the fluctuation of tide levels, the width of the fluctuation is small, and small fluctuations that cannot be noticed on the tide-level fluctuation graph are shown. Keeping a figure that contains the tide-level deviation not only for the organization but also as grounds for the calculation of the final results is convenient for examining singular phenomena other than the tide. The values in the monthly report of tidal data are often used to examine tide phenomena, storm surge, etc.

Fig. 2.3.17 shows an example of graphs indicating hourly values for a month, prediction values for the same period, tide-level variation values (every hour value - prediction value), and daily mean values.

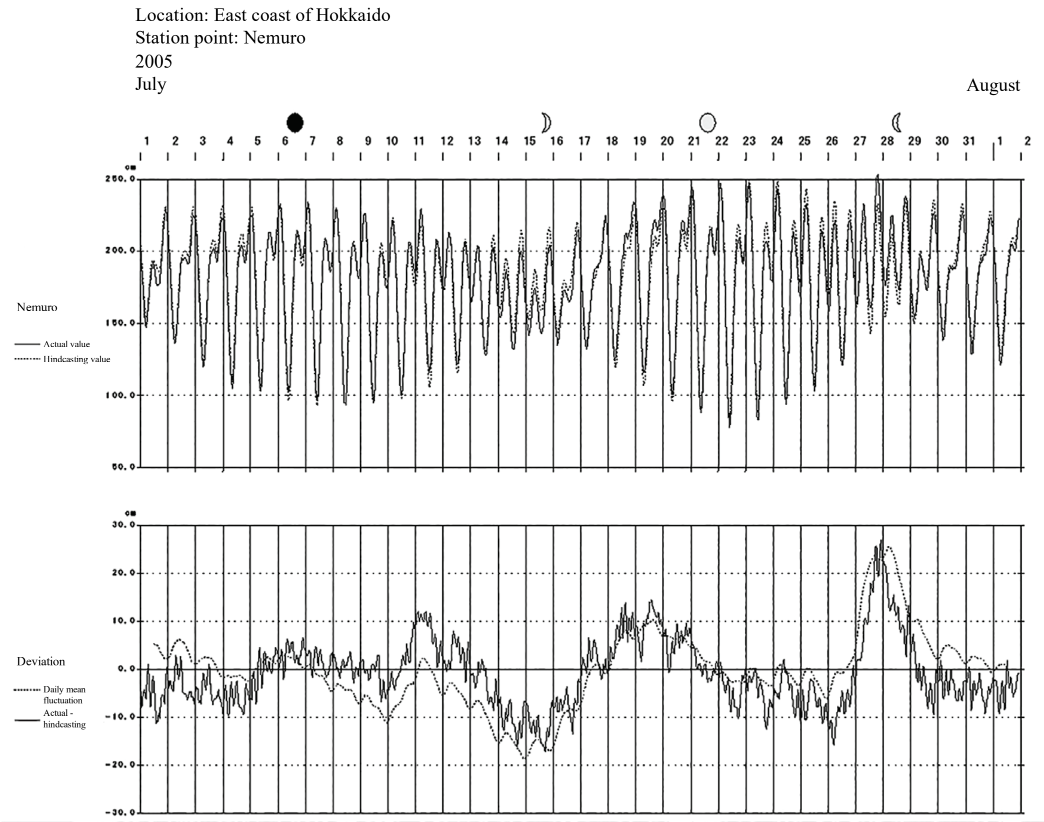


Fig. 2.3.17 Example of Graphs Indicating Hourly Values, Hindcasting Values, Tide-Level Variation Values, and Daily Mean Values

Considering that the daily mean values contain no one-day or lower periodic fluctuations, including the tide phenomenon, they are convenient for understanding abnormal tide levels or the fluctuation states of ocean currents. Strictly speaking, the daily mean values still contain the tide phenomenon, and it may be eliminated with a low-pass filter in calculation. **Fig. 2.3.18** shows an example of a time-series change chart of the daily mean value of the water level.

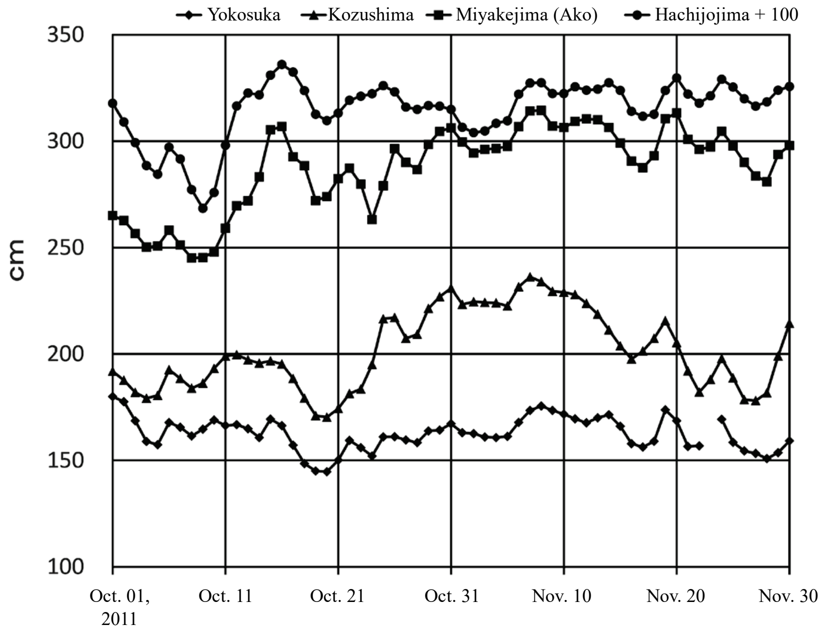


Fig. 2.3.18 Example of a Time-Series Change Chart of the Daily Mean of the Water Level Created by Obtaining Data Every Hour from the Japan Oceanographic Data Center (2016)
(The datum of Hachijojima is raised 100 cm for the sake of visibility.)

The fluctuations of the monthly mean sea levels in neighboring tide observatories are usually similar. The seasonal fluctuations along Japan's coast are between 20 and 30 cm and are higher in summer and lower in winter. **Fig. 2.3.19** shows an example of a time-series change chart of the monthly mean sea level. This figure indicates that the mean sea level fluctuates largely in Kozushima, Miyakejima, and Hachijojima owing to the effect of the meandering of Kuroshio. However, fluctuations in Mera, Chiba, and Okada, which are closer to Honshu, are very similar.

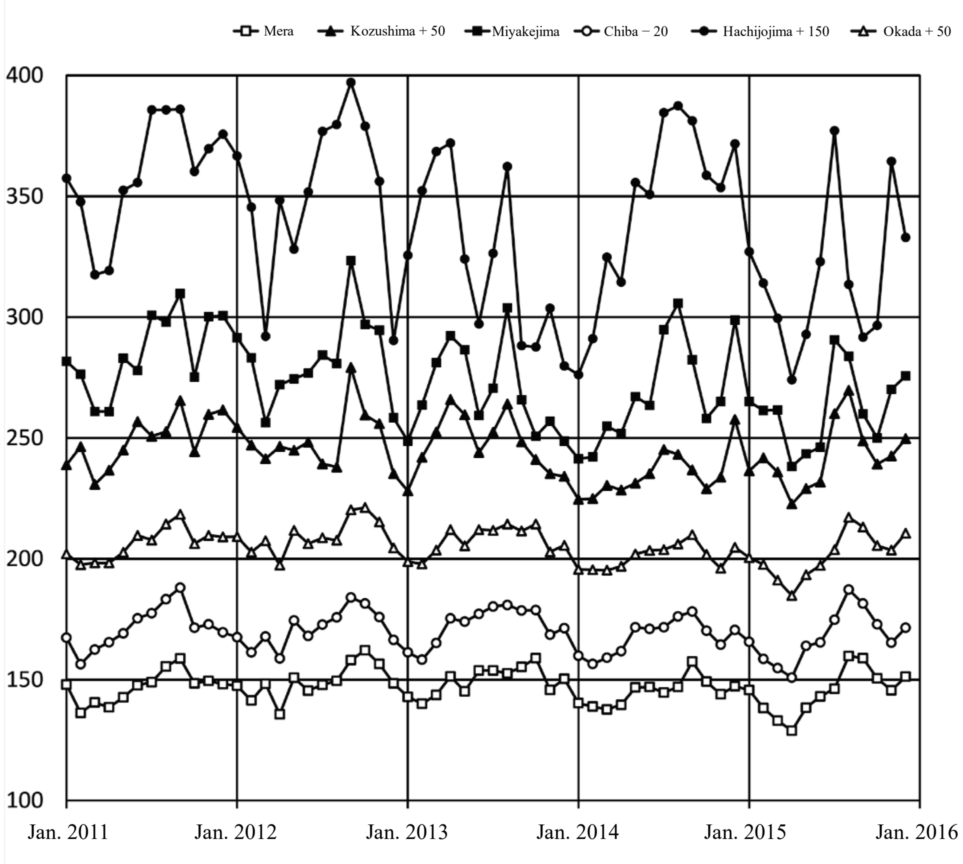


Fig. 2.3.19 Example of a Time-Series Change Chart of the Monthly Mean Sea Level

Created from the Data in the Coastal Movements Data Center (2016) and Annual Report of Tidal Data

(Some records are displayed by increasing the datum of the tide level for the sake of visibility.)

⑤ **Annual Report of Tidal Data**

Table 2.3.4 shows an example of the Annual Report of Tidal Data. Given that the monthly mean values, highest/lowest values in a month, highest/lowest values in a year, date of occurrence, mean monthly H.W.L./L.W.L., and date of occurrence are listed, it is convenient to check the date when an abnormal tide level occurred or update the maximum/minimum recorded tide levels.

Table 2.3.4 Example of the Annual Report of Tidal Level

Tidal level table

Jan. 2011 to Dec. 2011

Stipulated number of times	8760
Measured and acquired number of times	7776
Missed number of times	984 (11. 2) %

Name of the tide station　　Datum level: tide O.D.L.　　Unit: cm

Month	Monthly mean tide level	New moon tide level		Full-moon tide level		M.S.L. of spring tide		H.W.L.		L.W.L.		Missed measurement ratio (%)	Remarks
		High tide	Low tide	High tide	Low tide	High tide	Low tide	Day:Hours:Minutes	cm	Day:Hours:Minutes	cm		
1	118.5	182	41	181	27	182	34	6 15:40	182	20 22: 8	27	0.0%	
2	103.0	161	28	171	18	166	23	18 14:50	171	16 20:35	18	0.0%	
3	104.4	154	45	—	—	154	45	8 4:50	154	4 21:41	45	65.6%	
4	108.1	—	—	157	21	157	21	24 6: 0	160	22 11:53	21	68.9%	
5	109.5	163	19	173	13	168	16	21 4:23	173	19 10:11	13	0.0%	
6	115.7	169	23	172	19	171	21	18 3:26	172	16 9:27	19	0.0%	
7	120.1	178 175	32 38	173	41	175	38	4 4:17	178	2 9:56	32	0.0%	
8	126.0	185	55	183	61	184	58	12 1: 1	183	1 10:11	40	0.0%	
9	128.0	202	53	182	64	192	59	30 16:14	202	29 22:26	53	0.0%	
10	123.8	183	20	184	60	184	40	1 16:43	192	28 22:15	20	0.0%	
11	117.6	181	10	177	43	179	27	25 13:57	181	26 22: 2	10	0.0%	
12	113.2	180	20	166	20	173	20	24 13:32	180	26 22:35	20	0.0%	
Total	1387.5	2113	384	1919	387								
Year	115.6					175	34	Sep. 30 16:14	202	Nov. 26 22: 2	10	11.2%	

The new- and full-moon tide levels were processed as values of the month in which the new- and full-moon days as a basis of period are included and not as days when they occurred.

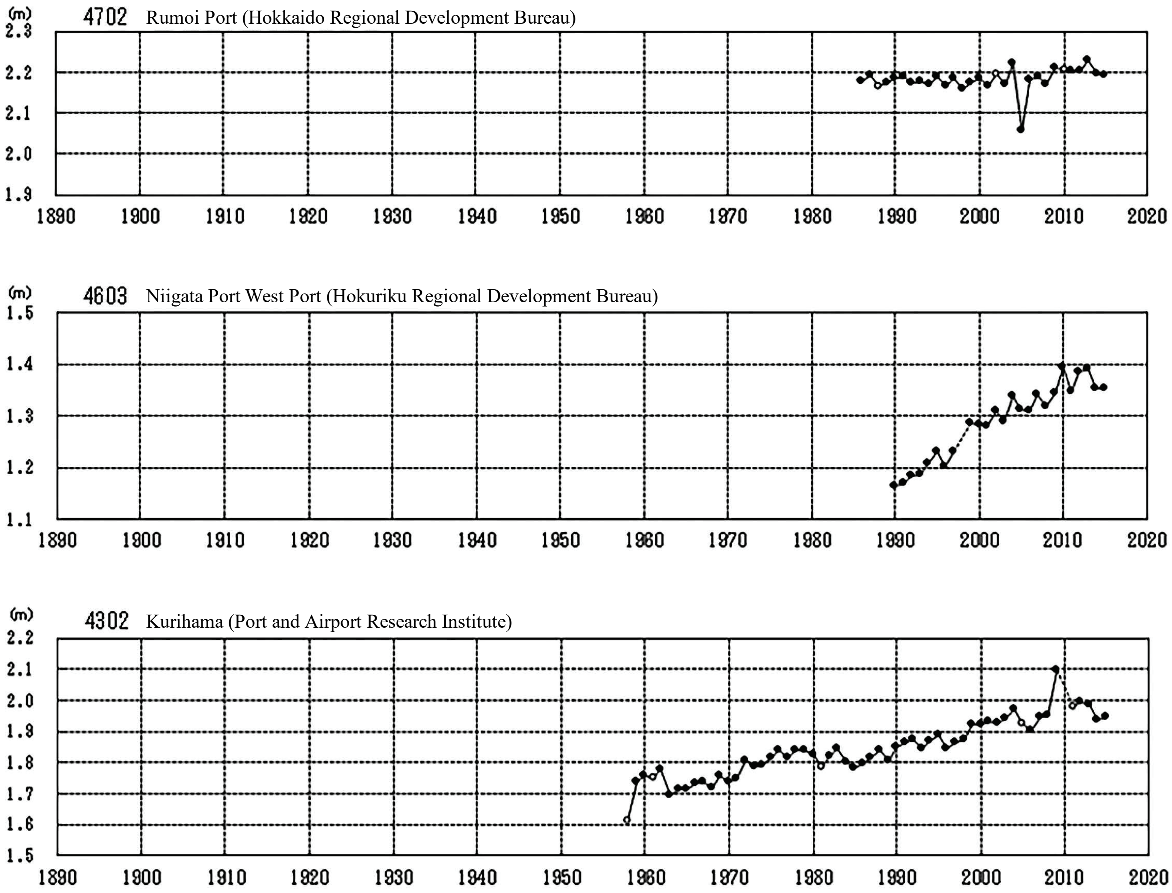


Fig. 2.3.20 Example of an Annual Mean Tide-Level Graph (Coastal Movements Data Center, 2016)

Fig. 2.3.20 shows an example of the annual mean tide-level graphs posted on the website of the Coastal Movements Data Center. The annual mean values of ground deformation or the change in sea level should be checked because they are deprived of seasonal fluctuations. These data are part of the measurement record of the difference in height between the ground and sea levels for some 10 years. It is affected by both the ground deformation and the change in sea level.

Given that the tide observation measures the difference in height between the ground and sea levels, the annual mean sea level similarly increases whenever the ground settles or the sea level rises. Given that the ground deformation and the change in sea level are difficult to strictly distinguish only from the tide observation record, local ground deformation is analogized unless the neighboring tide observatories show the same general tendency.

The ground deformation and the change in sea level may be distinguished by checking the relation with the T.P., but the elevation of the B.M. is not frequently updated. Even if it is updated, any fluctuation after the update of the B.M. that is becoming a base point is not reflected. Therefore, the T.P. is not considered accurate, and it was difficult to remove the ground deformation completely from the tide-level record by referring to the T.P.

However, the GNSS survey utilizing a satellite has become popular approximately 20 years ago. Considering that the GNSS survey is based on the coordinate system that makes the gravity center of the Earth its origin, basing the data on the GNSS survey made it possible to remove the ground deformation from the tide-level record.

(3) Results of the Tide-Level Data

① Tide-Level Harmonic Decomposition

The astronomical tide is the regular and periodic change in sea level primarily caused by the Moon's and Sun's gravitational pulls and can be considered a mix of many different periodic components called tidal constituents. **equation (2.3.2)** expresses the tide level as the function of time based on this idea.

$$h(t) = So + \sum_{i=1}^{60} fi\, Hi \cos(Voi + ui + \sigma it - Ki) \qquad \textbf{(2.3.2)}$$

where

h : Tide level

t : Elapsed time from the era time

So : Mean sea level

i : Tidal constituent number

fi : Astronomical argument of the tidal constituent i (amplitude correction term due to the change in the Moon's ascending node)

Hi : Amplitude of the tidal constituent i

Voi : Hour angle of the tidal constituent i at the era time

ui : Astronomical argument of the tidal constituent i (phase correction term due to the change in the Moon's ascending node)

σi : Angular velocity of the tidal constituent i

Ki : Lag of the tidal constituent

So, Voi, and σi in **equation (2.3.2)** are the default values. fi and ui are the correction terms called astronomical argument owing to the change in Moon's ascending node and others. They change in a long period but not so much on the order of one year. They can be precalculated on the basis of astronomical theory. Therefore, they are treated as default values in this equation. In the end, the above equation becomes a polynomial equation with 120 unknowns combining Hi and Ki. The amplitude and the lag of each tidal constituent can be calculated with the least squares method by using every hour tide-level values for one year.

This analysis method is called the one year harmonic decomposition for tide, and the calculated amplitude and lag of the tidal constituents are called the tidal harmonic constants. The number of tidal constituents (i.e., 60) is the number of tidal constituents obtained in the normal harmonic decomposition for 1 year.

② Tidal prediction

The tidal harmonic constant enables the calculation of the astronomical tide at any day and time. The tide level at time (t) is calculated by obtaining astronomical arguments (fi, Voi, ui) for the calculation period in advance and assigning them to **equation (2.3.2)** together with the harmonic constant (Hi and Ki). If the datum of the prediction tide level is equal to the datum level for port administration (the L.W.L.), adopt Zo as So. Considering that tidal prediction does not include the effect of atmospheric pressure, wind, harbor resonance, and others, it may be different from the actual tide level.

(4) Various Datum Levels for the Tide Level

There are many datum levels for tide, such as those primarily used for observation or maintenance, calculated from the analytical calculation, used for port construction or management of nautical chart, etc. Many of these datum levels have ambiguous names. For example, the following may be called D.L.: datum level for a tide-level table, O.D.L. (observation datum line; may be abbreviated to D.L.), construction datum level (D.L. as an abbreviation of design level), and L.W.L. (datum level of a nautical chart, C.D.L.). In this case, it is necessary to fully confirm which datum level the tide level is referring to. Given that the relation between datum levels may be different, the date of creation and sources shall be specified when making a relationship diagram of any datum levels.

① **Datum Levels Concerning the Observation**

The tide gauge usually records the tide-level relative to the O.D.L. Considering that the O.D.L. is a virtual datum level, check the differences in height of each section from the fixed point to the port B.M. via the standard mark, and associate levels up to the port B.M. The measurement of the difference in heights to Geospatial Information Authority of Japan's B.M. or the GNSS observation point enables the calculation of the tide-level relative to the T.P. or an ellipsoid. **Fig. 2.3.21** shows the datum level concerning the tide-level observation.

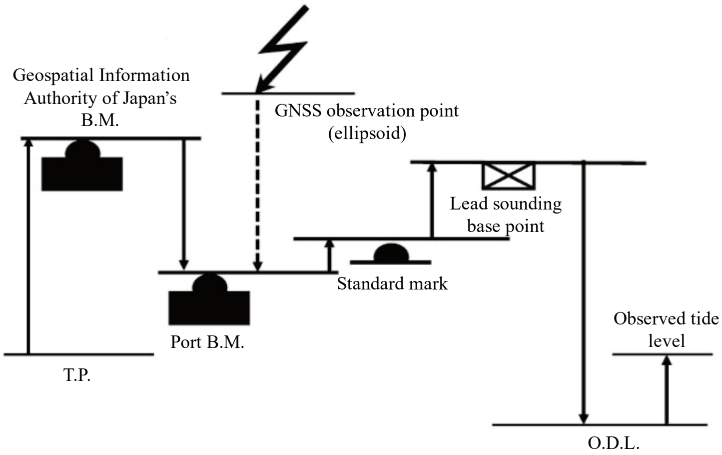


Fig. 2.3.21 Datum Level Concerning the Tide-Level Observation

② **Datum Level Obtained from Analysis**

The characteristics of the tide level can be understood by obtaining the following various datum levels from the results of long-term tide level observation. **Fig. 2.3.22** shows the principal datum levels obtained from analysis.

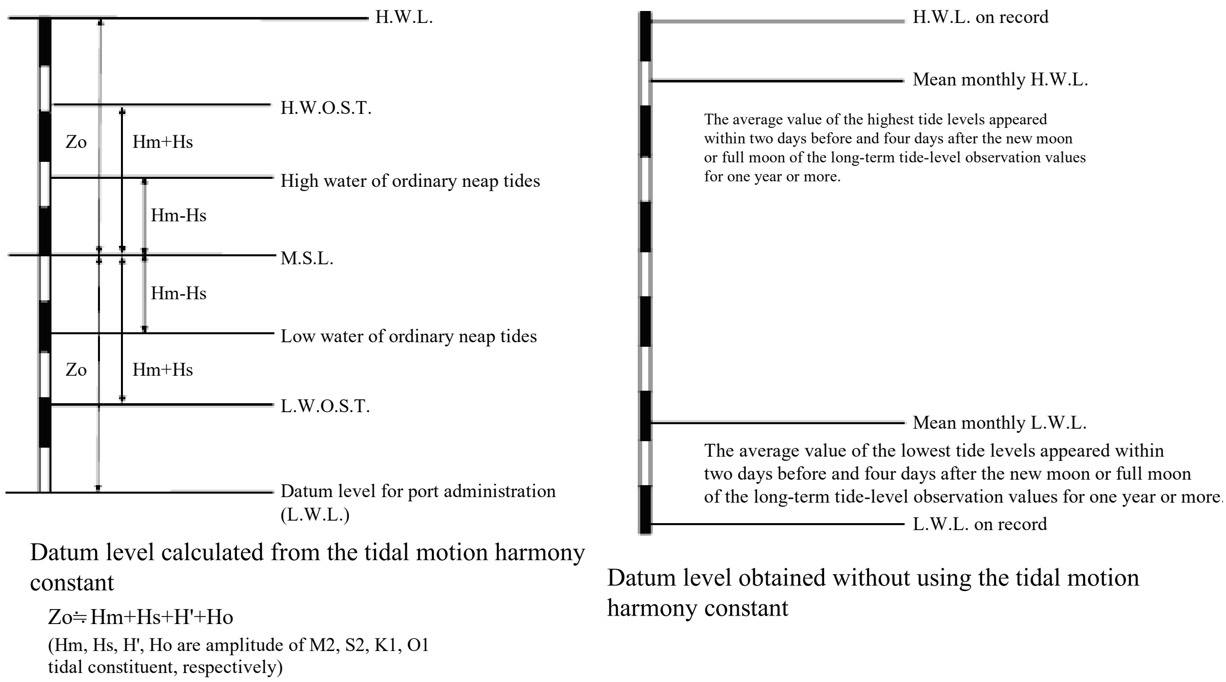


Fig. 2.3.22 Principal Datum Levels Obtained from Analysis

③ Datum Level Concerning the Construction

The fundamental datum level for the port construction is the datum level for port administration (C.D.L.). The datum level for port administration is set by the following equation after discussion with the Japan Coast Guard to coincide with the L.W.L., which is the water depth datum level on the nautical chart. Once it is set, it is not changed unless M.S.L. fluctuates beyond the adequate range.

$$C.D.L.=M.S.L.-Zo \tag{2.3.3}$$

where

C.D.L. : Datum level for port administration (L.W.L.)

M.S.L. : Mean sea level (in principle, the mean value for the last five years)

Zo : Value specified by the Japan Coast Guard on the basis of the sum of amplitudes of the four principal tidal constituents

The mean sea level (M.S.L.) is the value when C.D.L. is set. The latest value that the Japan Coast Guard specifies is used for *Zo*. The specific level is presented as the level on C.D.L. of the summit of the port B.M.

C.D.L. shall be monitored every year by using the latest M.S.L. to check if it is still in the range because it may deviate from the adequate range owing to ground deformation, fluctuation of the sea level, and other reasons. If it deviates ±10 cm or more from the previous C.D.L., reexamination is needed.

④ Actual Tide-Level Chart

A chart displaying the relation among various datum levels is called an actual tide-level chart. The displayed datum levels vary considerably depending on the application. The example in **Fig. 2.3.23** shows levels relative to the O.D.L. (0), but there are cases wherein the level is displayed relative to the datum level for port administration (C.D.L.) or the T.P. or wherein levels relative to several datum levels are also described.

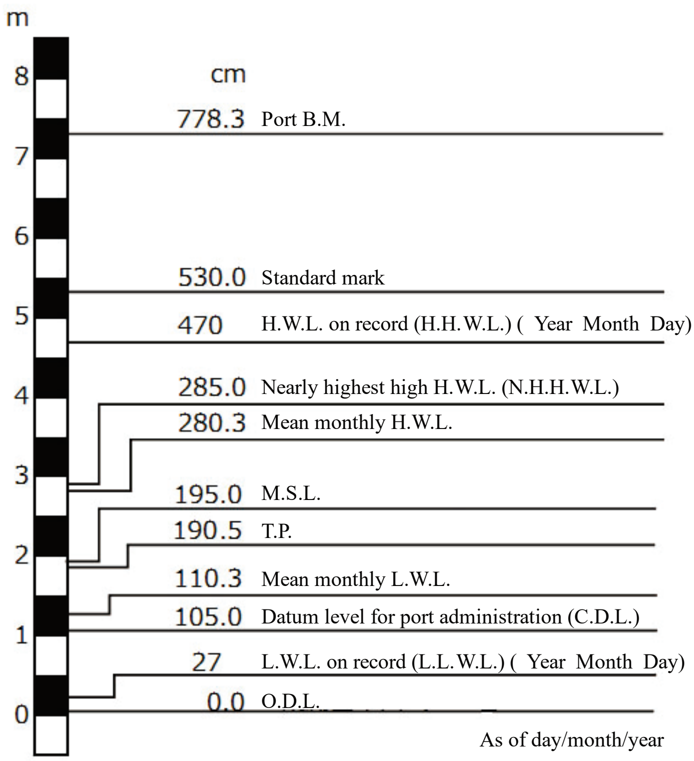


Fig. 2.3.23 Example of an Actual Tide-Level Chart

(5) Obtaining Previous Documents Related to the Tide-Level Observation and the M.S.L.

Tide-level documents can be collected on the following sites via the Internet. Reference [9] shows examples of previous documents that are useful for the organization and analysis of the tide-level observation and the M.S.L.

① **NOWPHAS** [13]

http: //www.mlit.go.jp/kowan/nowphas/

Data from the NOWPHAS-registered tide observatories is transferred from the site to the National Institute for Land and Infrastructure Management of the Ministry of Land, Infrastructure, Transport and Tourism and is posted on the NOWPHAS website in a tide-level graph for the last seven days. Past digital data are not available to the public via the website as of the writing of this article. The tide-level data managed by Regional Development Bureaus and others shall be basically collected by requesting the data from the Regional Development Bureau and others that manage the data.

② **Japan Meteorological Agency** [14]

http: //www.jma.go.jp/jp/choi/

For the real-time tide level, tide-level graphs for the last eight days and every hour values, high-/low-tide water value and others on the tide-level tables for several years observed in tide observatories administered by the Japan Meteorological Agency, Japan Coast Guard, Port Authorities, Geospatial Information Authority of Japan, autonomous bodies, and others can be viewed on the website of the Japan Meteorological Agency. In addition to these, the tide observatories administered by the Japan Meteorological Agency provide the following for viewing: hourly values and high-/low-tide water values (including preliminary figures) for the past dozen years, list of harmony constant used for the calculation of the latest tide-level table and mean sea level, datum level of the tide-level table, previous highest (lowest) tide-level records, and so on.

③ **Hydrographic and Oceanographic Department of the Japan Coast Guard** [15]

http: //www1.kaiho.mlit.go.jp/KANKYO/TIDE/real_time_tide/sel/index.htm

Tide observatories administered by the Japan Coast Guard and the Japan Meteorological Agency provide the tidal curves and the digital tide levels every five minutes for the last seven days for viewing. The level reference is the L.W.L. The monthly reports for the past several years can be viewed on the Hydrographic and Oceanographic Department website of the Regional Maritime Safety Headquarters.

http: //www1.kaiho.mlit.go.jp/KANKYO/TIDE/enkan/Suijun_hyo/Pub.No741/Top.htm

The Japan Coast Guard manages and publicizes nautical chart water depth datum level and others for the whole country. Given that this datum level usually coincides with the datum level for port administration, this is the most effective document to check the datum level for port administration.

④ **Geospatial Information Authority of Japan** [16]

http: //www.gsi.go.jp/kanshi/tide_index.html

Every hour values (old document may provide high-/low-tide values) from the commencement of observation until the previous month of the viewing day and the 30-second interval digital data from 10 years ago until the viewing day for tide observatories administered by the Geospatial Information Authority of Japan can be obtained. The datum level can be selected from T.P. and the O.D.L. Given that the tide observatories are always conducting the GNSS survey since 2002, it is possible to separate the sea level rise and the ground deformation by using this data.

http: //terras.gsi.go.jp/

⑤ **Coastal Movements Data Center** [17]

http: //cais.gsi.go.jp/cmdc/centerindex.html

The Coastal Movements Data Center promptly collects records in a unified manner from tide observatories established by many national institutions for their own purposes. The monthly mean data and graphs, associated water level results table, and others of the tide observatories from their commencement until the latest period administered by the Japan Meteorological Agency; Japan Coast Guard; Geospatial Information Authority of Japan; Hokkaido Regional Development Bureau; Ports and Harbours Bureau of the Ministry of Land, Infrastructure, Transport and Tourism; Earthquake Research Institute of the University of Tokyo; Regional Agricultural Administration Office; Okinawa General Bureau; local authorities; and others can be viewed.

⑥ **Japan Oceanographic Data Center** [18)]

http: //jdoss1.jodc.go.jp/vpage/tide_j.html

The Japan Oceanographic Data Center collects, manages, and provides ocean data and information obtained by national oceanographic survey organizations. For tide-level data, every hour data and attribute information concerning the datum level of the Port Authorities, the Geospatial Information Authority of Japan, the Japan Meteorological Agency, and the Japan Coast Guard can be viewed. The 30-second interval tide-level data from the tide observatories administered by the Japan Coast Guard can also be obtained.

⑦ **Other websites**

Many websites provide tide-level documents. Typical URLs that post tide-level documents are listed below:

Chugoku Regional Development Bureau: http: //www.bousai.cgr.mlit.go.jp/cyoui/

Kyushu Regional Development Bureau: http: //www.qsr.mlit.go.jp/bousai_joho/choi.html

Bureau of Port and Harbor. Tokyo Metropolitan Government:
http: //micos-sa.jwa.or.jp/metro/tokyop/topframe.htm

Osaka Prefectural Government: http: //www.osaka-kasen-portal.net/suibou/bunpuzu/tyouibunpu10_0.html

River Disaster Prevention Information System of Hiroshima Prefecture:
http://www.kasen-bousai.pref.hiroshima.lg.jp/rivercontents/p10501/10/1_1_1_0.html

Okayama Disaster Prevention Center: http: //www.bousai.pref.okayama.jp/bousai/observation/

2.4 Observation and Examination of Ocean Waves

2.4.1 Overview

The purpose of observing ocean waves is to collect observation data on required items (wave height, period, wave direction, etc.) for the required observation period (actual status values, in the order of several years, in the order of several tens of years, etc.) for the following purposes:

① Development of a port plan (evaluation of harbor calmness, etc.)
② Design of structures (calculation of external forces, etc.)
③ Development of a construction plan (verification of construction methods, etc.)
④ Safety management of construction works (safety management based on the actual status of ocean waves, etc.)
⑤ Recovery of disaster-stricken facilities (evaluation of subjected external forces in disaster, etc.)

④ above and actual status data are enough for the ocean wave observation data and the safe navigation of ships, respectively. For other purposes, data for a certain period are required for statistical analyses for the above purposes.

2.4.2 Observation of Ocean Waves

The following items need to be considered when observing ocean waves:

(1) Observation Items and Period

① Observation items are the wave height, the period, and the wave direction.

② The standard observation period is about five years or more for the evaluation of harbor calmness and about 30 years or more for the evaluation of the stability of structures. For details, see **Part II, Chapter 2, 4 Ocean Waves**.

(2) Observation Site Representing a Water Area

Observation sites shall be selected according to the purpose of observation. For the purposes listed in **2.4.1 Overview**, it is best to observe at the closest sites for each, but it may be practically difficult due to the cost, coordination with persons concerned, and so on. Therefore, observations must be made at sites representing the water area and the ocean wave in the whole water area must be estimated based on the observation results and considering wave shoaling, reflection, diffraction, etc.

(3) Selection of an Observation Site

The following points need to be verified while selecting an observation site to ensure its representativeness:

① Is the principal wave direction seized? Is it open to the principal direction from which waves seem to attack or of the waves that exert the most significant impact?

② Is there any impact from existing structures? Also consider structures in the planning stage.

③ Is the bottom topography complicated? Select flat bottom topography.

④ Is the ground firm enough to support the installation of observation equipment? If not, consider the design of frames, etc.

⑤ Will the movement of sediment bury the observation equipment? If possible, consider the frequency of the maintenance work, design of frames, etc.

⑥ Is the site subjected to the influence of sailing ships or fishing activities? Consider the effect of ship waves. Will the observation equipment be damaged by the fishing activities? On the contrary, will fishing nets and/or fishing gear be damaged by the observation equipment?

(4) Selection of Observation Equipment

Tables 2.4.1 and **2.4.2** show the characteristics of currently available ocean wave observation methods and data transmission methods of the observation equipment installed on the sea bottom. Select the observation equipment most suitable for the observation purpose based on these tables.

(5) Utilization of the Existing Data

The Ports and Harbours Bureau of Ministry of Land, Infrastructure, Transport and Tourism, the Japan Meteorological Agency, local authorities, power companies, and others are observing ocean waves along the coast of Japan. The observation data of the state and local authorities are essentially made public and some real-time data may also be available. As described later, Nationwide Ocean Wave Information Network for Ports and HArbourS (NOWPHAS)[1)] has the most fulfilling observation network, data management, and analysis structures among these.

In planning and implementing the ocean wave observation, first consider the utilization of the existing data, and then complement insufficient data with the individual observation. Existing high-quality data like NOWPHAS also helps check the reliability of the individual observation data.

2.4.3 Observation Equipment of Ocean Waves

(1) Selection of Ocean Wave Observation Methods

Principal ocean wave observation methods are as follows:

Table 2.4.1 Ocean Wave Observation Methods

Observation item	Installation location	Measuring principle	Characteristics, etc.
Wave height/period	Fixed (on the sea bottom)	Water pressure type	Not prone to be affected by wave breaking.
		Ultrasonic type	High accuracy, but prone to be affected by wave breaking. The cable-laying method enables stable observation, but the installation cost is relatively high.
	Fixed (in the air)	Ultrasonic type	Easy maintenance and inexpensive installation cost. Not suitable for observations of offshore waves.
	Fixed (in the sea)	Step type	No calibration is needed.
		Capacitive type/resistance wire type	Not prone to be affected by wave breaking.
	On the sea (buoyage type)	Accelerometer type	Less restriction in the selection of the installation site (water depth). Some are capable of the wave direction observation.

Observation item	Installation location	Measuring principle	Characteristics, etc.
		Measurement with Global Navigation Satellite System (GNSS)	"Global Positioning System (GPS) wave gauge" Scarcely restricted in the selection of the installation site (water depth). The wave direction observation is also possible if the period is within a certain range.
Wave height/ period/wave direction	Fixed (on the sea bottom)	Ultrasonic type wave gauge + multilayered Doppler velocimeter	"Doppler-type Wave Direction Meter." The directional spectrum observation is also possible. The cable-laying method enables stable observation, but the installation cost is relatively high. The standard equipment of the Nationwide Ocean Wave Information Network for Ports and HArbourS (NOWPHAS).

Furthermore, in addition to the equipment listed in **Table 2.4.1**, shortwave ocean radars, microwave radars, and so on, which are usable as ocean wave observation equipment, are being developed.

Shortwave ocean radars obtain the wave height and period by emitting electromagnetic waves on the sea surface and measuring the spectrum of the backscattered wave modulated by the Doppler effect. This observation method is characterized by installing two or more onshore observatories and obtaining the ocean wave information in the overlapped area of their observation ranges.

Unlike the ocean wave observation at fixed sites with direction wave meters and other equipment, this observation method features providing information on the planar distribution of ocean waves. Utilization in environmental and disaster prevention fields is investigated as this observation method is capable of providing information on flow in a broad area and the ocean waves at the same time.

The Japan Meteorological Agency observes the ocean waves at six sites in Japan using microwave radars. The radars obtain the significant wave height, the significant wave period, and the wave direction by emitting the microwave from the shore toward the sea surface and measuring the reflected wave modulated by the Doppler effect in accordance with the movement of the sea surface due to ocean waves. Although the measurement principal to measure reflected waves modulated by the Doppler effect is the same as the shortwave ocean radar, the microwave radar is more advantageous in that only one onshore observatory is needed as well as an antenna that is smaller in size.

Table 2.4.2 lists the transmission methods of the observation data from the sensor portion of the ocean wave observation equipment installed on the sea bottom to the onshore station. An adequate method shall be determined considering the observation period, the way to use the observation data, etc.

Table 2.4.2 Observation Data Transmission Methods from the Observation Equipment on the Sea Bottom

Data transmission method	Advantages	Disadvantages
<Direct recording method> Observation data are stored in the observation equipment installed on the sea bottom and collected after the end of the observation. Suitable method for short-term observation.	Simply installable and relatively easily coordinated with fishermen, etc. Inexpensive in installation cost.	Observation data cannot be utilized until they are collected. Whether the observation is normally done or not is not checkable. Necessity of periodical salvaging by a diver makes this unsuitable for long-term observation.
<Submarine cable method> Observation data are transmitted onshore via the submarine cable.	Observation data can be utilized stably throughout the observation period.	The installation cost is the highest depending on the cable length. Coordination with fishermen is required. There is a risk of damage to the cable by anchorage or fishing gear.

Data transmission method	Advantages	Disadvantages
<Radio transmission method> Observation data are transmitted onshore via the tower just above the observation equipment or the antenna on a buoy.	Observation data can be utilized throughout the observation period.	The installation cost is lower than the submarine cable method. Coordination with fishermen and others is easier than in the case of the submarine cable method.

When laying submarine cables, check the status of ship navigation and the fishing activities on the route of the cable in advance and determine the cable structure (armoring structure) and the depth of burial. It is desirable to bury the cable in sandy soil. Moreover, a protecting duct shall be used in a rocky shore and the rising area from the sea to the shore where strong waves strike.

(2) Kinds and Characteristics of the Ocean Wave Observation Equipment

The kinds and characteristics of the major ocean wave observation equipment currently used for observation are mentioned below.

① **Water pressure-type wave gauge**

Step-type wave gauges, water pressure-type wave gauges, and others were used in Japan for the steady observation of ocean waves until the 1950s, but the mainstream was the water pressure-type wave gauge that measures fluctuations in pressure on the sea bottom because it required no facilities such as an observation tower.

The water pressure-type wave gauges are characterized by a simple structure and low cost. But how to correct the decreased sensitivity for short-period waves was a problem since the movement of water particles due to deep water waves does not reach the sea bottom.

However, because a method to convert the fluctuation in water pressure to the surface wave profile with high accuracy was developed and improved in recent years, the utilization range of the water pressure-type wave gauges capable of observing ocean waves simply and inexpensively is re-expanding.[19)]

The structure of the water pressure-type wave gauges has been improved during the past 50 years in the following two ways:

(a) Measurement of the absolute pressure

Traditionally, the structure to cut the long-period wave component by a mechanistic filter and detect only the wave height component was widely adopted. This method was characterized by high sensitivity, regardless of the water depth of installation. However, because high resolution was achieved thanks to the recent improvements in the water pressure sensor described in **(b)**, the absolute pressure measurement capable of simultaneous measurement of the ocean waves and the long-term fluctuation of the water level is now prevalent.

(b) Structure of the water pressure measurement

The water pressure measurement method has been evolved from the PW type, which converts the stroke motion of metal bellows to the rotary motion to drive a potentiometer, to the strain gauge type which receives power from the strain gauge assembled in the bridge and the pressure detection sensor type which use a semiconductor pressure sensor. The adoption of this method reduced the power consumption and achieved one-year free maintenance because of small variations in sensitivity due to the attachment of algae, etc.

② **Ultrasonic-type wave gauge (Sea bottom installation type)**

The ultrasonic-type wave gauge (USW) was developed in the 1960s. As the USW can get a direct water surface waveform compared to the water pressure-type wave gauges, it came to be used at every site. This USW gauge comprises a transducer that is a sensor installed on the sea bottom, a converter (the main body of the wave gauge) that is installed onshore, and a submarine cable connecting the transducer and the converter.

The transducer installed on the sea bottom emits ultrasonic pulses at 200 kHz vertically upward at short time intervals (in the order of 0.5 s). These pulses proceed to the sea surface at the speed of sound in the sea (about 1,500 m/s) and the transducer receives the ultrasonic pulse reflected by the sea surface. The distance from the transducer to the sea surface can be obtained by measuring this propagation time.

The fluctuation of the water level can be continuously obtained by continuously measuring the distance between the transducer and the sea surface at short time intervals (in the order of 0.5 s), and the arithmetic processing with the zero-up-crossing method and so on gives the wave height and period.

Although this method has a flaw that makes the detection of the sea level position difficult under the condition that many air bubbles are swallowed near the sea level by wave breaking and so on, this method is advantageous in that it can directly measure the surface wave profile accurately and relatively inexpensively without constructing a tower and others as a steady ocean wave observation. It can also be applied to deep water ocean wave observation sites and there are many installation examples at sites in the order of 50-m depths.

Moreover, a composite-type USW gauge (**Fig. 2.4.1**) was developed recently, which can continue observation with an annexed water pressure sensor at the time of wave breaking to complement a problem of the USW gauge that it is vulnerable to wave breaking.

Fig. 2.4.1 Example of an Ultrasonic-Type Wave Gauge (USW-150; Source: Manufacturer's Catalog)

③ **Ultrasonic-type wave gauge (Air emission type)**

An air emission-type USW gauge emits ultrasonic pulses from the transducer that is a sensor installed in the air to the sea surface, contrary to the sea bottom emission type. The distance from the transducer to the sea surface can be obtained by measuring the propagation time from the reflection of this pulse on the sea surface to the wave received by the transducer, and the wave height and period can be obtained by the arithmetic processing similar to that for the sea bottom installation type.

As the transducer must be installed in the air, this type is suitable for observations on the front side of a quaywall and others, but not for observations of the offshore wave. On the other hand, installing the transducer in the air finds merit in easy installation, removal, and maintenance.

It should be noted that the temperature in the air varies more than that in the water and this becomes an error factor. Furthermore, because the wind velocity may reach some percentage of the rate of ultrasonic progress (about 340 m/s), the wind effect becomes another error factor.

Fig. 2.4.2 Example of an Ultrasonic-Type Wave Gauge (Air Emission Type) (US-500; Source: Manufacturer's Catalog)

④ **Step-type wave gauge**

The step-type wave gauge has mutually insulated and vertically aligned electrodes at regular intervals to detect the sea level stepwise and obtains the wave height and period with subsequent arithmetic processing, by utilizing the property that the electrode becomes electrically ON when submerged in the sea and OFF when exposed in the air.

This structure is advantageous because this type of wave gauge needs no calibration.

⑤ **Capacitive-type wave gauge or resistance wire type wave gauge**

The capacitive-type wave gauge is equipped with a wire coated by dielectric materials and installed vertically from undersea to above the sea level. The electric capacitance between this wire and the sea water varies according to the ups and downs of the water level. On the other hand, the resistance wire-type wave gauge is equipped with a resistance wire installed vertically from the sea level to the undersea. The short circuit length of the resistance wire varies according to the ups and downs of the water level. These wave gauges detect the sea level by measuring the change in electric capacitance or short circuit length respectively, and obtain the wave height and period with subsequent arithmetic processing. Both methods have an advantage in the linearity and responsibility of output.

⑥ **Buoyage-type wave gauge (Accelerometer type)**

This type of wave gauge moors a buoy with a mooring rope so that the buoy can move freely on the sea level, measures the change in water level by detecting the vertical acceleration of the buoy, and obtains the wave height and period with subsequent arithmetic processing.

The accelerometer held by a gimbal mechanism detects the vertical acceleration. Then, the displacement in the extending direction is detected by integrating the vertical acceleration two times.

The buoyage-type wave gauges have few restrictions in the installation location and the installation cost is relatively low since they need no cables for power supply and data transmission. However, the influence on accuracy due to the effect of a mooring rope and the measuring principle to obtain the displacement by two integration operations of the acceleration are a drawback as they are not suitable for observations of long-period waves.

What resolved these problems and enabled a wide-ranging offshore ocean wave observation with long-term stability is the Global Positioning System (GPS)-based GPS wave gauge described below.

⑦ **GPS wave gauge (Buoyage-type wave gauge (GPS type))**

(a) Configuration

The measuring principle to use an accelerometer as a sensor and obtain the change in water level by integrating the vertical acceleration twice posed an essential problem to the buoyage-type wave gauge as it is not suitable for observations of long-period waves. However, the utilization of the GPS developed by the U.S. to support the navigation of aircraft, ships, and other vehicles induced the development of the GPS wave gauge enabling observations of change in water level in a wider range, including long-period waves and tsunamis without any restrictions in the installation depth.

The GPS wave gauge comprises a buoy part installed on the sea and an onshore system. **Fig. 2.4.3** shows an example of the buoy part of a GPS wave gauge installed at the depth of −204 m off the coast of Nanbu, Iwate Prefecture, in April 2007.

Fig. 2.4.3 Buoy Part of a GPS Wave Gauge

The buoy is moored to an anchor or a sinker (when the anchor holding power is not expected) on the sea bottom with a chain (a parallel line cable may be used partially depending on the installation depth).

The ocean wave observation with a GPS wave gauge analyzes the positioning data obtained on the buoy. Currently operated GPS wave gauges use a positioning method called the real time kinematic (RTK). The RTK method is capable of measuring the position of an antenna installed on a buoy to an accuracy of several centimeters, but error factors caused by the ionosphere or water vapor in the atmosphere must be eliminated. Therefore, the data obtained from reference stations whose latitude, longitude, and height are known are used as the correction information.

Fig. 2.4.4 shows the configuration of a whole GPS wave gauge system as of installation including the correction data. The onshore station behaves as the reference station described above.

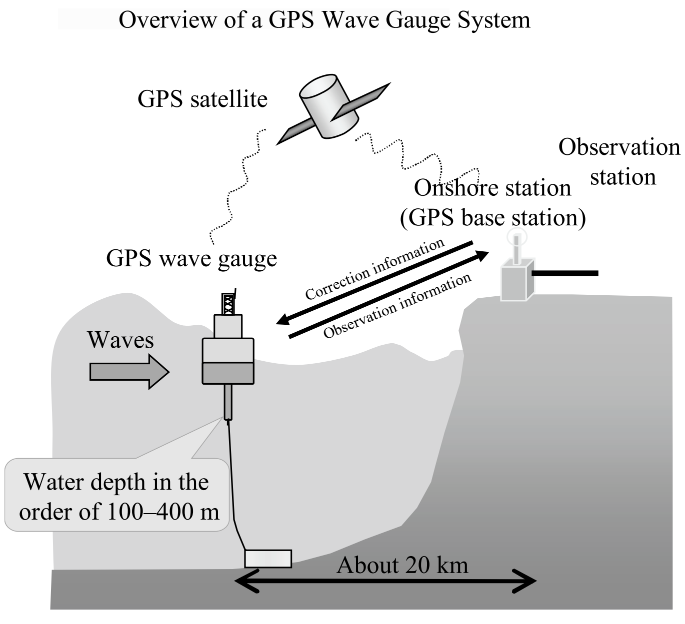


Fig. 2.4.4 Overview of a GPS Wave Gauge System

The tsunamis due to the earthquake off the Pacific in Tohoku region in 2011 caused devastating damage to port facilities, but as is well known, the tsunami's waveform was grasped by the GPS wave gauge and greatly contributed to the updating of warnings and evacuation activities.

However, the functions of observatories located near the seashore and of the communication networks to transmit information to the observation center or the Japan Meteorological Agency were lost on the arrival of tsunamis, while the functions of onshore stations (reference stations) located at places on high elevation with good visibility were maintained after the arrival of tsunamis (however, the observation data were stored in onshore stations (reference stations) and contributed greatly to subsequent analysis and verification).

Therefore, as shown in **Fig. 2.4.5**, improvements in multiplexed communication lines, securement of emergency power sources, enhancement of distribution servers, and others have currently been achieved so as not to lose functions even if communication is interrupted by floods, submergence, etc.

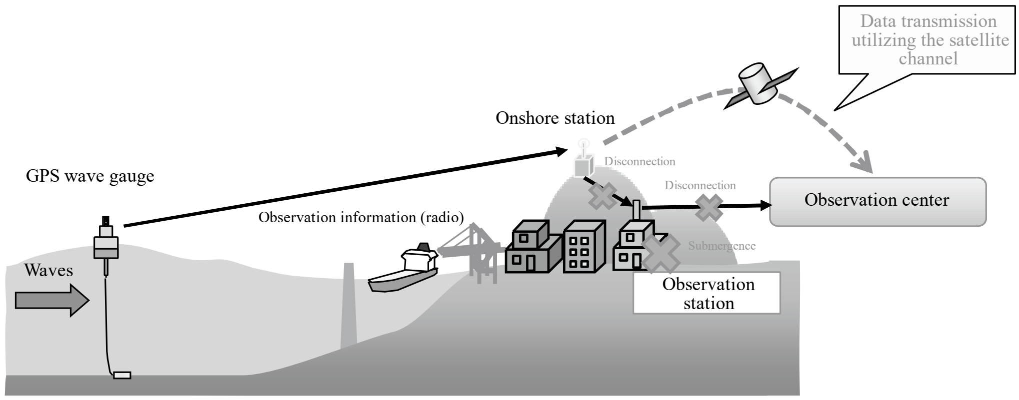


Fig. 2.4.5 Improved GPS Wave Gauge System

(b) About the observation data obtained by the GPS wave gauge

As the positioning sensor (GPS receiver) installed in the GPS wave gauge provides three-component positioning data, not only vertical positioning data but also two-directional horizontal data can be obtained. However, the horizontal movement of water particles due to waves does not necessarily coincide with the movement of a buoy as a floating body, and thus the fluctuation response characteristics of the buoy itself need to be considered.

The comparison between observation and calculation values in the vertical direction of the buoy gave a response magnification factor around 1.0, except for a short-period region where the significant wave period is 6 seconds or less. This shows that the GPS wave gauge properly observes the wave height. On the other hand, the deviation between observation and calculation values for a region where the significant wave period is 6 seconds or less brought up the necessity to correct the response of a buoy.[20)]

⑧ Ultrasonic velocimeter-type wave gauge (CWD)

The ultrasonic velocimeter-type wave gauge determines the wave direction with the arithmetic method called the covariance method from three components comprising the velocity of water particles in two-directional horizontal components due to ocean waves and the water pressure measured with the annexed hydraulic gauge. The velocity of water particles is measured utilizing the phenomenon that the propagation velocity of the ultrasonic wave propagating between two points in flowing water changes according to its flow velocity component.

As the transducer of this wave gauge is installed on the sea bottom, the attenuation of the movement of water particles makes the observation more difficult as the installation depth increases when the wave height is low or the wave period is short. It is common to install this type of wave gauge at 30-m or shallower depths when its purpose is to measure the wave direction.

Although the data observed by this wave gauge have a problem that the influence of low-attenuating long-period waves in the depth direction is exaggerated, this wave gauge is used widely as the most common gauge to observe coastal waves.

⑨ **Direction wave meter**

The direction wave meter developed in 1995 and put to practical use has been utilized in the observation of Japan's coastal waves since then as the NOWPHAS-standard oceanographic observation equipment. In recent years, it is also utilized in the observation of long-period waves and tsunamis.

The direction wave meter combines functions of the traditional USW-type gauge to be installed on the sea bottom and of the multilayered Doppler velocimeter, and is largely characterized by its ability to measure the wave height, period, wave direction, and directional spectrum with a single observation equipment even at very deep waters of −50 m.

Figs. 2.4.6 and **2.4.7** show the undersea sensor portion and the measuring principle of a direction wave meter, respectively.

Fig. 2.4.6 Undersea Sensor Portion of a Doppler-type Wave Direction Meter (USW-1000)
(Source: Manufacturer's Catalog)

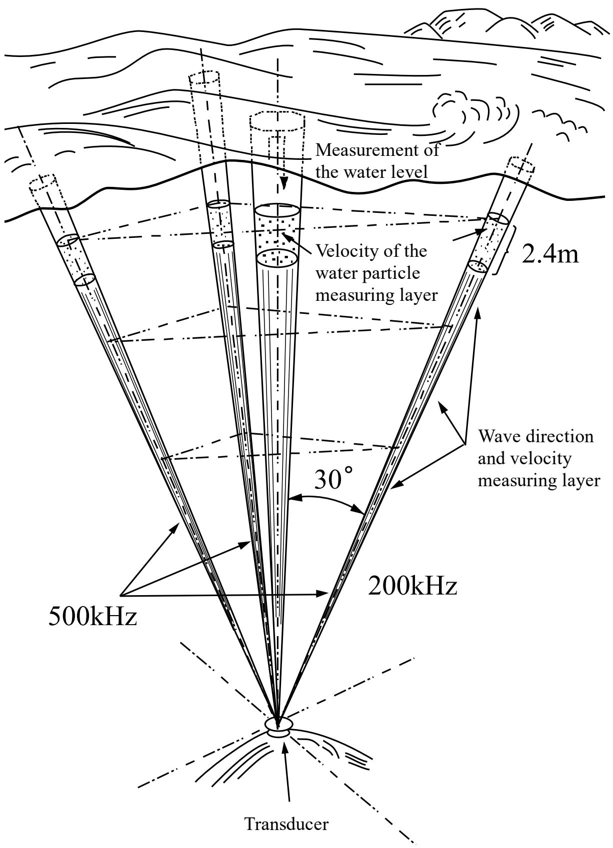


Fig. 2.4.7 Measuring Principle of a Direction Wave Meter

As shown in **Fig. 2.4.7**, a total of four ultrasonic transducers, that is, a 200-kHz transducer to measure the change in the overhead water level and three 500-kHz transducers to measure the velocity of water particles in three directions inclined 30° around it from the extension axis are arranged.

The change in water level is measured in the same manner as traditional USW-type gauges. However, the velocity of water particles is measured by utilizing the Doppler effect, which changes the frequency of returning ultrasonic waves reflected by fine particles floating in the sea, such as plankton, in the measuring layer below the sea level.

Based on the change in water level and the velocity of water particles measured in this way, not only the wave height, period, and wave direction but also the directional spectrum representing the waves having what period and energy are attacking from which direction can be calculated.

Furthermore, while the attenuation of the movement of water particles made it difficult to observe the wave direction at the installation depth deeper than 30 m with the ultrasonic velocimeter-type wave gauge, the direction wave meter does not have such a restriction. This is one of the reasons why the direction wave meter has been adopted as the NOWPHAS-standard oceanographic observation equipment.

The direction wave meter has been utilized for many years, as described above. More recently, in response to the need to monitor the bay environment by observing flow conditions in the vertical multilayer and for early detection of tsunamis by understanding the short-period fluctuation of the water pressure on the sea bottom, a new direction wave meter capable of the following has been developed[21)]:

- More detailed observations of flow conditions by increasing the number of flow velocity measuring layers from the current three to more layers
- Observations of short-period fluctuations of the water pressure on the sea bottom caused by an earthquake by a higher resolution of the annexed water pressure sensor and higher sampling (50 times/s)

2.4.4 Maintenance of the Ocean Wave Observation Equipment

(1) Necessity of and Problems in Maintenance

Since the sensor portion of the ocean wave observation equipment is installed in the sea, troubles such as failure during observation cannot be repaired in a short time and may result in prolonged missing data. Therefore, the stable operation of the ocean wave observation equipment requires scheduled maintenance. The following points become problems when considering installing sensor portions in the sea or on the sea bottom and continuously observing as a main target.

① Pollution due to attachment of organisms or sedimentation of earth and sand lead to the degradation of data quality.

② Troubles such as failure cannot be addressed at once.

③ Accompanying operations on the sea or undersea should pay adequate attention to safety.

④ No calibration as measuring equipment can be done in many cases.

⑤ No official authorization system is available like for meteorological observation equipment.

⑥ The Navigation Aids Act restricts towers and others constructed on the sea.

⑦ Transmission of the observation data onshore via radio waves may be subject to restrictions by the Radio Act, as shown in **Table 2.4.2**.

(2) Maintenance Method

When continuing the observations through the year, regular maintenance as preventive maintenance means the continuous measurement and acquisition of observation data without missing, instead of individually addressing troubles such as failure when they occur.

In principle, the frequency of regular maintenance is once a year. In addition to general function checks, the following points must be adhered to:

① legal regulations (the Navigation Aids Act, the Radio Act) must be followed for equipment subject to them

② the attached organisms and sedimentation of earth and sand, if any, must be removed (cleaned) from the sensor portion installed on the sea bottom

③ any zinc or aluminum for corrosion control on the sensor portion installed on the sea bottom must be replaced at the time of maintenance; the frequency of replacement shall be determined according to the actual loss in weight

④ the check of submarine cables in rocky areas, where cables cannot be buried because of high probability of damage, and rising areas from the sea to the land must be focused on

⑤ onshore equipment, if expected life span can be estimated, must be replaced during the maintenance time so that expected life of the equipment will be kept.

2.4.5 Organization and Summary of the Ocean Wave Observation Data

(1) Overview

The data observed by the ocean wave observation equipment shall be analyzed according to the observation purpose except for cases where the actual status values are immediately utilized.

① **Elimination of outliers**

Noises mixed into the observed data due to various reasons need to be eliminated before analysis. Before the digital recording has been generalized, outliers in the analog records of the observation data were eliminated by visual checks. Nowadays, noises can be efficiently eliminated by software.

② **Analytical processing**

The typical analytical data processing of definitive values after noise elimination is as follows:

(a) Calculation of the ocean wave specifications

(b) Directional spectrum analysis

(2) Calculation of the Ocean Wave Specifications

Changes in the sea level are generally observed in time series as the ocean wave observation. In Japan, the wave height and period are measured with the zero-up-crossing method using the data of changes in the sea level. The data at port-related observation facilities are also processed with this method.

As shown in **Fig. 2.4.8**, a wave period is defined as the time between the start and the end of the wave, where the start time is when the water level crosses the mean water level upward from below and the end time is when the water level subsequently crosses the mean water level upward from below. Furthermore, the wave height is defined as the difference between the lowest and the highest water levels. Individual waves are measured and statistically processed one by one in a specified period. A 20-minute period is set as the specified period for the ocean wave observations in ports.

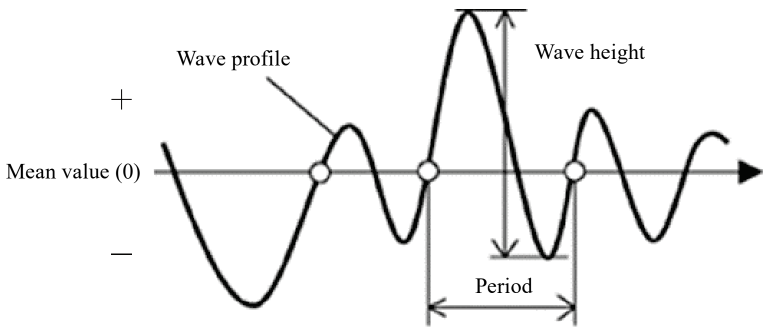


Fig. 2.4.8 Measurement of the Wave Height and Period with the Zero-up-crossing Method

Normally, 100 or more waves are observed in 20 minutes. The highest wave (H_{max}, T_{max}), 1/10 maximum wave ($H_{1/10}$, $T_{1/10}$), significant wave ($H_{1/3}$, $T_{1/3}$), mean wave (H_{mean}, T_{mean}), and other ocean wave specifications are calculated based on the wave height and period of these continuous 100 or more waves.

The highest wave is the wave that showed the highest wave height among the wave height and period within one observation period. The 1/10 maximum wave or the significant wave is the mean of the top 1/10 or top 1/3 wave heights and periods within one observation period. The mean wave averages all waves within one observation period.

These calculated ocean wave specifications are statistically processed and utilized for the analysis of the appearance characteristics of ocean waves in the sea area.

(3) Directional Spectrum Analysis

As the ocean waves are the combination of irregular waves coming from various directions, the wave direction also cannot be easily defined with a single parameter. To resolve this, the concept of directional spectrum showing the magnitude of energy of the ocean wave component related to the frequency and direction is useful.

Fig. 2.4.9 shows an example of the directional spectrum measured and acquired with a direction wave meter off Murotsu Port (directional spectrum obtained utilizing the velocity of the water particles in the upper layer caused by the swell before the attack of Typhoon No. 0918 and measured with a direction wave meter[22]).

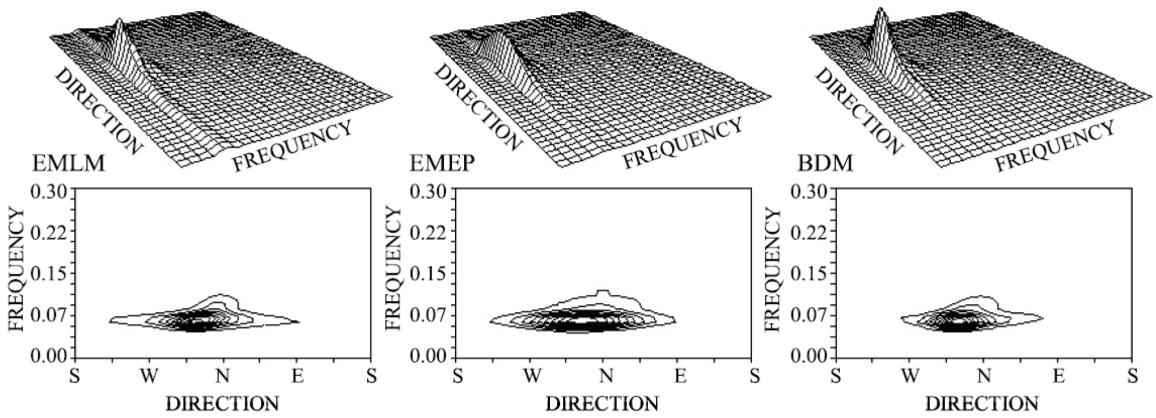


Fig. 2.4.9 Directional Spectrum Measured and Acquired with a Direction Wave Meter

The extended maximum likelihood method, the Bayesian method (BDM), the extended maximum entropy principle method (EMEP), and others have been proposed as an analysis method of the directional spectrum.

Moreover, a method to estimate the highly reliable directional spectrum using not only the velocity of the water particles in the upper layer but also the observation data acquired with a new direction wave meter that measures the velocity of the water particles in multiple layers described in **2.4.3 (4) ②** has been verified.[23]

Various wave direction parameters, such as the mean wave directions, the main wave direction and the peak wave direction, and the energy distribution conditions can be shown based on the calculated directional spectrum of ocean waves. These may be input conditions for the simulation of the ocean wave deformation or harbor calmness.

(4) Utilization of the Nationwide Ocean Wave Information Network for Ports and HArbourS (NOWPHAS)

As described in **Reference (Part II), Chapter 1, 2.4.2 (5) Utilization of the Existing Data,** it is important to consider if existing data can be utilized when observing ocean waves. Even if the data cannot be directly utilized, they may be utilized to verify the reliability of observed data and for the preliminary verification.

NOWPHAS currently has the most fulfilling observation network, data management, and analysis structures. Since 1970, the NOWPHAS ocean wave observation data have been compiled in the Annual Report on Nationwide Ocean Wave Information Network for Ports and Harbours[24] or the Long-term Statistics Report published annually and publicized. These NOWPHAS ocean wave observation data[1] are available in real time on the website.

2.5 Observation and Examination of Tsunamis

2.5.1 Overview

Previously, only the wave run-up trace and tide observation records were utilized as the situ measurement data of tsunamis. Such data are, of course, important to clarify the existing conditions of tsunamis, but are not sufficient. Specifically, as the tide level fluctuation measured in the tide observatory's well in a port records the water level fluctuation via a conduit, it is practically difficult to accurately understand the vibration component in the order of 10 minutes or a shorter period. Therefore, acquiring the wave profile record of tsunamis offshore will be important.

Observation equipment, such as the GPS wave gauges of Nationwide Ocean Wave Information Network for Ports and HArbourS (NOWPHAS) and so on, installed on the Tohoku coast observed wave profiles of tsunamis in the Great Eastern Japan Earthquake. Inverse analysis using these observed wave profiles provided an estimation of fluctuations of the initial sea level at the tsunami wave source area.[25)]

The wave run-up trace, tide observation records, and tsunami wave profile records are valuable data contributing to the improvement of accuracy in the tsunami numerical analysis, investigation of the mechanism of tsunami damage, research and study of reduction of damage, etc. In the performance verification of port facilities, it becomes necessary to verify the required specifications with the fit-for-purpose hydraulic model experiment or the numerical analysis. Among these topics, for the model experiment concerning the tsunami-resistant performance of structures, see **Reference (Part II), 2.10.9 Hydraulic Model Experiment Concerning the Tsunami-Resistant Performance of Structures**.

2.5.2 Observation of Tsunamis

Previous field observations of tsunamis were conducted by checking the tide level record obtained with the tide gauges that have low-pass filter effect by the conduit. Although care should be taken of the possibility that the short-period components may be underestimated, the field measurement is an important record of tsunamis.[26)]

In the wake of the 1993 Hokkaido-Nansei-Oki Earthquake tsunami, the understanding of change in the offshore long-period sea level became possible by continuous observations using the NOWPHAS ocean wave observation equipment installed on the sea bottom,[27)] and the clarification of the substance of tsunamis and long-period waves is progressing.[28)] Furthermore, the development of the GPS buoy that can measure the position coordinates of the offshore-installed buoy every second enabled the observation of the offshore astronomical tide, tide level deviation due to storm surge, and tsunamis.[29)]

Here, an example of the tsunami observation is introduced.

(1) Tsunami Observation Record Obtained by a Tide Gauge

Fig. 2.5.1 shows a wave profile recorded by the Kurihama Tide Observatory when the 1960 Chilean Earthquake tsunami struck. The wave profile record on the tide observation recording paper was read finely with a digitizer and quantified. As the tide observation recording paper was fed 2 cm/hour and read every 0.2 mm, the data sampling interval was 36 seconds.

A smooth tide level fluctuation was seen until 3 o'clock on May 24, but after that time a significant water level fluctuation was observed for a period of about 80 minutes, which is shorter than the astronomical tide. This is the tsunami wave profile. The observation water level is indicated in the figure in conjunction with the astronomical tide level estimated from the harmonic decomposition. This figure shows that the water level after the tsunami attack significantly changes up and down around the astronomical tide. This difference (deviation) is the wave profile of tsunamis. The figure shows that the maximum deviation of this tsunami exceeded 1 m in both vertical directions and the fluctuations of the water level due to the tsunami continued for several days.

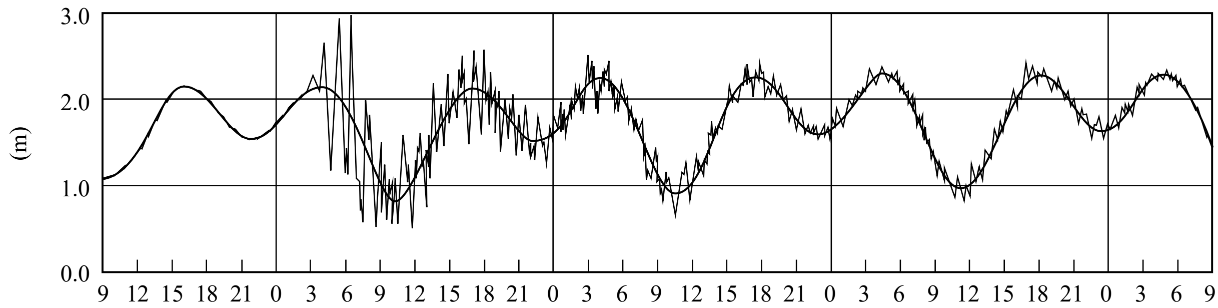


Fig. 2.5.1 Tide Observation Record of the 1960 Chilean Earthquake Tsunami

(2) Tsunami Observation Record Obtained by an Offshore Wave Gauge

Fig. 2.5.2 shows a typical example of the 1993 Hokkaido-Nansei-Oki Earthquake's tsunami wave profile recorded with an offshore wave gauge offshore of Wajima Port after the tsunami attack. This earthquake occurred at 22:17 of July 12. The comparison with the tide observation record in the port clarifies that the first wave that propagated in the Sea of Japan could be just observed during the Wajima Port offshore ocean wave observation time from 23:50 to 24:10.

The horizontal axis shows the 20-min observation time, and the water level fluctuation η (m) measured and acquired with an ultrasonic-type wave gauge at the water depth of 50 m; the vector expression result of the horizontal flow with a current meter at the water depth of 27 m (the direction and the scale is as shown in the legend), the absolute value U (m/s) of the horizontal flow velocity, and the direction of the horizontal flow θ (°) are shown from the top in this order. A significant horizontal flow velocity at 0.3 m/s or more appears twice in the record. The first peak occurred around 23:56 and the flow direction was S, which means that it was a reverse back rush of the tsunami. The second significant peak of the flow velocity occurred around 24:04. The flow direction was N, which means that this time the tsunami was leading waves. Assuming that twice the time difference between the back rush and the leading waves is the period of a tsunami, the period of the tsunami can be assumed to be about 960 seconds.[30)]

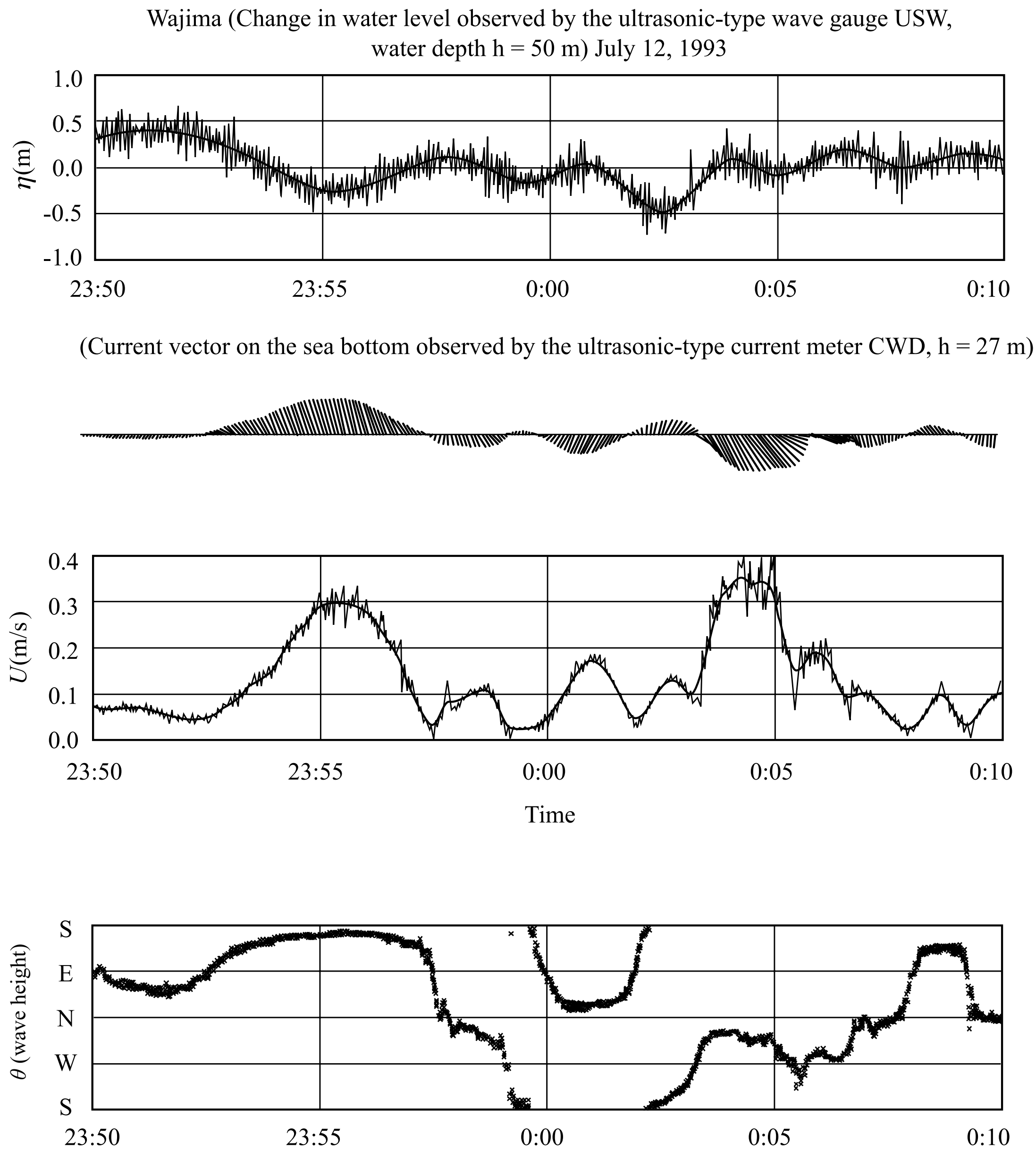


Fig. 2.5.2 Offshore Wave Profile of the 1993 Hokkaido-Nansei-Oki Earthquake Tsunami

(3) Tsunami Observation Record Obtained by a GPS Wave Gauge[31)]

Examples where NOWPHAS' GPS wave gauge network recognized clear tsunamis are the 2010 Chilean Tsunami[31), 32), 33)] and the 2011 Great East Japan Earthquake tsunami (including the foreshock on March 9).

Fig. 2.5.3 shows the tsunami wave profiles at several locations in the Great East Japan Earthquake.[34), 35), 36)] Five wave gauges between the offshore of northern Iwate and the offshore of central Miyagi began with the back rush of several tens of centimeters and the subsequent leading waves were the highest. The highest tsunami arrived at 15:12–15:19 (approximately 30 min after the occurrence of the earthquake) at the six wave gauges between the offshore of northern Iwate and the offshore of Fukushima, which was the earliest in the offshore of central and southern Iwate. The highest tsunami of 2.6–6.7 m was observed offshore of southern Iwate. In the offshore of central Miyagi, back rushes as high as 5 m occurred subsequent to the highest tsunami. The wave profile at five wave gauges between the offshore of central Iwate and Fukushima was generally 0.24–0.95 m higher than that before the tsunami attack, which is considered as the result of the change in the crust. If this effect is offset, the height of the maximum wave at six wave gauges on the Pacific coast of the Tohoku district was 2.1–6.1 m. In the offshore of southern Iwate, the first tsunami wave was the most prominent and decreased gradually from the second to the seventh waves. The periods from the first to the third waves were irregular, but those from the fourth to the seventh waves were about 50 min. The relatively high consecutive tsunamis in the order of seven waves, including the tsunami arriving directly from the source area of the tsunami and the reflected wave from the shore, were commonly seen from the offshore of northern Iwate to central Miyagi.

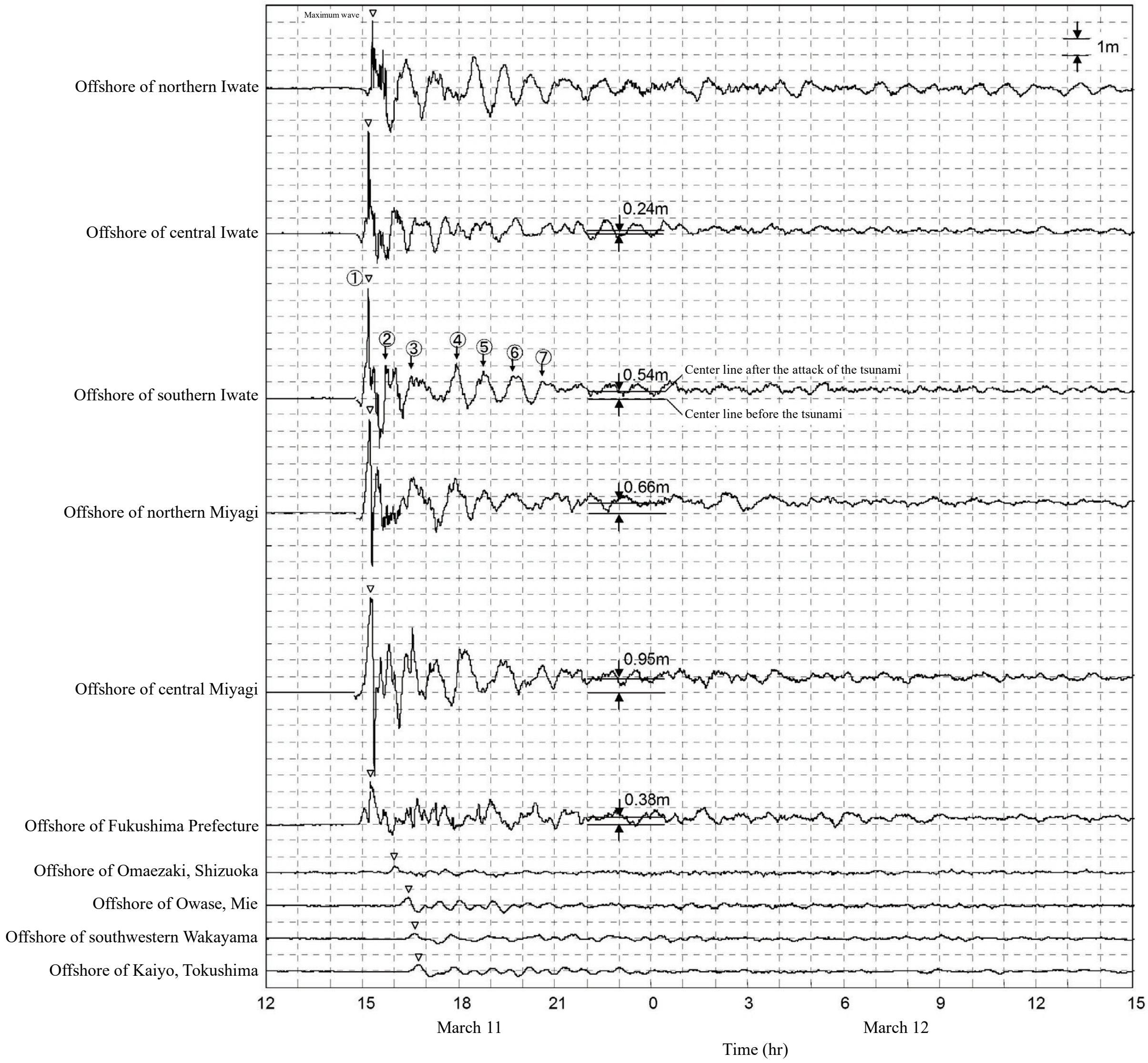


Fig. 2.5.3 Tsunami Wave Profile Observed by the GPS Wave Gauges in Japan[35)]

2.5.3 Kinds and Maintenance of the Tsunami Observation Equipment

For the overview and maintenance of the tide gauges and wave gauges (offshore wave gauges, GPS wave gauges) used for the tsunami observation, see **Reference (Part II), 2.3 Observation and Examination of the Tide Level** and **Reference (Part II), 2.4 Observation and Examination of Ocean Waves, both in this chapter**.

2.5.4 Organization and Summary of Observation Data

(1) Flood Trace Height of Tsunamis

Tsunamis that flood the coast leave traces, such as water marks on buildings or the slopes of hills. The height measured from the ground to the trace left on the wall of the building or others in an inundated area is generally defined as the inundation depth of a tsunami, and the height from the tide level when a tsunami attacked (or Tokyo Peil (T.P.), the standard for the elevation of land) to the trace is generally defined as the inundation height. The inundation depth is 0 m at the trace on the slopes of hills and others. The height of a trace (on the borderline of an inundation area) where the inundation height becomes 0 m is called the wave run-up height.

The trace tsunami heights (wave run-up heights) of past tsunamis have been organized in existing documents, and the record-high tsunami height in the concerned sea area can be checked.

- Comprehensive List of Japan's Damaging Earthquakes 599-2012, University of Tokyo Press, March 2014.
- New Edition of Comprehensive List of Japan's Damaging Earthquakes [Revised and enlarged edition 416-1995], University of Tokyo Press, 1996.
- Comprehensive List of Japan's Damaging Tsunamis [2nd Edition], University of Tokyo Press, 1998.
- Comprehensive List of Japan's Damaging Tsunamis, University of Tokyo Press, November 1985.

The inundation height of the tsunami and the wave run-up height in the Great East Japan Earthquake are summarized in the 2011 Off the Pacific Coast of Tohoku Earthquake Tsunami Information (http://www.coastal.jp/ttjt/) as a result of field surveys conducted by the Coastal Engineering Committee of the Japan Society of Civil Engineers, the Japan Geoscience Union, and other concerned parties.[37] The highest wave run-up height and inundation height in Japan are 40.0 m at Ryori, Ohunato City, Iwate Prefecture, and 33.0 m at Minami Sanriku-cho, Motoyoshi-Gun, Miyagi Prefecture, respectively (extracted from the data with reliability level A or B). **Fig. 2.5.4** shows the inundation height and the wave run-up height organized using the data with reliability level A or B included in the above database. The wave run-up height in Japan is the highest in Iwate Prefecture and it has the tendency to decrease on going south to Miyagi Prefecture, Fukushima Prefecture, and so on.

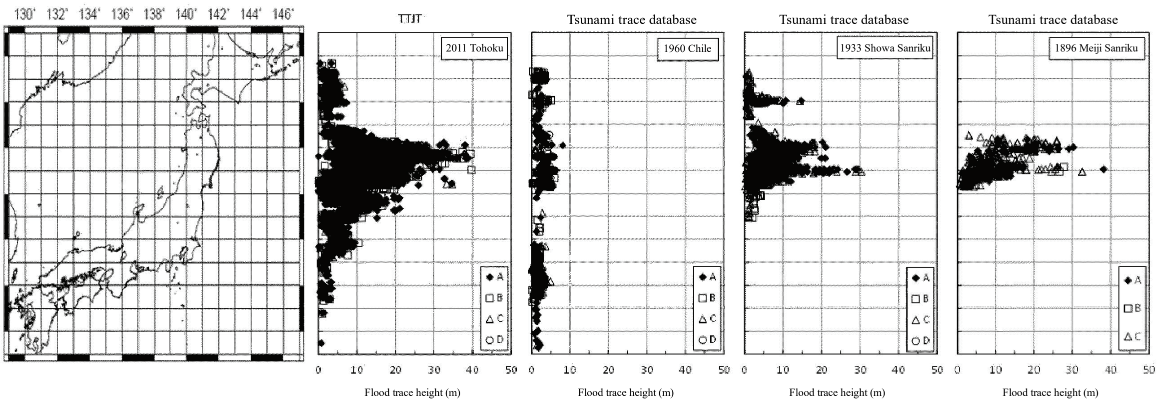


Fig. 2.5.4 Comparison of the Flood Trace Height with Past Tsunamis[38]

(2) Wave Profile Data of Tsunamis

The observed water levels, astronomical tide levels, and tide level deviation data observed by the NOWPHAS wave gauges and tide gauges concerning the tsunamis in the Off the Pacific Coast of Tohoku Earthquake that occurred on March 11, 2011 have been made public and available on the following website:

Tsunami observation data in the Off the Pacific Coast of Tohoku Earthquake

Website: https://nowphas.mlit.go.jp/prg/pastdata/

2.5.5 Simulation of Tsunamis

The phenomenon where the crust slips is called a fault, and the slipped fault is called a fault plane. The inclination of the fault plane, the magnitude of slip, and others can be estimated from the result of seismic observations. When the specifications of a fault plane are determined, how the sea bottom may deform can be calculated assuming the ground as an elastic body. The numerical calculation of the propagation of tsunamis and wave run-ups is possible with the fluid motion equation and the equation of continuity with the displacement of sea level accompanying the change in the sea bottom as an initial condition.

(1) Manuals and Guides

The method for propagation and inundation calculations of tsunamis is organized as follows, as a standard method. In any manual or guide, a planar 2D numerical calculation model based on the shallow water equation is basically used for propagation and inundation calculations of tsunamis.

① Supplement to the Guide for Enhancement of Tsunami Protection Countermeasures in Regional Disaster Prevention Planning, Manual for the Tsunami Disaster Estimation: National Land Agency; Fire and Disaster Management Agency; Japan Meteorological Agency, March 1997.

② Tsunami and Storm Surge Hazard Map Manual: Cabinet Office; Rural Development Bureau of the Ministry of Agriculture, Forestry and Fisheries; Fisheries Agency; River Bureau and Ports and Harbours Bureau of the Land, Infrastructure, Transport and Tourism, March 2004.

③ Guide for Setting Tsunami Inundation Estimation Ver. 2.00: Seacoast Office, Water and Disaster Management Bureau, Ministry of Land, Infrastructure, Transport and Tourism; Coast Division, River Department, National Institute for Land and Infrastructure Management, October 2012.

④ Guide for River Run-Up of Tsunami: Japan Institute of Country-ology and Engineering, May 2007.

(2) Problems in Numerical Calculations

The biggest characteristic of the shallow water equation is that it assumes static pressure (the pressure in water equals the weight of water above it) as the pressure in water. For a tsunami, the wavelength L, which is quite long compared to its water depth d ($d/L < 1/20$), that is, a long wave, this assumption generally gives a good approximation. However, there are limitations in applying the calculation model as described below:

① The equation assuming the static pressure cannot be applied for tsunamis that divide on a shallow beach, etc. On the other hand, the **Guide for River Run-up of Tsunami (2012)** indicates that a non-linear dispersive wave equation (Boussinesq equation), a planar 2D analysis model, and the shallow water equation may be used for the numerical calculation of tsunamis that divide on a shallow beach, etc.

② A planar 2D analysis may not be adequate even for the tsunami, which is a long wave. Fujima et al. (2002)[39] conducted a hydraulic model experiment targeting a breakwater that has a mound in openings, such as the breakwater at the mouth of Kamaishi Bay, and indicated the necessity of a non-static pressure 3D model for analysis of the flow velocity of tsunamis passing the openings.

2.6 Bathymetric Survey

2.6.1 General

(1) Purpose of the Bathymetric Survey

Bathymetric survey is conducted according to planning of the facilities, supervision of construction work, inspection of completion, and port and navigation channel control for navigation safety by drawing nautical charts and each business purpose. Some examples of business purposes and typical bathymetric survey contents, including businesses other than port development projects, are shown below.

① **Control survey for port maintenance or dredging works**

Confirms if the planned water depth value of the navigation channels or the front surface of quaywalls is maintained and the soil volume is calculated as necessary from a result of comparison with the previous survey result and the planned water depth.

② **Survey to correct a nautical chart**

Conducted to set or confirm the change in the planned water depth on a nautical chart, construction of quaywalls, correction of nautical charts by removal, and establishment of port facilities, such as revetment and breakwater. Surveys must be conducted in accordance with the water area classification, measurement methods, and examination specified in the rules, such as public notices. Create necessary results and documents, which are examined by the Japan Coast Guard from the aspect of navigation safety.

③ **Survey as a disaster response**

Primarily conducted for examination of obstacles in the harbor, navigation channel, or others and navigable water depth (works for opening a navigation channel) and others. A bathymetric survey may be conducted based on a tentative water depth datum level or a reference mark due to the ground deformation or others. It is important to leave as many records as possible on positioning, used equipment, tide level, underwater sound velocity, condition of the sea area, and others.

(2) Methods of a Bathymetric Survey

A bathymetric survey is conducted by using such surveying tools and sounding apparatus as shown in **Table 2.6.1** by the type of survey.

Table 2.6.1 Systematic Organization of the Examination Methods of Bathymetric Surveys

<table>
<tr><th>Type of surveys</th><th colspan="2">Surveying tools, sounding apparatus, etc.</th><th>Remarks</th></tr>
<tr><td>Survey at a reference mark</td><td colspan="2">GNSS survey</td><td></td></tr>
<tr><td>Tide observation</td><td colspan="2">Tide gauge</td><td>Tide observatory</td></tr>
<tr><td>Offshore positioning</td><td colspan="2">Sextant, altazimuth instrument, range finder, satellite positioning machine, etc.</td><td></td></tr>
<tr><td rowspan="3">Bathymetric survey
Hydrographical survey</td><td colspan="2">Single beam echo sounder</td><td>Includes multielement echo sounders</td></tr>
<tr><td rowspan="2">Swath echo sounder</td><td>Multi-beam (for shallow sea) echo sounder</td><td></td></tr>
<tr><td>Interferometry echo sounder</td><td></td></tr>
</table>

Reference: **Common Work Specifications for Port Design, Survey, Examination, and Others** (March 2017, Ports and Harbours Bureau of the Ministry of Land, Infrastructure, Transport and Tourism)

2.6.2 Points to Note in Conducting a Bathymetric Survey

Four points should be noted when conducting a bathymetric survey: the survey plan, selection of an echo sounder, applicable laws and regulations, and understanding of the survey site.

Moreover, when conducting soundings using an echo sounder as an ICT dredging work, use the **Bathymetric Survey Manual Using the Multi-beam (for dredging works) (Plan)** (March 2017, Ports and Harbours Bureau of the Ministry of Land, Infrastructure, Transport and Tourism).

(1) Survey Plan

Bathymetric surveys need to be properly conducted in accordance with the laws and regulations, rules, specifications, manuals, and others as proper surveying ranges, density, and accuracy need to be secured according to each business purpose. The laws and regulations regarding the bathymetric survey are publicized by the Japan Coast Guard as work methods, performance standards, and others regarding hydrographical survey, the rules (**Rules for Hydrographical Survey**; hereinafter referred to as “Rules”) of which are published as reference document by the Japan Marine Surveys Association.

It should be taken into account that the data processing method may differ depending on the purpose of the survey. For example, the water depth is read from the topographical transition point so that the seabed topography can be properly expressed in the bathymetric survey. In the bathymetric survey for facility planning or construction work supervision, the water depth at the mesh like lattice point is read so that the mean soil volume for construction forms or plans for the top of slope, foot of slope, and others can be calculated. Moreover, the shallowest water depth is preferentially read in the hydrographical survey to draw or correct nautical charts.

Survey planning may be done based on special specifications or according to the purpose of hydrographical survey. In either case, it is necessary to fully understand the items and content of the bathymetric survey.

The survey method, range of work, equipment to be used, surveying ship, work period, and others must be clarified in the survey plan, considering the purpose and accuracy of the survey and results to be expected and when.

(2) Selection of an Echo Sounder

Echo sounders are classified into a single beam or swath sounder by the number of transmitting/receiving wave beams, analog or digital sounders by the recording form and for shallow sea or for mid- and deep and for deep sea sounders and others by the measurable depth of the sounder.

It is difficult to use a single echo sounder model for every sounding work. The echo sounder should be selected considering the purpose of the operation, the target depth in the sea area to be surveyed and its accuracy, the seabed conditions, and others.

Recently, a combination of the high-accuracy global navigation satellite system (GNSS) and the swath sounder makes effective sounding possible to improve the measuring density and accuracy of the topography of the seabed. In the digital data processing, the sounding noise or error data can be judged from the monitor window where the records are displayed continuously or the echo sounding recording paper. Refer to **2.7 Swath Sounding** in **this chapter** for details regarding swath sounder and swath sounding.

Aerial laser survey and others have been utilized in order to safely perform works near the waterside or surf zone and conduct survey efficiently. The measuring principle of the aerial laser survey is to measure the water depth from the reflex times of near-infrared pulse reflected by the sea surface and the green pulse reflected by the seabed through the water by emitting a near-infrared pulse laser (wavelength = 1064 nm) and a green pulse laser (wavelength = 532 nm) from an aircraft.

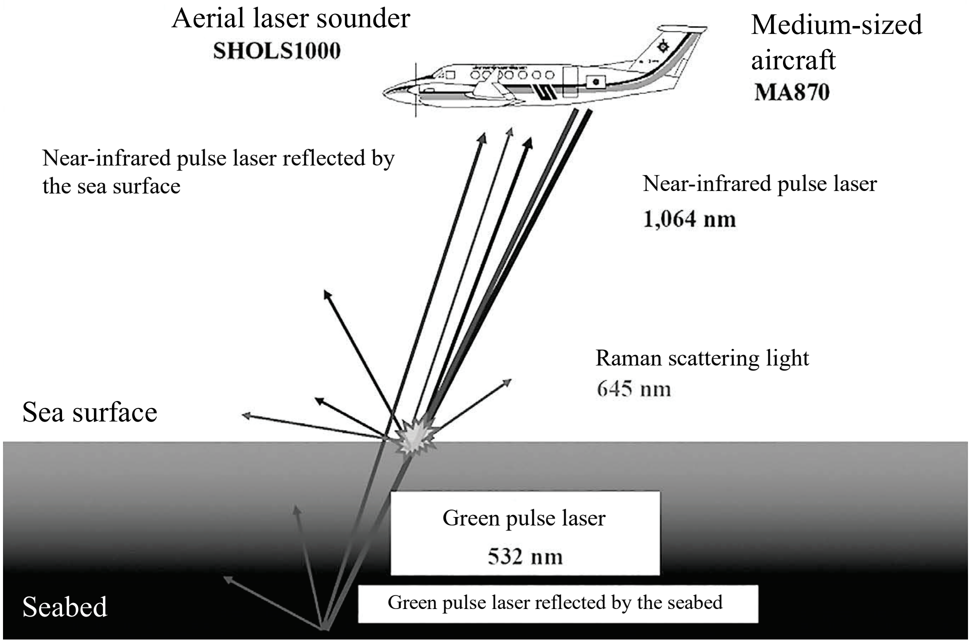


Fig. 2.6.1 Concept of the Aerial Laser Survey

(3) Handling of the Floating Mud Layer

In places where dredging is frequently done to secure navigation channels for large ships, the water depth surveyed using an echo sounder may indicate the upper surface of the floating mud layer deposited on the sea bottom. In practice, the reflection surface of the echo sounding using high frequency (e.g., 200 kHz) is often regarded as the sea bottom when judging the sea bottom in bathymetric survey.

Studies on soil properties (water content, ignition loss, etc.) of the floating mud layer or difference in the reflection surface location due to the difference in frequency[44)] may be referred to. According to these studies, the difference in the reflection surface between 10 and 200 kHz is large (**Fig. 2.6.2**), and the reflection surface becomes deeper as the frequency decreases at an area where the floating mud layer is confirmed to be thick.

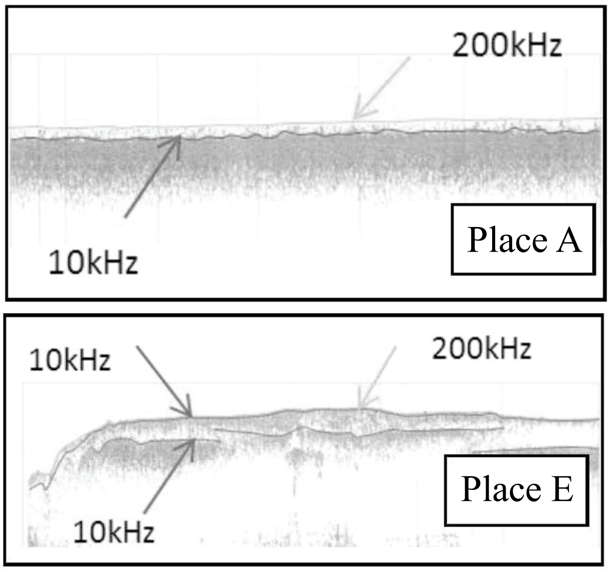


Fig. 2.6.2 Reflection Surface between 10 and 200 kHz

(4) Applicable Laws and Regulations

A bathymetric survey needs to be done in accordance with the survey purpose and applicable laws and regulations. Examples of these are the **Act on Services Related to Waterways** and the **Survey Act**, which are laws and regulations related to law amendment, survey standards, and method for bathymetric survey in accordance with the transition to the World Geodetic System (WGS). They are outlined as follows:

① **Introduction of the World Geodetic System**

The nautical charts have been so far drawn and surveyed based on the **Japan Geodetic System (JGS)** meeting the Japan-specific JGS in compliance with the **Act on Services Related to Waterways** as the standards for hydrographical survey. However, the JGS was replaced by the **World Geodetic System 84 (WGS84)**, a new standard, by the **Act Partially Amending the Survey Act and the Act on Services Related to Waterways** enforced on April 1, 2002.

The change from the JGS to the WGS was prompted by the rapid spread of Global Positioning System (GPS), which is a nautical instrument based on the **WGS84 (WGS84 ellipsoid, WGS84 coordinate system)**. It was triggered by the difficulty in achieving safe navigation of ships that became apparent as a result of the big difference in the ship position measured using GPS and the position in the JGS. Thus, the amendment of the **SOLAS Convention** in July 2002 required ships to meet a certain standard to carry the AIS equipment, which enables automatic and mutual communication between ships based on the position measured in the WGS and other reasons.

On the other hand, the Survey Act specifying the standards for land survey needed to introduce the WGS for scientific rationality, such as geodesy, the ground formation for utilization of GPS, and others. Therefore, the Survey Act and the Act on Services Related to Waterways were partially amended to make a transition from the JGS to the WGS.

Although the WGS84 geodetic coordinate system and the GRS80 system define the ellipsoid differently, they are almost the same, and there are no practical problems. Moreover, the geodetic coordinate system has no concept of height (elevation of land), and the standard of height is based on the previous one which remains unchanged. As the vertical distance from the surface of the ellipsoid may be treated as the height by determining an ellipsoid, better understanding of the definitions of height, elevation of land, geoid height, and ellipsoidal height is necessary.

② **Act on Services Related to Waterways**

The **Act on Services Related to Waterways** has the purpose of developing the hydrographical survey result or other scientific documents regarding the ocean. Moreover, it defines the hydrographical survey as the water area survey, its accompanying land survey and the terrestrial magnetism survey to utilize their results in navigation in its Article 2.

Moreover, Article 6 of the Act stipulates that when people other than the members of the Japan Coast Guard want to conduct a hydrographical survey, the whole or a part of the cost of which is borne or subsidized by the national or local government, the people shall be permitted by the Director General of the Japan Coast Guard.

The Act stipulates the conduct of hydrographical surveys, recommendations for the method of hydrographical surveys, public notification of hydrographical surveys, standards for hydrographical surveys, maintenance of hydrographical surveying markers and surveying ships, submission of results, etc.

The main content of the Rules is as follows:

(a) Order for Enforcement of the Act on Services Related to Waterways

The **Order for Enforcement of the Act on Services Related to Waterways** stipulates the survey standards, such as the height and depth of the hydrographical surveys and the semi-major axis and ellipticity of the earth revolving ellipsoid accompanied by the transition to the WGS

(b) Ordinance for Enforcement of the Act on Services Related to Waterways

The **Ordinance for Enforcement of the Act on Services Related to Waterways** stipulates the hydrographical surveying markers, special cases for the hydrographical survey standards, and markers of ships conducting hydrographical surveys or observation of oceanographical phenomena.

(c) Public notices of the Japan Coast Guard

The following public notices stipulate the survey standards, water area classification, and others for measurements and examinations:

- No. 102: public notice for the method of measurement or examination in hydrographical surveys by classifying water areas.
- No. 103: public notice for the heights of mean water level, the highest water level, and the lowest water level.
- No. 156: public notice for the class of permanent markers used for the measurement of horizontal positions.
- Four public notices (Nos. 157, 158, 159 and 160) designate each area stipulated in the Ordinance for Enforcement of the Act on Port Regulations or the Order for Enforcement of the Port and Harbor Act as special class water area in Appendix 1 of the Public Notice No. 102.

(d) Rule of the Hydrographical Survey Works (Partially Amended on March 31, 2014)

This Rule aims to unify the method and standards for hydrographical survey works conducted by the Japan Coast Guard and their examination standards and ensure the accuracy of hydrographical surveys. This Rule shall be abided by unless laws, cabinet orders, ministerial ordinances, and other act up to the previous paragraph stipulate.

(e) Detailed Enforcement Regulations for the Rule of Hydrographical Survey Works (Partially Amended on March 31, 2014)

This is the rule specified based on Article 8 of the Rule in the previous paragraph and stipulates minute standards and survey method from the performance of the equipment used for origin entry, shoreline and topography survey, tide observation, and water depth survey to the result of hydrographical survey. Hydrographical surveys shall be conducted in accordance with these Detailed Regulations.

(5) Understanding of the Current Situation of the Survey Site

The current situation of the survey site shall be understood in advance through desktop examination by collecting existing documents and site reconnaissance.

① Desktop examination

Nautical charts and topographical maps required for desktop examination are available in the market. Topographical maps are also publicized on the web. Moreover, in the present day, aerial photos and various examination and statistical data can be viewed on and obtained from the Internet. Especially the **Marine Cadastre**[45)] presents natural (seabed topography, ocean current, water temperature, etc.) and social (military exercise area, fishery right area, etc.) information that the Japan Coast Guard or related organizations own on the Internet. They can be utilized since the presented types of data are successively expanded.

Moreover, the **Marine Information Clearing House**[46)] compiles a database of marine information and data that the national organizations own so that the information and data can be easily searched, obtained, and utilized.

The existing documents required for the examination to understand the current situation listed in **Table 2.6.2** with their sources may be collected and can be used to examine the survey site on the desktop.

Table 2.6.2 Primary Documents Required to Examine the Current Situations and Their Sources

Source	Document to collect
Hydrographic and Oceanographic Dept., Japan Coast Guard	[Public notice] List of the highest and lowest water levels, public notices regarding the hydrographical surveys, etc. [Topography] Basic chart (topography chart of the seabed), nautical chart, digital water depth data of the longshore sea [Geology] Basic chart (tectonic map of the seabed), nautical chart of the longshore sea [Marine cable, pipeline] Nautical chart [Navigation safety] Notices to mariners, regional navigational warning (weekly notice), tidal current chart, ocean current chart [Real-time information] Tide, ocean current flash, seawater temperature [Bibliography] Coast pilot, tidal table, guide to use hydrographical charts [Marine Cadastre] Marine information, social infrastructure, information on ship navigation quantity, etc.
Geospatial Information Authority of Japan	[Topography] Topographical map (1/25,000, 1/50,000), topographical map of coastal zones, aerial photograph [Geology] Land condition map of coastal zones [Reference mark results] Results of electronic reference marks, triangulation points, bench marks
Japan Meteorological Agency	[Meteorology, oceanographical phenomena] Tide forecast, ocean observation data, meteorological statistical data
Regional Development Bureaus of the Ministry of Land, Infrastructure, Transport and Tourism	[Geology] Wave observation data, tide, port planning drawings
National Institute of Advanced Industrial Science and Technology	[Geology] Geologic maps, marine geologic maps
Ministry of the Environment	[Natural park] National parks, quasi-national parks
Prefectures	[Fishery] Demarcated fishery charts, list of stationary fishing nets, fishing season [Port] Port area, port planning drawings [Natural park] Natural park area chart
City, town, and villages	[Industry] City, town, and village handbook, etc.

② **Field survey**

Fully examine the collected existing documents, make a survey plan, visit the site, check the site condition, and confirm and correct the content of the survey plan. Basic items for examination of the current conditions at the survey site are as follows:

(a) Topography, geology, etc.

As the information on the sea area to be surveyed, surrounding onshore topography or features and conditions of the seabed become necessary in planning the water depth survey, reference mark survey, shoreline (shore topography) survey, and other surveys, collect existing documents such as nautical charts, topographical maps, city maps, or port planning drawings to examine the current condition.

The nautical charts or port planning drawings become important reference documents in surveys as they contain not only principal navigation channels and facilities, such as piled piers, but also reclaimed lands, dug areas, and others. These charts and others should be prepared early since they are used as the basemap of the whole bathymetric survey plan.

(b) Meteorology

Offshore operation depends on the natural environmental conditions, and the change in meteorological and oceanographical phenomena conditions largely impacts the progress of operations. Therefore, the collection of meteorological documents, such as rain and wind, is essential in planning a survey plan, and these documents are also necessary in determining the appropriate time to conduct examinations.

As the bathymetric survey measures the depth using a surveying ship, it should be conducted on the day when the sea is calm. Thus, it is necessary to understand the wind direction and wind force based on the meteorological statistics at the survey site and estimate number of days when the survey cannot be conducted. It is a good idea to collect statistical data from port public offices and others that observe the weather besides the statistics of the Japan Meteorological Agency.

(c) Tidal current

Tidal current inside a port is not necessarily considered as it does not have a significant effect. On the other hand, narrow navigable channels or sea area where the tidal range is large needs to be examined in advance because it affects the efficiency of the survey operation or the navigation time of the surveying ship to and from the survey site.

The tidal currents in Tokyo Bay, Ise Bay, Shimabara Bay, Kagoshima Bay, and the Seto Inland Sea are compiled in the **tidal current chart** (Japan Coast Guard). The flow velocity at flood-tide and falling tide at major bays and channels are published in the tidal current section of the **tidal table** (Japan Coast Guard). Places listed in the table indicate strong flow areas. For other sea areas, the tidal current condition is indicated by the tidal current arrow mark and the appended flow velocity value on the nautical chart. Moreover, tidal current information on Japan is presented on the Japan Coast Guard website.

(d) Tide

The tide is published in the **Tidal Table** (Japan Coast Guard) listing the estimated values of tide levels and hours and also in the **Tide Table** (Japan Meteorological Agency). The hours and the tide level at low tide and high tide can be obtained from these estimated values.

As for the information on datum level (lowest water surface) as a base of bathymetric survey, the Japan Coast Guard publicly notifies the values of datum levels and the classification chart of the height from the lowest water level to the mean water level (Zo, called Z zero) around Japan as a **list of mean water level, the highest water level, and the lowest water level**, and this shall be used.

If a permanent tide observatory is near the sea area to be surveyed, its observation document can be used as the tidal data used for the bathymetric survey. If not, a temporary tide observatory needs to be installed at the survey site. In a bathymetric survey, when there are amended Zo classifications of tide, care should be taken not to make a mistake in choosing a value to correct when arranging the water depth data.

(e) Existing reference mark

The water depth position expressed on a nautical chart or a plan for construction work shall be indicated as a longitude and latitude value or a coordinate value based on the standard stipulated in the Act on Services Related to Waterways and the Survey Act. Newly established reference marks shall be sought in reference to existing triangulation points, traverse stations, or public institution's existing reference marks surveyed so far by the GNSS positioning or measurement of angles, distances, or others.

As the planar rectangular coordinates are often used for result charts, it is necessary to examine the arrangement of known reference marks and usage related to these marks and collect the results of reference marks in order to clearly specify the used coordinate system. When surveying a position with the GNSS, check the accuracy of the surveying equipment to be used and obtain the result of reference marks near the survey site in order to confirm the set geodetic system.

(f) Troubles in operation

Troubles in operation often occur due to marine construction works near the area being examined or due to conditions of meteorology, oceanographical phenomena, and others. Moreover, it may be necessary to select a survey method considering the position of the incoming satellite and the arrangement time since poor reception due to surrounding topography and features, such as bridges, may occur depending on the celestial condition at the satellite observation point.

Moreover, it is also important to examine in advance a work method suitable for the site condition to avoid troubles in operation, such as hindrance of survey ship sailing on a planned survey course due to ship navigation or anchorage of other ships at the survey area.

2.6.3 Conducting a Bathymetric Survey

(1) Preparation of a Plan

As most works of a bathymetric survey are conducted in a harsh water environment, a plan for the following content shall be prepared as a survey plan considering safety and environment.

① **Content of a plan**

A survey plan generally consists of the following content:

(a) Purpose of the survey: Describe the purpose of the survey concisely and clearly.

(b) Survey site: Illustrate the survey area on a nautical chart and others.

(c) Survey process: Describe the process from the starting date to the end date considering spare days for severe weather and troubles.

(d) Survey items: Describe items and used equipment by the type of works.

(e) Survey method: Describe the datum level, tide level, underwater sound velocity, calibration of each equipment and arrangement of the sounding lines, interval, organization of survey staff, surveying ship, etc.

(f) Deliverables: Describe the deliverables conforming to special specifications and the purpose.

② **Policy of the survey operation**

Describe the survey range, interval of the sounding lines, accuracy of sounding, results of the survey, etc.

(a) Survey range

The survey range is generally clearly identified on the special specifications. It is set including the sea area modified by the construction works and the sea area required for works in surveys related to marine construction works. However, in the hydrographical survey, the modified execution of works or up to 30 m to the exterior of the planned range is set to the sounding sea area reflecting the purpose of the digging survey (**Operating Procedures of the Memorandum regarding Hydrographical Survey Accompanying Marine Construction Works** (April 1972)).

Established survey area should be displayed on a nautical chart. If a quadrangle area to be surveyed is indicated on the chart as a list of four position indicators, such as a, b, c, and d, and their corresponding longitudes and latitudes, that chart may be utilized as an accompanying figure to a notification document to the Japan Coast Guard and others.

Moreover, if a bathymetric survey is conducted as part of environmental research or others, examine and establish the density and accuracy of the water depth required for the research. As a survey plan considering also the comparison between the survey result and existing data may be made, the planned survey range needs to be precisely set.

(b) Interval of the sounding lines

Although the interval and direction of the sounding lines are usually specified in the special specifications or others, it is necessary to consider the purpose of the survey and the target sea area (distance from the shore, water depth, condition of the sea area, etc.) when making a plan. For example, it is important to determine the direction of a survey line in detail considering the requirements, such as “effective in the progress of works,” “easy to understand the topography of the sea area to be surveyed,” or “easy to grasp the construction face line,” and the purpose of the survey or the desired accuracy.

(c) Accuracy of sounding

The degree of sounding accuracy to be set in the bathymetric survey becomes an important element. As the sounding accuracy is determined by the accuracy of horizontal position and that of water depth and dominated by the water depth, seabed slope, or the scale of undulation of the target sea area, carefully select the sounding method and apparatus to be used.

(d) Results of the survey

As the description of the result of the bathymetric survey differs depending on the purpose of the survey, such as planning of the facilities, construction plan, supervision of construction work, preparation and correction of nautical charts, prepare after fully investigating and confirming.

For example, as the examination of the secular change and others in the topography of the seabed compares the water depth values, it is necessary that the position of the sounding line to be set is common to every period (the timing of implementation). If the shoreline survey is included, the result shall be prepared by inspecting the existence condition of an onshore reference mark and considering concurrent use of the GNSS survey.

Basic result and document of the bathymetric survey are shown below:

Result: bathymetric map (survey master drawing), tide observation (datum level determination list), and survey report

Document: track chart (bathymetric chart), bathymetric list (GNSS positioning data), tide observation list, and echo sounding record

Besides the abovementioned result and document of the survey, the following charts and calculation sheets may be requested:

- Administrative survey for port maintenance or dredging work: sectional view and soil volume calculation
- Survey for fishing ground construction plan: construction volume water depth chart, sectional view, and soil volume calculation
- Survey for correction of a nautical chart: result of the digital survey, datum level determination list, etc.
- Survey for disaster response: 3D topographical map (bird's eye view), old/new comparison chart, etc.

(2) Details of the Survey Work

The survey work links a series of surveys from onshore to offshore, and the position relation of the established onshore reference mark and the position of water depth value obtained from the bathymetric survey need to be clarified in the same coordinate system.

The survey items must be carefully planned to avoid missing any work to do and to ensure enough accuracy of the survey for a requested result. The target accuracy of the survey is described in detail in the method of measurement or examination by the water area classification specified in the hydrographical survey (**Appendix 2 of the Japan Coast Guard Public Notice No. 102**), which may be referred to according to the purpose of the survey.

① Reference mark survey and tide observation

(a) Reference mark survey

The reference mark survey determines the position and height of reference marks, features, and others that are required for the survey of the shoreline or water depth. In the point selection of reference marks, consider their most effective arrangement to cover the water depth survey area, and the accuracy of the reference mark survey must also be set considering the purpose of reference mark usage.

(b) Tide observatory

Tide observation is an operation required to determine the amount of tide level alteration and others for measured and obtained water depth, height, etc. Whether a permanent tide observatory is used or a new tide observatory is established shall be determined in the planning stage. When there are no permanent tide observatories near the survey area, tide observation shall, in principle, continuously be done during the survey period at the survey site with a self-recording tide gauge.

When using measured tide levels or tide data obtained from the Internet, it is generally verified with the simultaneous comparison tide observation (observation with an auxiliary gauge) to judge if the obtained tide observation values are correct or not. Moreover, the difference in tidal hour and the ratio of tide levels (see the tide table) between the tide observatory and the survey area or tidal characteristics, such as harbor resonance, need to be taken into account. If there is a hydrographical surveying marker or others related to the specified datum level, a leveling between the tide gauge or the auxiliary gauge becomes necessary.

② **Marine positioning**

Marine positioning is performed with the optical positioning, distance positioning, or satellite positioning and is an important operation in bathymetric survey done concurrently with sounding. The GNSS positioning system utilizing a satellite requires positioning inspection at an existing reference mark in advance.

Marine positioning with the GNSS survey outputs the position of the surveying ship as a digital data, which enables rapid data organization linking with the digital sounding values to achieve labor saving.

③ **Sounding**

An echo sounder shall be used in sounding. If it cannot be used in the harbor, directly measure using lead sounding or others.

Echo sounders are classified into a single beam or swath sounder by the number of transmitting/receiving wave beams, analog or digital sounders by the recording form and for shallow sea or for mid- and deep and for deep sea sounders and others by the measurable depth of the sounder.

It is difficult to use a single echo sounder model for every sounding work. The echo sounder should be selected considering the purpose of the operation, the target depth in the sea area to be surveyed and its accuracy, the conditions of the seabed, and others.

The standards for record handling of an echo sounder, document, result for hydrographical survey for nautical chart correction are stipulated for sounding.

Recently, a combination of the high-accuracy GNSS positioning system and the swath sounder makes effective sounding possible to improve the measuring density and accuracy of the seabed topography. In the digital data processing, the sounding noise or error data can be judged from the monitor screen where the records are displayed continuously or the echo sounding recording paper.

④ **Bottom sediment examination**

As the bottom sediment examination is done to confirm or judge the type of bottom sediment on the seabed and abnormal records obtained from the echo sounding records, the examination spots should be selected in effective density by understanding the outline of the topography of seabed in the whole sounding sea area and others after the sounding operation. If the existence of a floating mud layer is recognized in the navigation channel or basin as a result of sounding, it may be necessary to examine the thickness or distribution of the floating mud layer.

⑤ **Shoreline and topography survey**

The shoreline and topography survey actually measures and maps the shoreline and the surrounding topography and features. When it is necessary to write shorelines on the resulting chart of the onshore survey, the accuracy of the onshore topographical map, aerial photograph, or others as existing documents should be examined and used. When using the resulting chart of onshore survey, if the scale of the existing document and that of the resulting charts is different, reduce the larger scale to match both scales as expanding the smaller scale results in increased errors.

Moreover, when using an existing document, it is basically better to plan conducting an actual measurement so that the actual measurement matches the current condition.

⑥ **Surveying ship**

A ship used for a bathymetric survey needs to be selected to meet operational scale of the survey. Essential points for selection are good maneuverability, economy, and so on. As a small ship is usually hired at the site, it is necessary to understand the ship chartering market condition in advance.

A surveying ship shall be selected based on the following requirements:

(a) As a surveying ship equipped with a “ship inspection certificate” and a “ship pilot license” required for offshore operation notification to the Coast Guard Office, documents of each copy can be attached.

(b) The size is suitable for the meteorological and oceanographical phenomena in the water area to be surveyed and good in stability and steering performance.

(c) The engine of the work ship vibrates less and emits less noise.

(d) Easy to install an echo sounder transducer.

(e) The positioning equipment and the sounding apparatus can be set up in the ship where there is no influence of the splash of seawater. In a large surveying ship, cable wiring to the cabin (observation room) shall be securely fastened so that there is no interference with other observation equipment to prevent accidents, such as breaking of wires.

(f) There are a few obstacles to angle measurements, bar check works, and others on board, and the steersman and surveyor can communicate easily.

(g) A GNSS antenna for positioning can be installed safely and securely. It is better to sail in variable speed from high speed to low speed. The relative merits of steering performance may affect the safety in the survey and the operating efficiency in traveling to the water area to be surveyed or operations in the water area congested with ships, such as navigation channels and shallows.

(h) Preparation, inspection, and transport of surveying instruments

The number of surveying instruments classified into equipment and consumables used for field work and indoor work is quite large. Some consumables cannot be purchased at the survey site. Make a list of the instruments and select the ones required so as not to miss anything during preparation. For instruments that could not be prepared in advance and should be rented, make the reservation with a rental company near the survey site. Confirm the location and closed days of the rental company as well as the stock when carrying in and so on so that there is no interference with the operation.

It is important that instruments to be taken to the site be inspected beforehand on how they operate, maintained as needed, and assembled, including the accessories, spares, and tools. As digitization of surveying and sounding data has increased the use of precision instruments, such as personal computers (PC), in the operations on board, it is necessary to pay attention to handling of equipment when packing them securely to endure loading and transportation on vehicles. A set of instruments for swath echo sounding, in particular, comprise an enormous number of equipment and members, including outfitting materials; it is also necessary to make enough room for service vehicles to be used.

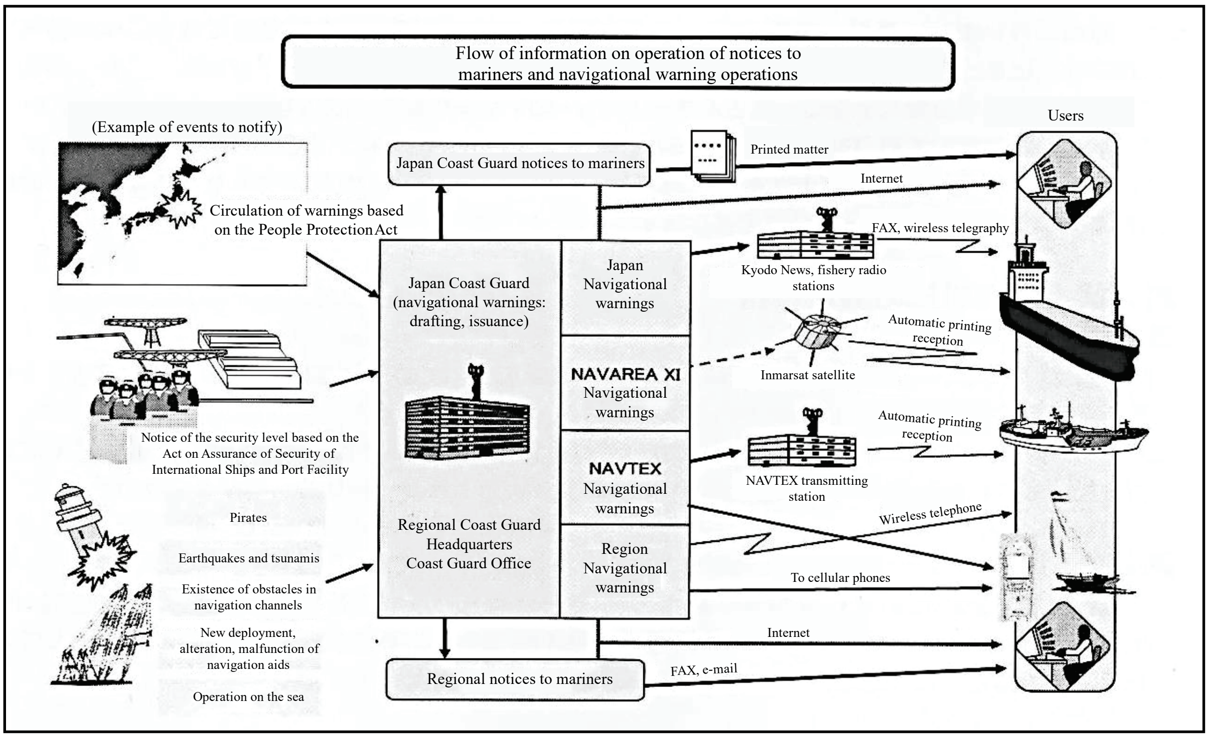
Service vehicles loaded with a set of instruments must be driven safely to the survey site conforming to the traffic rules with a time schedule having enough margin in the arrival time.

(3) Safety Management

As the bathymetric survey is conducted in severe work environment, such as on a surveying ship being constantly moved by the wind, waves, and others on the sea, close attention toward safety management is needed. Especially surf zones are dangerous water areas where the wave breaking energy and the complicated surrounding flow condition may cause grounding or capsizing. Moreover, in the navigation channel where many ships are congested, safety countermeasures such as placing sufficient number of people to watch to avoid the danger of collision and deploying range guards are required.

Information on the sea area near the bathymetric survey area is also important. The plan must pay heed to safety in operation by collecting notices to mariners, navigational warnings, and others shown in **Fig. 2.6.3** from public notices or websites of Coast Guard Stations and others. Environmental countermeasures, such as oil spill and discharge of waste to the ocean, may be requested to be included in the plan. When a time-critical accident or disaster occurs, the damage must be minimized by responding rapidly and accurately according to the predetermined emergency contact system chart, route chart to hospitals, and so on.

A Pocketbook on Safety (Japan Marine Surveys Association)[47] should be carried to the site and referred to as required. For safety countermeasures, it is important to attend courses on nurturing safety supervisors or emergency aids or patrol boat control and operation courses sponsored by the Regional Coast Guard Headquarters, the Japan Association of Marine Safety, or others on a routine basis to deepen knowledge toward safety management.



Source: Bibliography No. 801 published by the Japan Coast Guard

Fig. 2.6.3 Japan Coast Guard's Notices to Mariners and Navigational Warnings

(4) Various Notifications

When conducting a bathymetric survey, submit in advance permits for hydrographical survey, permits for and notification of offshore operation, and permits and notifications stipulated in other related laws and regulations. Also, observe agreement, acceptance, and others mandated by local regulations, communities, and others and address adequately for their implementation.

Moreover, it is necessary to let the content, method, and process of operation be known to organizations controlling the survey site and persons concerned when conducting the work.

① **Related laws and regulations and various procedures**

Procedures based on laws and regulations include those performed by the survey-planning organization and by the survey-taking organization. As some procedures may take time, check with the institution to which the procedure is applied.

Laws and regulations related to the bathymetric survey are as follows:

(a) Act on Services Related to Waterways

(b) Laws and regulations related to safety of offshore operation or trespassing

• Act on Port Regulations
• Maritime Traffic Safety Act
• Act on Preventing Collisions at Sea
• Port and Harbor Act
• Act on Development of Fishing Ports and Grounds

(c) Laws and regulations related to shore, park, and others

• Coast Act
• Natural Parks Act
• Urban Park Act
• Nature Conservation Law
• Act on Prevention of Marine Pollution and Maritime Disaster

(d) Other related laws and regulations

• Ship Safety Act
• Radio Act

(5) Notification to Organizations Concerned

Not only limitations by the related laws and regulations but also restricted entry to the premise of a factory or fishery facilities may interfere with the survey operation. Therefore, it is essential to examine the site in advance to avoid interference in the survey operation.

Notification of operation to parties concerned and so on or written consent and others for the survey operation may become necessary. Examples of parties to be notified include Harbor Section of the local governments, harbor offices, fishery cooperatives, marine construction companies, ferry service companies, and oil transportation companies, depending on the survey water area. Especially, it is important to explain and notify the purpose, method, and others of the survey, including the hired surveying ship, to fishery parties. Moreover, if the survey area is included in the navigation channel where ferry or others operate, it is necessary to check the time table of the ferry or others and do the safe sounding operation, avoiding the period of time overlapping the survey time.

2.7 Swath sounding

2.7.1 General

(1) Information and Communication Technology (ICT) and Swath Sounding

Improvement of technology related to information or communication is utilized in many fields, and the range is still expanding. A rational production system utilizing this information and communication technology called ICT is being introduced also in the civil engineering and port fields. Utilization of ICT enables storing, processing, and transferring of information in the 1) examination, 2) planning, 3) design, 4) estimation, 5) construction, 6) inspection, and 7) maintenance stages and streamlining and improvement of a series of production system in the port construction field.

The starting point of this system is the data created in various examinations. It has become more important than ever before that data such as meteorological phenomena, oceanographic phenomena, water depth, and soil boring log represent the natural conditions and are credible and of high quality. In addition, as the frequency of large ships entering into a port from other countries is increasing, high-quality water depth data complying with international standards is becoming necessary for ships entering into a port to operate safely.

Therefore, the swath (meaning: covers wide breadth at one time) sounding method that can sound a surface effectively and with high precision is attracting attention. **Figure 2.7.1** shows a complete picture of a construction information system based on ICT in the port construction field that is also a basis for utilization of the swath sounding.

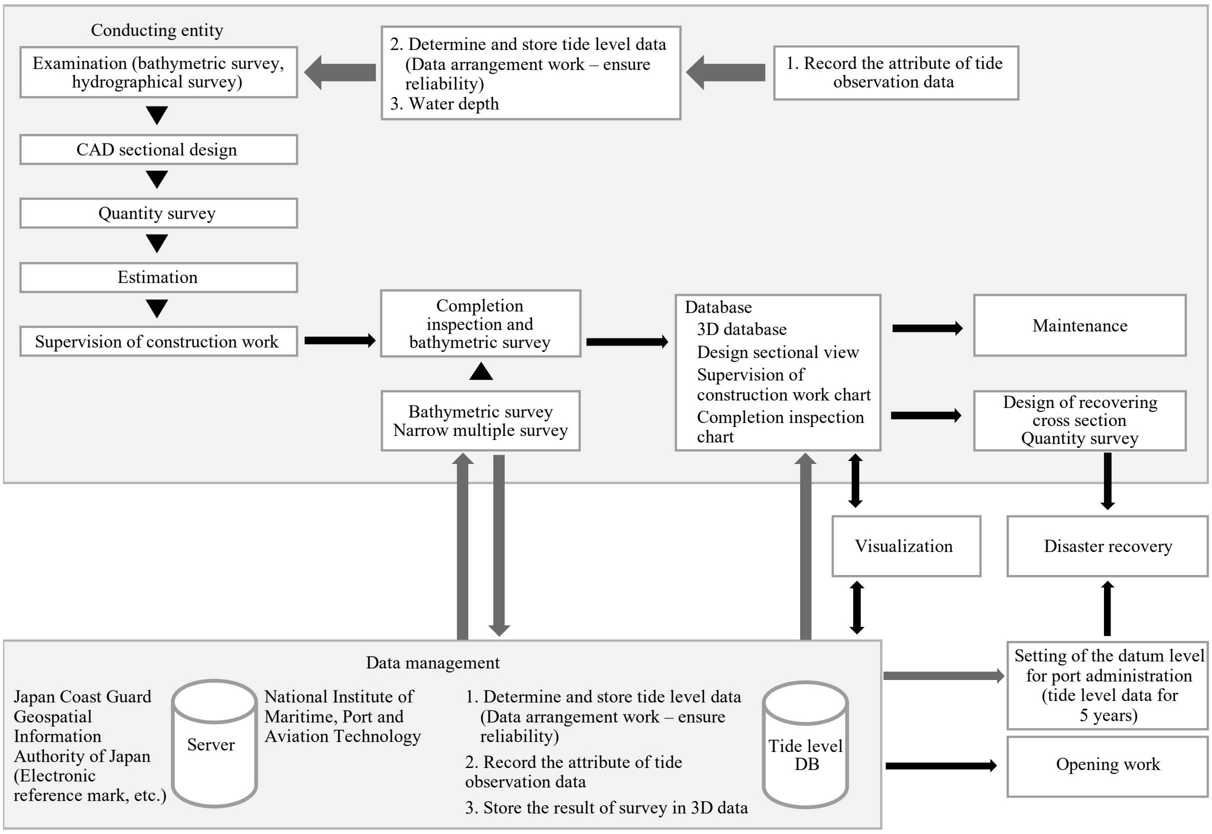


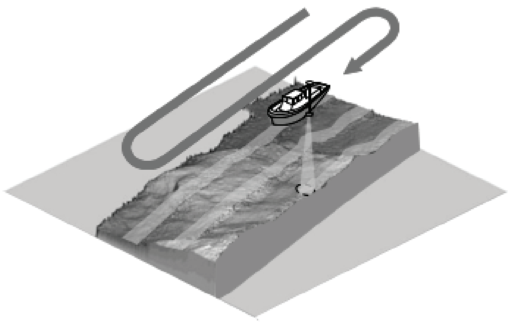
Fig. 2.7.1 Complete Picture of a Construction Information System Based on ICT in the Port Construction Field

The Ministry of Land, Infrastructure, Transport and Tourism has introduced the ICT dredging works since FY 2017 as an "overall utilization of ICT" which is one of the i-Construction top runner policies. The bathymetric survey using multibeam, which is one of the swath sounding methods, was introduced in these works, and standards such as a **Manual for Bathymetric Survey** Using Multibeam **(Dredging Works Section) (Draft)** were prepared.

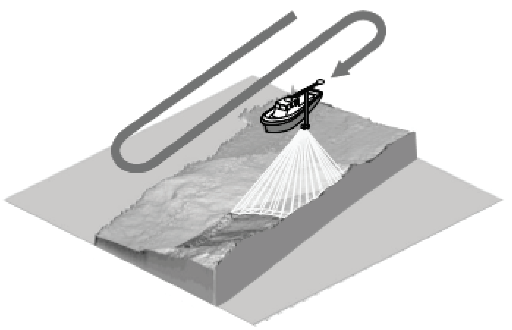
(2) What is Swath Sounding?

Swath sounding measures the water depth values at many points at a time as the surveying ship moves by emitting echo beam with high directivity to the seabed in the right and left directions of the surveying ship.

In contrast to the traditional single-beam sounding which measures the seabed in a line as water depth information just below the transducer, the swath sounding measures the detailed topography of the seabed as a surface.



Single-beam sounding:
measures the seabed in a line



Swath sounding: measures the seabed as a surface,
leaving no part unmeasured

Fig. 2.7.2 Concept of a Single-Beam Sounding Method and a Swath Sounding Method

The development of the swath echo sounder was started in the 1960s by the US Navy, and the sounder became commercially available in the 1980s. The original sounder was used for sounding of deep seas and installed at the bottom of a large ship. In the 1990s, the sounder was downsized and lightened, and a portable system for shallow seas capable of rigging on the side of a ship of 20 t or less was developed.

In Japan, a private company introduced the sounder in 1994 for the first time, and it became rapidly and widely used since 1995, for example, the Japan Coast Guard installed the sounder on its surveying ship. A few dozen systems are now operating in Japan.

The introduction of the swath sounding technology enabled the measurement of the topography of seabed in detail and as a surface. As a result, new knowledge on the topography of seabed became available.

Detailed topography data of seabed is widely utilized for drawing of nautical charts, port construction and dredging works, development business such as examination for laying seabed pipeline and others, and maintenance field such as confirmation of the amount of deposited silt in a dam reservoir or riverbed height, and scouring condition of port facilities, accompanied by the development of digital analysis technology or ICT technology such as GIS.

It was utilized for confirmation of obstacles, water depth, and damage situation of underwater structures and others for the prompt opening of navigation channels at the time of large-scale disasters, such as 1995 Great Hanshin-Awaji Earthquake and 2011 Great East Japan Earthquake, and it also greatly contributed as a basic data for recovery and reconstruction.

Multiple models of recent swath echo sounders have become widely used, sophisticated, and downsized. Models that can be installed in ROV (remotely operated vehicle: underwater exploration equipment remotely controlled on board) or AUV (autonomous underwater vehicle: exploration equipment acting autonomously underwater) have been developed and are widely used also as basic data collection tools for seabed resource exploration.

2.7.2 Principle of Sounding

The swath echo sounding methods are divided into two systems that employ different sounding principles: the narrow multibeam sounding and the interferometry sounding.

(1) Multibeam Sounding Method

This may be called a narrow multibeam sounding because of sounding with fine (narrow) multiple (multi) beams, but here, it is called a “multibeam sounding.”

The principle of a cross-fan-beam sounding method used for a multibeam sounding can grasp reflected waves from the portion (footprint) where the wave transmission fan beam and the wave receiving fan beam are overlapped when the wave transmitter transmits fan-like wave transmitting beam (wave transmitting fan beam) to the sea bottom in the direction perpendicular to the navigation direction of a surveying ship as shown in **Fig. 2.7.3** and the wave receiving fan beam of the wave receiver is formed in the navigation direction of a surveying ship. By measuring the transducing time (time from wave transmission to wave receiving) of this reflected wave, the distance from the transducer to the reflection surface can be obtained.

By altering the direction of the wave receiving fan beam and moving the position of footprint in the wave transmitting fan beam, the wave transmission fan beam can be measured by dividing by footprint with one ultrasonic wave transmission and receiving.

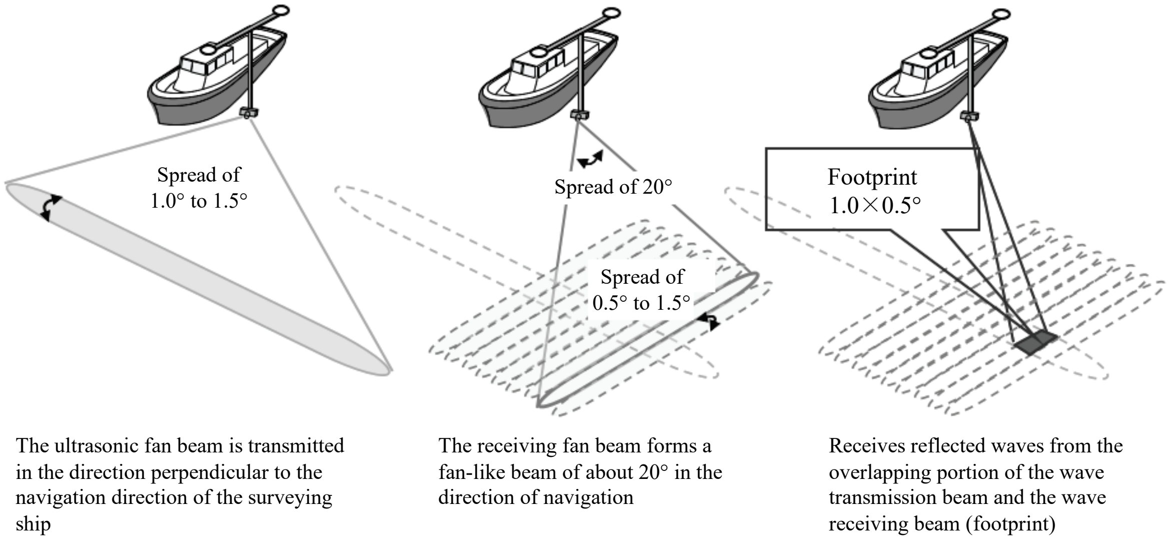


Fig. 2.7.3 Concept of a Cross-Fan Beam

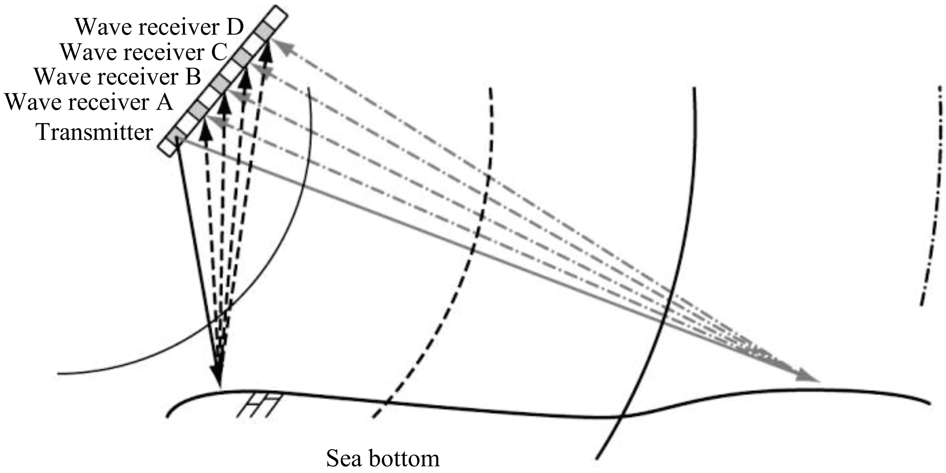
In recent years, some models employ a method to transmit wave transmission beams by dividing into some portions in advance and correcting yaw and pitch individually in order to stabilize the footprint with this cross-fan-beam method.

Also, some other models have been put on the market that employ the technology called “dynamic beam focusing” to improve the precision by calculating the distance between each point on the center axis of the wave receiving beam corresponding to the turnaround time and the wave receiving element for wave receiving line array signal, shifting signals by time so that the same phases overlap and forming fine beams.

(2) Interferometry Sounding Method

Interferometry is a method to determine the angle from which the sound wave comes utilizing the difference in phase of the ultrasonic reflected on the seabed, also called the interference wave method.

The interferometry sounding individually receives echo from the seabed with four or more wave receivers, measures slight difference in the phase, determines the direction angle from which the echo arrives, and calculates the water depth value from those combinations.



(a) Irradiates sound waves from the transmitter to the sea bottom
(b) Receives reflection from the sea bottom with four wave receivers (A to D)
(c) Measures the difference in time of arrival from the difference in phase of the sound waves
(d) Determines the returning angle from the four phase differences and converts the angle to the water depth value by the combination with linear distances

Fig. 2.7.4 Concept of the Interferometry

(3) Major Characteristics of the Measuring Principle

The measuring density by the beam forming in the multibeam sounding method becomes coarser for the data of the external side than the central side of the irradiation range of sound waves (hereinafter, "swath width"). However, as there is only one resolution of turnaround time for each angle, markedly uneven topography, such as quaywalls and reefs, can be measured accurately.

On the other hand, as the interferometry method uses the interference wave, the number of measuring points near the center of the swath is significantly reduced. However, a lot of measuring points can be obtained as the swath width is wider (8 to 12 times of the water depth) than the multibeam sounding method. Therefore, more effective sounding work can be expected than the multibeam sounders, especially in ultra-shallow sea areas. It has also the side scan function which judges the bottom sediment from the reflection intensity from the sea bottom. However, accurate measurement may be difficult in markedly uneven topography, such as quaywalls and reefs, because the reflected signals from the sea bottom and the wall interfere. **Fig. 2.7.5** shows an image of sounding and the shape of transducers for each measuring principle.

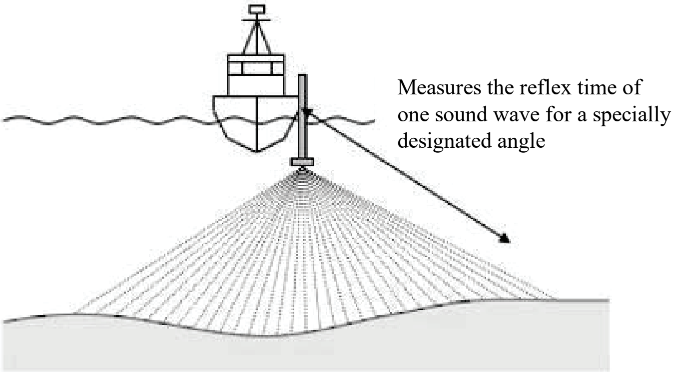


Image of measurement with the multibeam sounding system

Example of a transducer

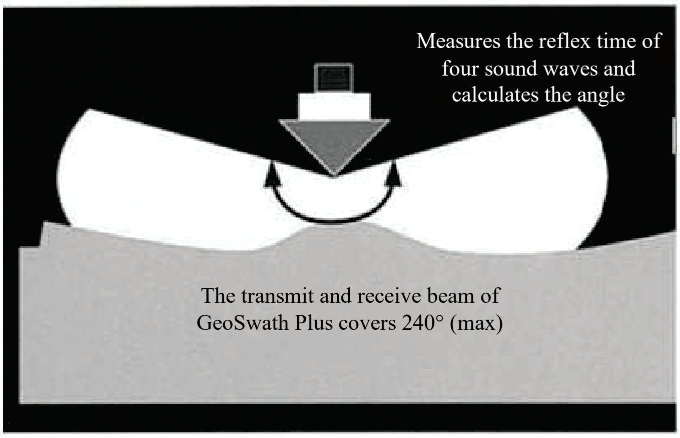
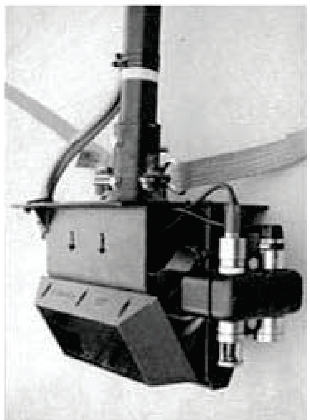


Image of measurement with the interferometry sounding system



Example of a transducer

Fig. 2.7.5 Image of Sounding and Concept of Transducers per Measuring Principle (Agency's Catalog (Partially Corrected))

By understanding the measuring principles and features of the multibeam sounding method and the interferometry sounding method and devising the measuring method or analysis processing, the sea bottom can be measured using any model of whichever method.

Models of the multibeam sounding method are often selected in general in the measurement for port facility maintenance or work process control of dredging works considering the reflection by structures.

On the other hand, selecting a model of the interferometric sounding method which can measure widely at a time makes it possible to carry out work efficiently near the mouth of a river or gently sloping topography.

Both the multibeam sounding method and the interferometry sounding method have merits and demerits. However, as the introduction of the multibeam sounding method has been attempted in the port construction field with the preparation of manuals and others, since March 2017, the multibeam sounding is explained below.

2.7.3 Configuration of the Multibeam Echo Sounder

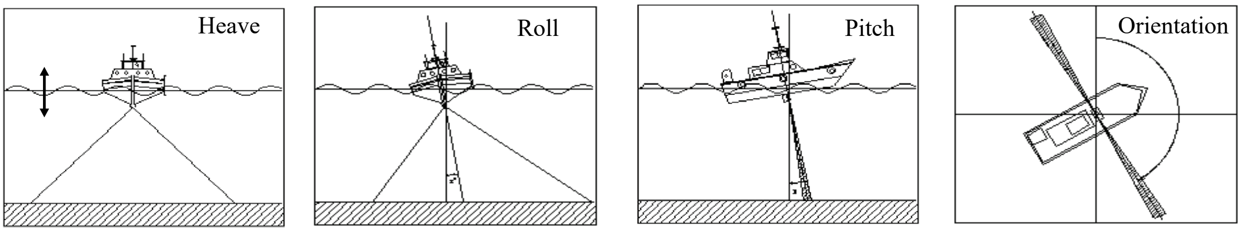
The multibeam echo sounder comprises the sound wave transducer and processor called the main body and peripherals, such as motion sensor, orientation sensor, position sensor, and sound velocity sensor.

The main body and peripherals are rigged on a surveying ship, and the data is compiled in a lump in dedicated software installed in the PC.

The function of each peripheral is shown in **Table 2.7.1**, and the images of correction applied to up-and-down motion (heave) by waves, horizontal oscillation (roll) by waves, vertical oscillation (pitch) by waves, and direction of ship's navigation direction (orientation) are shown in **Fig. 2.7.6**.

Table 2.7.1 Functions of Peripherals

Name of sensor	Function
Motion sensor	Corrects the position where the sound wave reached the sea bottom (measurement position) by measuring the ship's or sonar's horizontal oscillation (roll), vertical oscillation (pitch), up-and-down motion (heave)
Orientation sensor	Corrects the orientation in which the multibeam transducer is transmitting/receiving by measuring the ship's or sonar's orientation (yaw)
Position sensor	Identifies the position where the sonar is irradiating by measuring the ship's or sonar's position
Sound velocity sensor	Corrects the arrival time of the sound wave by measuring the underwater propagation velocity of the sound wave which changes according to water temperature, salinity, and others
Compilation PC	Compiles information from each sensor and sounding sonar with time synchronization



Correction of up-and-down motion by waves — Correction of horizontal oscillation by waves — Correction of vertical oscillation by waves — Correction of ship's navigation direction

Fig. 2.7.6 Correction Images with Peripherals

Some of the peripheral motion sensors or orientation sensors ensure accuracy by employing 3-axis accelerometer/3-axis angular velocimeter. Moreover, in recent years, some models (POS system) that measure the behavior of a ship more stably by adding the GNSS information to these sensors and utilizing the inertial navigation have started to be used.

2.7.4 Preparation of a Plan

As a preparation of a multibeam sounding plan, a survey line plan and a ship chartering plan are necessary.

(1) Survey Line Plan

① **Planning of survey lines**

The survey line must be planned, paying attention to the following matters:

(a) Plan so that the sounding work can be effectively done considering the topography of the seabed and water depth.

(b) Avoid continuous survey lines as much as possible since the sound velocity to correct differs in an area where the seawater temperature or salinity differs.

(c) Avoid a survey line plan which continuously sounds different sea area as much as possible since the datum level to correct differs in an area where the lowest water surface is different.

(d) Plan not to leave unsounded width considering the effective sounding width of the swath echo sounder to be used and the deflection of the surveying track in and near the navigation channels, basins, anchorages, and quaywalls. In this case, 20% of the effective sounding line width is generally overlapped.

(e) Plan a survey line that sufficiently overlaps with the neighboring survey lines (an overlap of 100% or more of one side beam width is recommended) so that the shallowest part can clearly be captured in the sea area where obstacles at the sea bottom, such as reefs, fish shelters, and sunken ships, exist or where such obstacles are anticipated.

(f) Plan a double cross survey line to verify the accuracy of sounding.

② **Effective sounding width**

When planning a survey using a multibeam sounding system, it is important to set the effective sounding width, considering the water depth of the target water area, the resolution of the result (mesh size), and the purpose of measuring (accuracy).

The multibeam sounding has a characteristic that the sounding points become coarser at the edge of the swath than the point just below the transducer. The deeper the target water depth, the wider the swath width. However, the resolution becomes coarser as each footprint becomes larger. Footprint (lateral length) can be calculated using the following equation.

Footprint X = Water depth α m × ((tan θ) - tan (θ -β)) **(2.7.1)**

where

α : arbitrary water depth

θ : 1/2 of the target beam angle

β : smallest beam angle

For example, if a model that gives a footprint = 0.5 × 1.0° at the sea area of water depth = 40 m as shown in **Fig. 2.7.7** is selected, α = 40 m, β = 0.5°, and the lateral spread of the beam just beneath = 0.349 m. Although it is 0.692 m at 90°, it spreads to 0.998 m at 108° and 1.376 m at 120°. At this time, if the resolution of the result is set to 1 m mesh, angles up to 108° can be used, and thus, a survey line considering a swath width that can be obtained within 108° must be planned.

On the other hand, if the resolution of the result is set to 1 m mesh at a sea area where the target water depth is 20 m, a survey line with 120° swath width can be planned as the footprint is no more than 0.688 m at 120° beam.

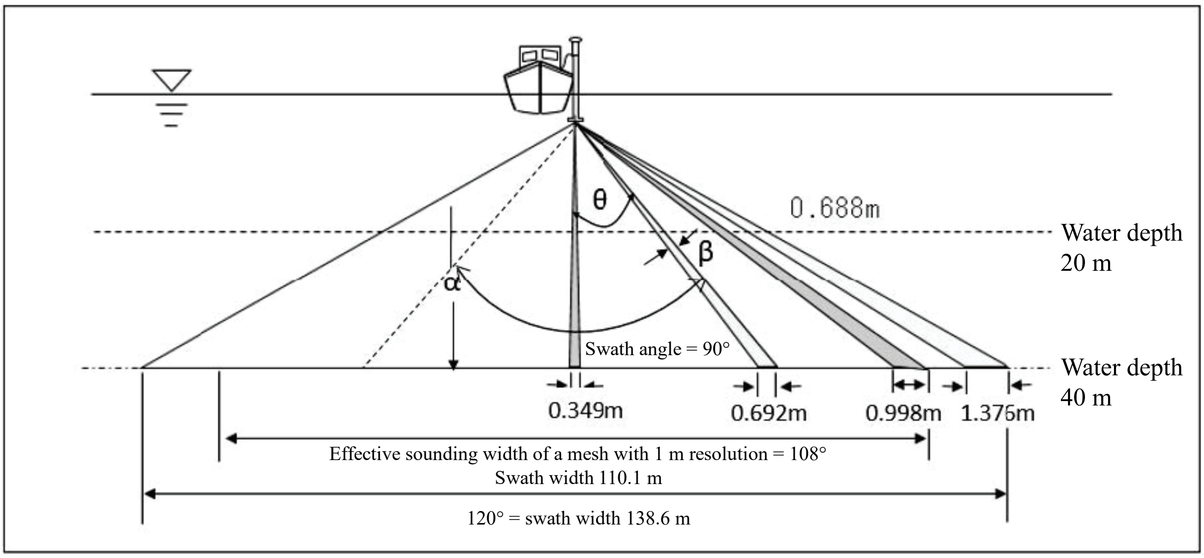


Fig. 2.7.7 Footprint Image of a Multibeam Sounding

Some recent models have a function to divide the interval of each beam evenly at the target water depth (equidistance mode). In this case, the beam interval is equally divided by the number of beams at a water depth. When the number of beams is 256 at the water depth of 40 m, the beam interval becomes 0.541 m, and thus, a swath width of 120° can be planned.

③ **Unmeasured area**

The survey line shall be planned so that no area is left unmeasured, considering the effective sounding width, reduction of the swath width due to meander of a surveying ship (deflection from the planned survey line) as shown in **Fig. 2.7.8**, or the change in water depth, change in the irradiation range due to horizontal oscillation (roll), existence of protruding objects, and others.

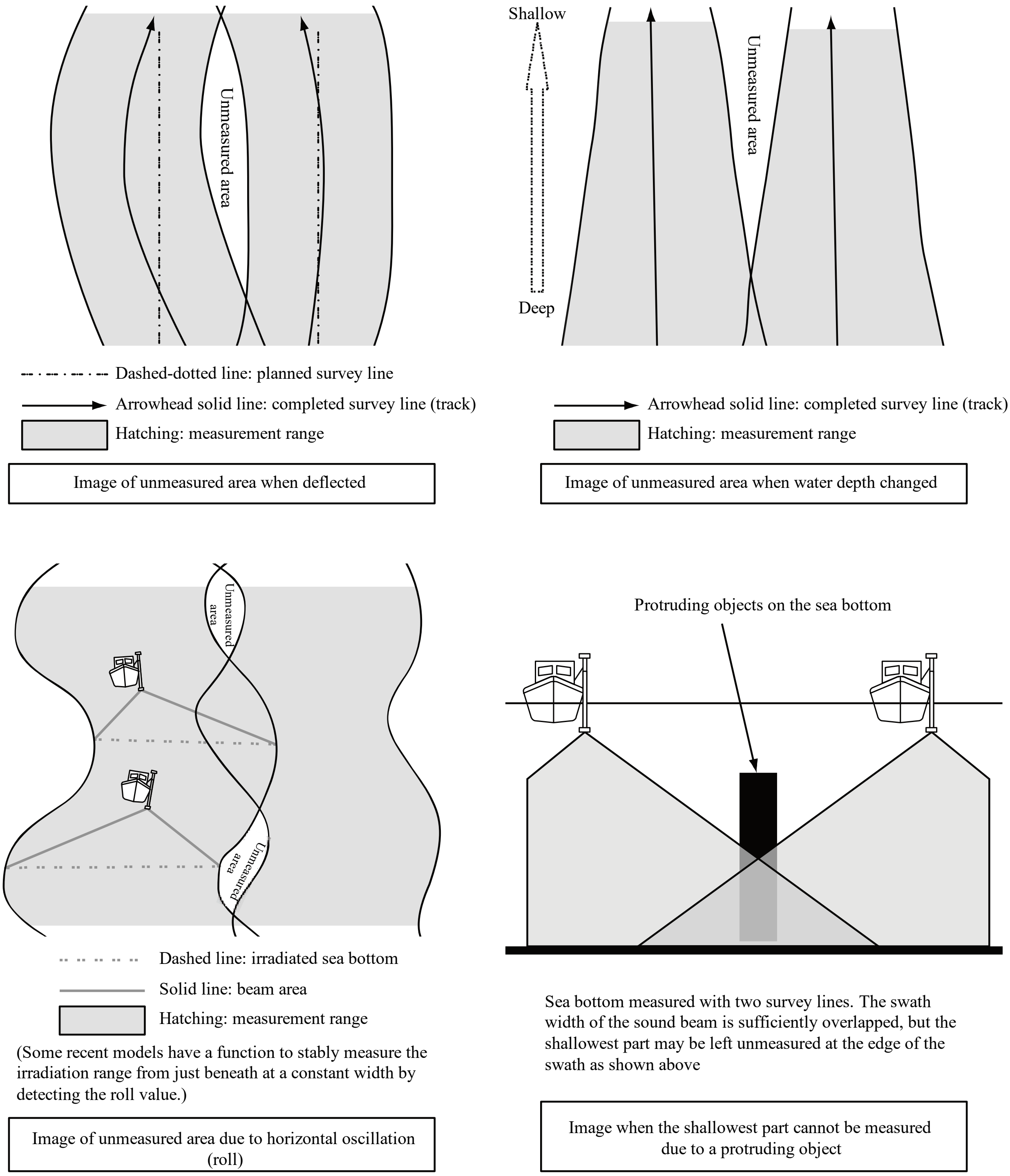


Fig. 2.7.8 Images of Consideration at Survey Line Planning

It is preferable to plan a survey line by fully considering the overlap of effective sounding width so that no unmeasured area exists.

In addition, it is necessary to conduct the measurement work at site confirming that measurement is progressing as planned, and there are no unmeasured areas using the PC screen on board.

(2) Ship Chartering Plan

A ship that meets the following conditions is preferable to be used as a surveying ship in the ship chartering plan.

① A ship with stout sides as a transducer is fixed on the sides

② A ship capable of welding on the side if it is large (20 t or more)

③ A ship with little rolling (with a certain depth of draft)

④ A ship ensuring a space where a compiling PC or processor does not get wet due to the weather, splash, or others

⑤ A ship that can be dedicated to the work during the work period (no day-by-day rigging is required)

2.7.5 Rigging and Test

Rigging means equipping and installing the main body of the multibeam sounding apparatus and its peripherals on the surveying ship. They need to be firmly secured so that their mounting positions do not move during measurement.

Each equipment shall be checked if working normally by confirming the movements of each equipment and a test measurement by sailing of the surveying ship after the rigging has been finished.

(1) Confirmation of the Accuracy of GNSS[48)]

GNSS must be sufficiently accurate regarding the reference mark survey used for the water depth survey and the method for offshore positioning. The accuracy of GNSS must be confirmed before the survey according to the following operation standards:

The accuracy shall be confirmed in a prior check at an existing reference mark.

① The observation time shall be 10 min or more, and the recording interval shall be once or more per second.

② The allowable measuring error in offshore positioning shall be 0.5 m or less.

③ The result of the observation shall be compiled in the GNSS accuracy administration table.

Although no GNSS positioning method to be used is stipulated here, the accuracy of D-GNSS or more shall be required to meet the work process control standard of dredging works.

It is necessary to use a system with the positioning accuracy which can ensure the accuracy to properly reproduce the final point group data. It is common that the GNSS equipment to be used is preliminarily accepted by the client.

(2) Mounting of the Equipment (Offset)

The positional relation between the main body of the multibeam sounding apparatus and its peripherals shall be clarified, and the equipment shall be mounted so that the positional relation does not also change during the measurement.

The measured offset values shall be recorded in the multibeam sounding system inspection log. If the rigging condition changes, be sure to measure again.

An example of measurement items of the required offset values is shown in **Fig. 2.7.9**.

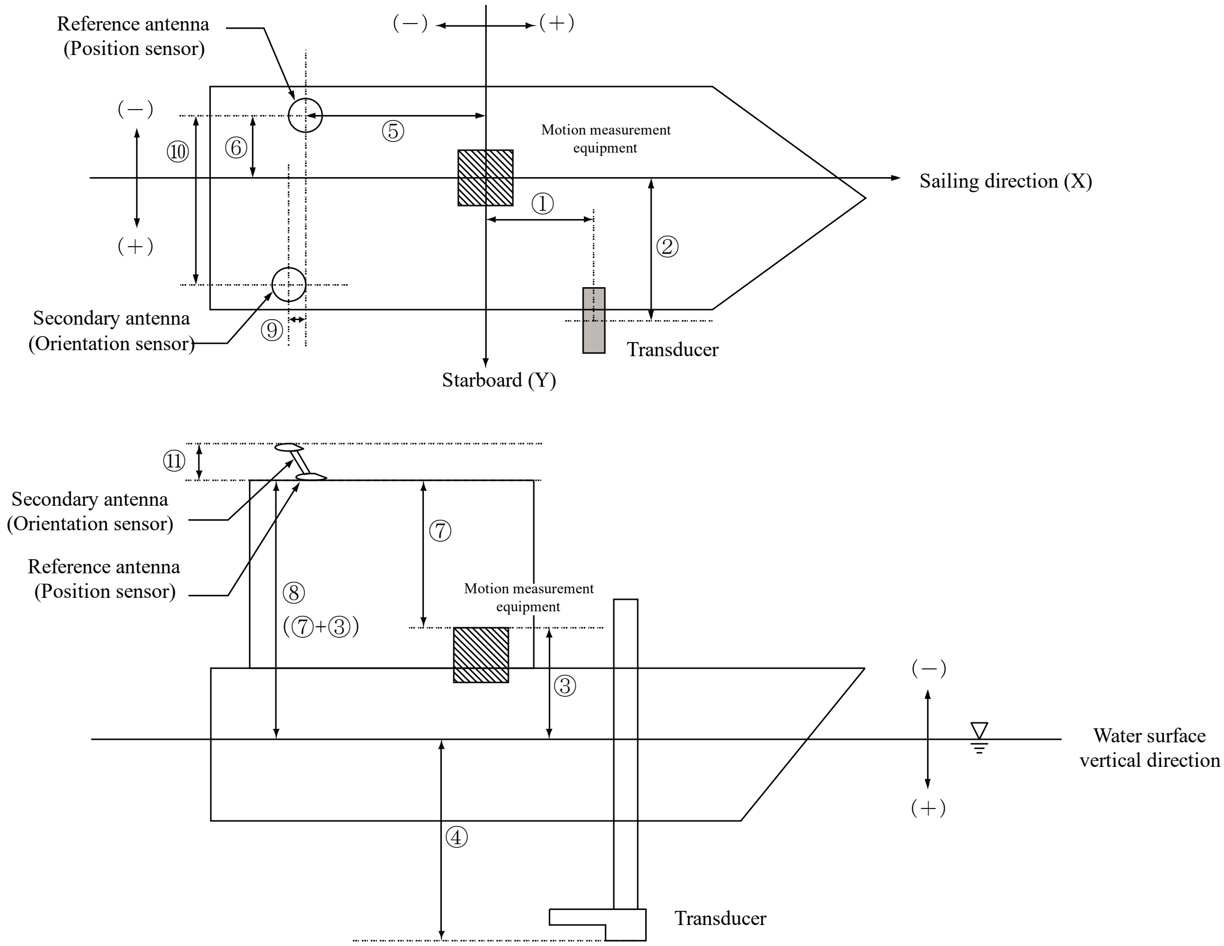


Fig. 2.7.9 Measurement Items of Offset Values

① Points to note when rigging

Each equipment shall be secured with ropes and others to prevent it from spinning. Interference of cables shall also be kept in mind.

If the rigging condition is changed (when rigged again to fix loosening or others, when draft has been changed, when orientation of the sonar head has been changed, etc.), be sure to measure again and record that statement in the sounding log.

(a) The transducer shall be secured perpendicular to the water surface by selecting a place which is less impacted by bubbles generated by the surveying ship during sailing. (It is generally preferable to secure the transducer near the bow or ship’s motion axis (pitch direction) and below the ship’s draft or below 1 m or more from the sea surface, but the situation may differ according to the structure of the ship.)

(b) The transducer must be secured rigidly with wire or other materials on the ship’s front-back direction and the lateral direction (girth) so that it does not vibrate.

(c) The motion sensor shall be rigged near the center of the ship’s motion or near the transducer in the same direction as the transducer.

(d) The orientation sensor shall be rigged in the same direction as the transducer.

(e) The position sensor (GNSS) shall be rigged near the transducer where sky can be secured.

(f) After rigging, measure the position relation of each instrument with a measure or the like, record and input so that measuring can be started soon.

② **Position in horizontal direction**

The relative position shall be measured as precisely as 1 mm referring to the position specified by each system or the recording software. The result of the measurement shall be entered into the recording software, written on the multibeam sounding inspection log, and confirmed to be properly used in data processing.

③ **Position in the vertical direction**

The relative position shall be measured as precisely as 10 mm referring to the water surface. The result of the measurement shall be entered into the recording software, written on the multibeam sounding inspection log, and confirmed to be properly used in data processing.

④ **Mounting position of equipment**

The mounting position of equipment ≈ offset value shall be confirmed to be applied to correction for the result of sounding by entering to the data recording and processing software.

(3) Confirmation of Draft

① **Method for confirmation of draft**

The draft shall be confirmed with a bar check. Set the water surface as a reference (0 m), hang and secure a reflector at several meters below, measure using a multibeam sounder, and record the distance from the sonar head to the reflector. The value of the hanging length from the water surface as a reference minus the distance between the measured sonar head and the reflector shall be the draft value. Repeat this process three times and confirm the draft value with the mean value.

Also, measurement using a leveling rod and reading of the draft scale appended to the mounting pipe shall be done simultaneously.

② **Points to note when confirming the draft**

The rope used for confirmation of the draft shall be measured in advance, and it shall be confirmed that there is no expansion or contraction. In addition, to eliminate errors caused by motions during the confirmation work, select a sea area which is as calm as possible near the sounding sea area.

(4) Patch Test

The multibeam sounding system is basically rigged as horizontally and vertically as possible to the water surface, but the shape of the ship and the tension of the securing wire or others inevitably cause installation errors. A patch test shall be done in order to know the misalignment in the installation angle of the transducer of the multibeam sounding system (hereinafter, “bias value”) and the recording delay (hereinafter, “latency”) of each equipment. If the rigging condition changes during sounding, repeat measurement and conduct a patch test every day as a slight misalignment during working causes an error.

The patch test shall be done basically on the parallel survey line and on the turnaround survey line. It shall be done on the crossed and other survey lines if needed. It is preferable to do the patch test where stable data can be obtained, such as beach, inside a port, and others, avoiding markedly uneven sea areas, such as reef.

① **Types and methods of the patch test**

The following bias values and latency shall be obtained using the patch test.

<Bias values>

Roll: lateral installation angle to the navigation direction of a ship

Pitch: installation angle to the navigation direction of a ship

Yaw: orientation of the transducer to the navigation direction

Latency: delay (recording delay from the request of data transfer to the equipment to its response)

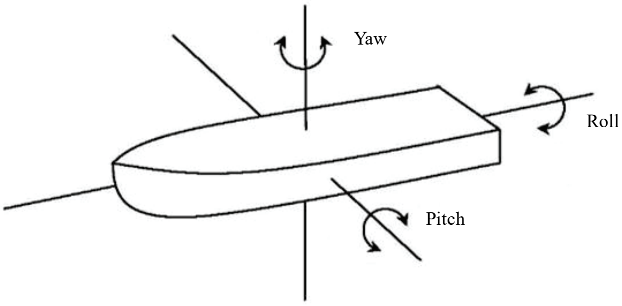


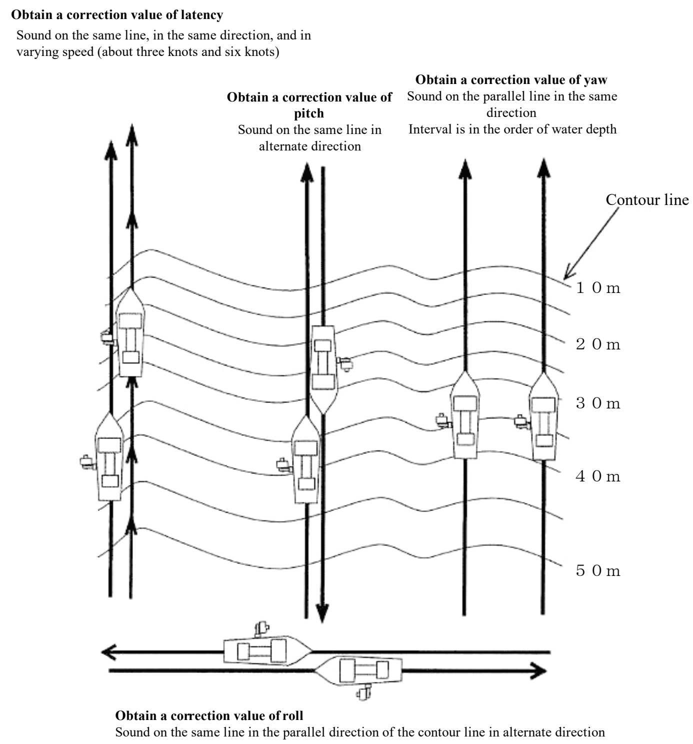
Fig. 2.7.10 Types of Bias Values

② **Measurement condition in the patch test**

It is preferable to measure in the patch test under the following conditions.

<Examples of the survey line setting in the patch test>

*Select a place where the sea bottom is oblique, if possible, and sound for about 300 m.



<Outline of the patch test by bias values>

Bias value	Sea area	Navigation method
Roll	Inclination: less than 1%	Round trip on the same survey line at the same speed
Pitch	Inclination: 5% or more	Round trip on the same survey line at the same speed
Yaw	Inclination: 5% or more	Two trips on parallel survey lines in the same direction at the same speed
Latency	Inclination: 5% or more	Two trips on the same survey line in the same direction at low and high (more than twice of the low) speed

Fig. 2.7.11 Measurement Conditions in the Patch Test

The patch test shall be done basically on the parallel survey line and on the turnaround survey line. It shall be done on the crossed and other survey lines if needed. Since the patch test is an effective way to correct installation errors of a transducer, it is preferable to do the patch test where stable data can be obtained, such as beach, inside a port, and others, avoiding markedly uneven sea areas, such as reef.

In addition, the patch test assumes that the position relation between peripherals and the transducer has not changed. Therefore, the patch test shall be conducted every day as a slight misalignment during working caused by refastening of securing wire of the transducer or collision of the sonar with a driftwood and so on becomes an error factor.

2.7.6 Bathymetric Survey Using Multibeam

(1) Measurement of Underwater Sound Velocity

① **Measurement method of underwater sound velocity**

The underwater sound velocity shall essentially be measured using the underwater sound velocimeter once or more times a day at the deepest part of the target surveying range. The measurement location shall also be recorded. Moreover, the result of the recording shall be compiled in the result table of sound velocity measurement.

If this method cannot be used, the sound velocity can also be calculated from the pressure, water temperature, and salinity and used after prior consultation.

The sound velocimeter to be used shall be calibrated by the STD (salinity-temperature-depth profiler: an equipment used to observe salinity, temperature, and water depth) or the CTD (conductivity-temperature-depth profiler: an equipment used to observe conductivity, temperature, and water depth) once or more times a year. Or use an equipment verified by the bar check method.

(Refer to Appendix 11 **Underwater Sound Velocity Calculation** in **Detailed Enforcement Regulations for the Rule of Hydrographical Survey Works** (April 27, 1983, Hosuikai No. 13).)

② **Entry of measurement values**

Compile the result of measurement during descent and ascent of the underwater sound velocimeter to the mean values at least every 1 m and create the result in a form that can be used by the recording and analysis software. Enter the created underwater sound velocity data into the analysis software and apply it.

(2) Points to Note When Sounding

The multibeam sounding work shall be conducted understanding the following points to consider in the whole work and the error factors. If the data stability is poor, then another measurement must be done again.

① Sail at a constant speed and avoid sharp turns, quick acceleration, and quick deceleration that may become error factors during the sounding work.

② The sounding operator shall watch the PC’s real-time screen and coordinate various functions, such as transmitting and receiving signal and TVG (time varied gain: a method to make the level of returning signals constant by changing the gain of amplifier over time) and so on to get data with good S/N ratio (signal/noise ratio).

③ Avoid a dot sequence on the arc with the radius of the beam just below (prevention of the tunnel effect).

④ Try to grasp the information concerning the target surveying sea area (near the fish reef or fish preserve, relation of the bottom sediment condition, etc.) to distinguish the fish school or sea weed near the sea bottom from the sea bottom, compare with the display profile and verify on board if there is missed measurement.

⑤ Check the capturing condition of the shallowest part of the reefs and others and consider complementary remeasurement. The shallow check shall be conducted twice at ultra-low speed. Moreover, when using a model which can sound more points in equidistance mode, utilize the function and sound densely.

⑥ Sounding shall be done when the water surface is as calm as possible and avoid situations where there are waves.

(3) Error Factors in Sounding and Handling Methods

It is necessary to specifically make sure that the following phenomena, which are characteristic error factors in multibeam sounding, are not occurring:

① **Tunnel effect phenomena**

In the topography sounding data of the flat seabed, there exists a phenomenon in which the strong reflection echo just below the transducer enters into the echo near both edges to be erroneously recognized as a shallow seabed. There is another case where backscatter data is recognized (see **Fig. 2.7.12 (a)**).

A countermeasure against these phenomena is to weaken the transmission power and coordinate to reflect evenly in an entire echo.

② **Smile curve phenomenon**

This phenomenon occurs when the beams at both edges become weakly reflecting echo on flat sea-beds and the center of gravity of the measuring slit moving inside makes the measurement too shallow or the result of underwater sound velocity measurement is incorrect (see **Fig. 2.7.12 (b)**).

A countermeasure against this phenomenon is to lessen the range of weakly reflecting echo by narrowing the swath width or to obtain and analyze correct sound velocity structure by increasing the count of underwater sound velocity measurement.

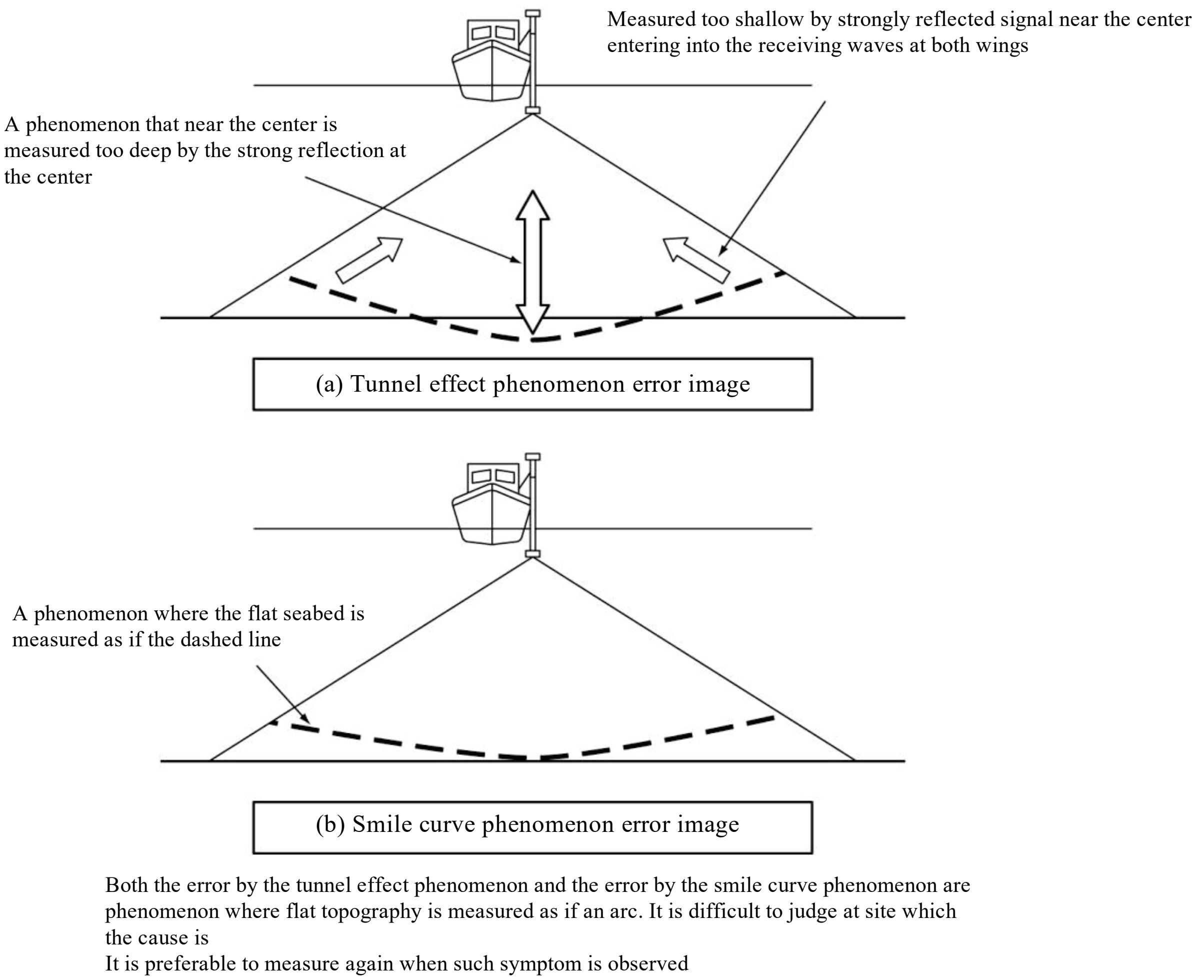


Fig. 2.7.12 Error Images of Error Factors (1/2)

③ **Ghost phenomenon**

This is a phenomenon where data is generated by irregular reflection in the L-shaped part at the base of port structures, such as quaywalls or breakwaters, and the L-shaped part is measured too shallow. The same phenomenon occurs in steep cliff topography and others (see **Fig. 2.7.13 (c)**).

This phenomenon can be restricted to a certain degree by decreasing the ship speed and suppressing the transmission power as a countermeasure.

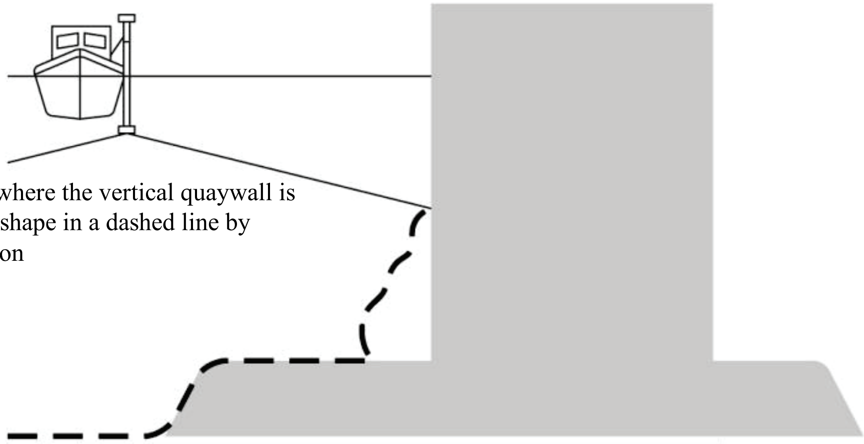
④ **Motion correction error phenomenon (caterpillar phenomenon)**

By not correcting each motion component (roll, pitch, yaw, and heave) correctly, the flat topography of the seabed deflects up and down and right and left per beam shot, and a phenomenon of becoming patterned indented topography like a caterpillar occurs (see **Fig. 2.7.13 (d)**).

As this phenomenon is hard to confirm during the sounding work, it is important to correctly set various filters of the motion sensor in advance. Also, when the oceanographic phenomena condition is worsening, it is recommended to suspend the work immediately since motion sensors may not perform reliably.

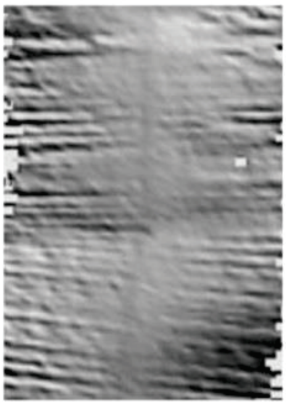
Furthermore, analyze the data, check if the caterpillar phenomenon is occurring, and judge if it is resolved by coordination of the correction value of the motion component. If it cannot be resolved, repeat measurement.

A phenomenon where the vertical quaywall is measured as the shape in a dashed line by irregular reflection



(c) Ghost phenomenon error image

A phenomenon where the flat seabed is measured as the patterned indented topography, such as caterpillar traces, as shown in the figure on the right



(d) Caterpillar phenomenon error image

Fig. 2.7.13 Error Images of Error Factors (2/2)

2.7.7 Verification of Accuracy[49)]

Recorded data shall be verified with the double cross survey lines as shown in **Fig. 2.7.14**. The double cross survey lines set two parallel sounding lines and two sounding lines that cross at right angles with them so that the right and left beams overlap 100% and evaluate the accuracy with the difference in water depth at the overlapping part of this data.

The criteria of evaluation shall comply with the Appendices 1 and 2 of the **Japan Coast Guard Public Notice No. 102** (April 1, 2002) and the **Japan Coast Guard Public Notice No. 110 Partial Amendment** (March 31, 2009).

The inspection work shall be conducted once a day, and that everyday data is measured correctly shall be verified. As the point data on the overlapped survey line is not measured at the very same position, compare and verify in the mesh size of resolution required for work process control.

The inspection shall be conducted at proper locations near the sea area to be examined (avoid oblique parts or markedly uneven places).

In addition, accuracy in measurement shall also be checked with the bar check (check the distance from the sonar head with a reflector hanged at a constant depth).

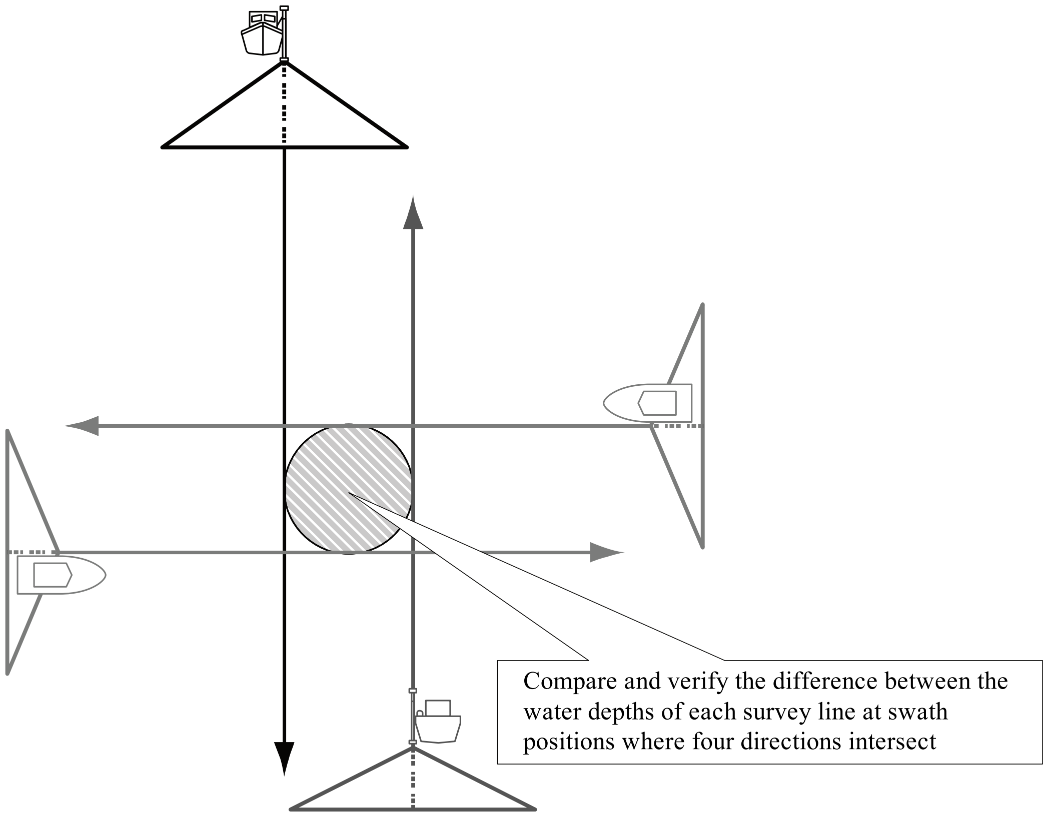


Fig. 2.7.14 Image of the Accuracy Verification Survey Line

2.7.8 Organization, Analysis, and Compiling of the Sounding Data

(1) Flow of Data Organization and Analysis

The result of the corrected and noise-processed bathymetric survey data using multibeam shall be organized in the whole target sea area by survey line and saved as point group data with the recorded horizontal positions and water depths. Besides the principal data (random point group data), various correction data, such as underwater sound velocity and tide level, offset values, result of the patch test, and others, shall be organized and saved.

A general flow of data organization and analysis is shown in **Fig. 2.7.15**.

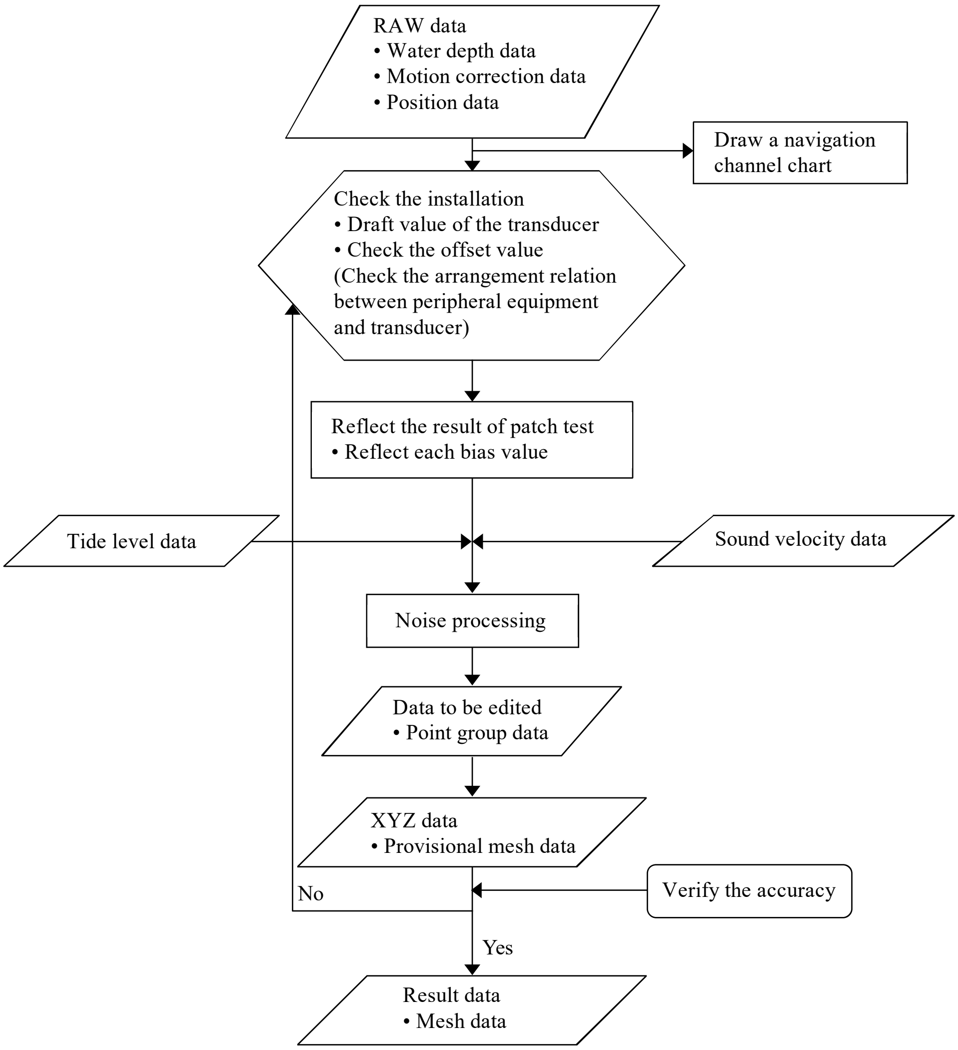


Fig. 2.7.15 Organization and Analysis Flow of the Multibeam Sounding Data

① **Drawing of track chart**

Check the geodetic datum, projection, and coordinate origin and draw a track chart from the RAW data.

② **Creation of the tide level data and the underwater sound velocity data**

Edit the tide level data from the gage datum and each day's underwater sound velocity data in the analysis software format of the multibeam sounding system (confirm that the tide level or the underwater sound velocity data have no outliers at this time).

③ **Reflection of bias values by the patch test of transducers**

Reflect the installation angle value obtained in the patch test on the analysis data.

④ **Noise processing**

Identify noises by the survey line and eliminate data other than the topography of seabed. In addition to acoustic or electric noises, noise is also generated from floating objects in the sea, such as floating matters, fish school, and bubbles. Noises can be statistically eliminated to some extent using the analysis software (see **Fig. 2.7.16**), but as there is a limit in statistical processing, it is finally necessary to display the profile and eliminate noises manually. Records that are difficult to judge shall be judged comprehensively by leaving images and others and from records of other survey lines.

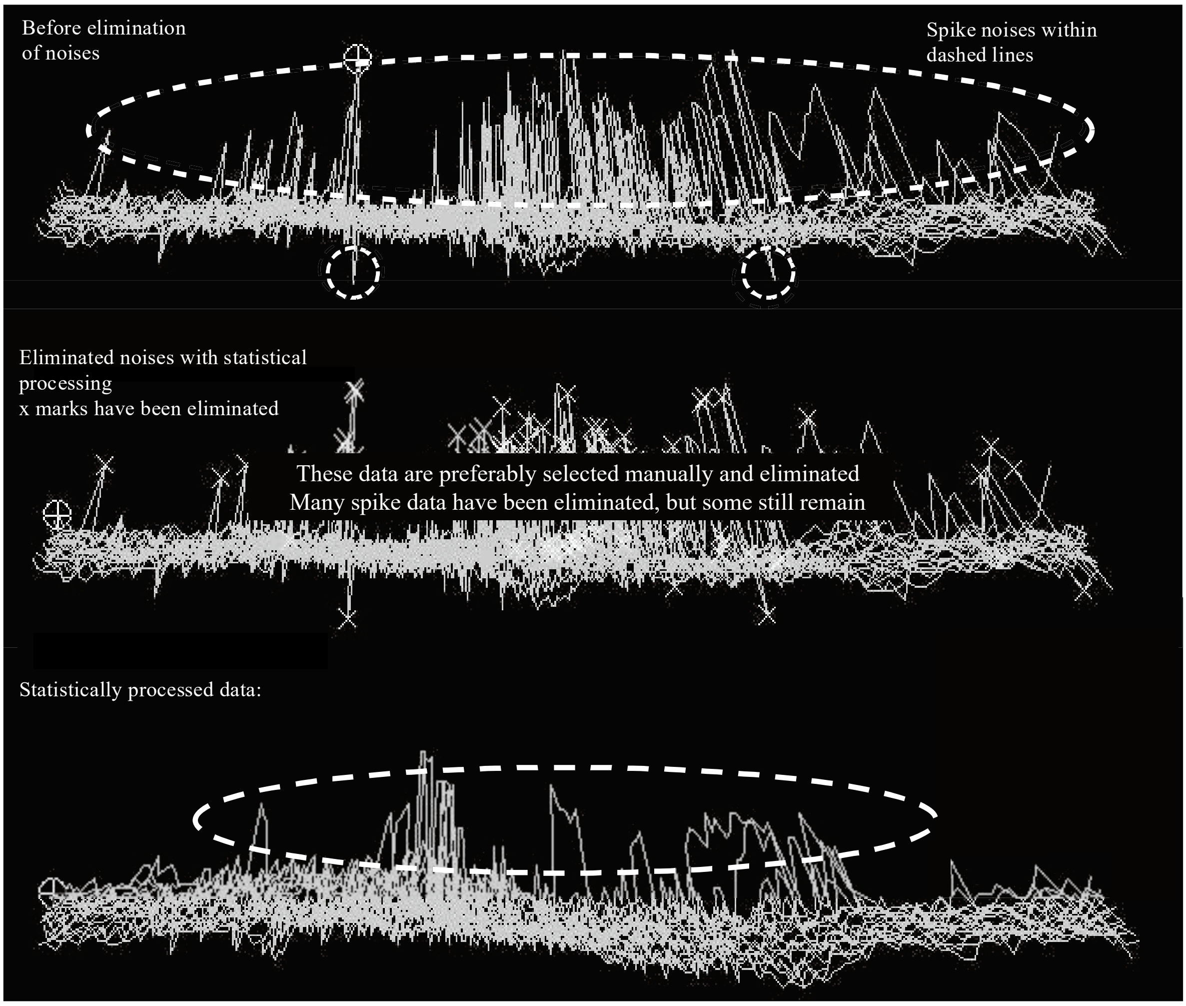


Fig. 2.7.16 Examples of Noise Elimination

⑤ **Points to note when editing data**

It is important that various correction data have been created correctly. In addition, it is necessary to confirm with special care that the phenomenon which is a characteristic error factor in multibeam sounding as shown in **Reference [Part II], Chapter 1, 2.7.6 (3)** has not occurred. It is also necessary that the required number of data is secured even after the number of data has decreased due to noise elimination.

⑥ **Integration of data**

Integrate the noise-eliminated point group data by the survey line by the target area.

⑦ **Creation of XYZ (provisional mesh) data**

Create a result-sized mesh or smaller-sized mesh data by using the edit data. Check that there is no mismatch between the survey lines and so on from automatic contour line drawing, bird's eye view (whale's eye view), and others. If there is a mismatch, the bias value or sound velocity data are often the cause of the problem. Thus, check each data and repeat from the patch test.

⑧ **Verification of accuracy**

Eliminate noises from the survey line for accuracy verification and verify if the noise is within the allowable accuracy error range.

⑨ **Creation of a result data**

If it has been confirmed that there is no problem in the accuracy verification and others, create a mesh data for deliverables.

(2) Creation of the Principal Data[50)]

The result of corrected and noise-processed bathymetric survey shall be compiled for the entire target sea area by the survey line and saved as point group data with the recorded horizontal position and water depth. After confirming that the point group data is dense enough to be provided to the soil volume calculation and the work process control and others, create the point group data of 0.5 m plane grating per point.

The following points should be noted when creating data for obtained point density and the point group:

① Cover the whole target surveying sea area with a 0.5 m plane grating and ensure that the obtained point density is three or more for 90% or more of the total number of plane grating (achievement ratio: 90% or more).

② However, plane gratings with less than three points must not be distributed continuously as shown below.

③ After confirming the achievement ratio and that plane gratings with less than three points are not distributed continuously, extract a median or the shallowest value within the 0.5 m plane grating and create a point group data for 0.5 m plane grating per point. For plane gratings with less than three points, making it difficult to extract a median or the shallowest value, an interpolation shall be possible with space analysis or others from the surrounding extraction point data.

- When used for soil volume calculation: median
- When used for work process control: the shallowest value

* If dredging points are scattered or for dredging works in a special sea area where the topography of seabed is frequently changing due to the effect of sand wave or others, another consideration may be taken.

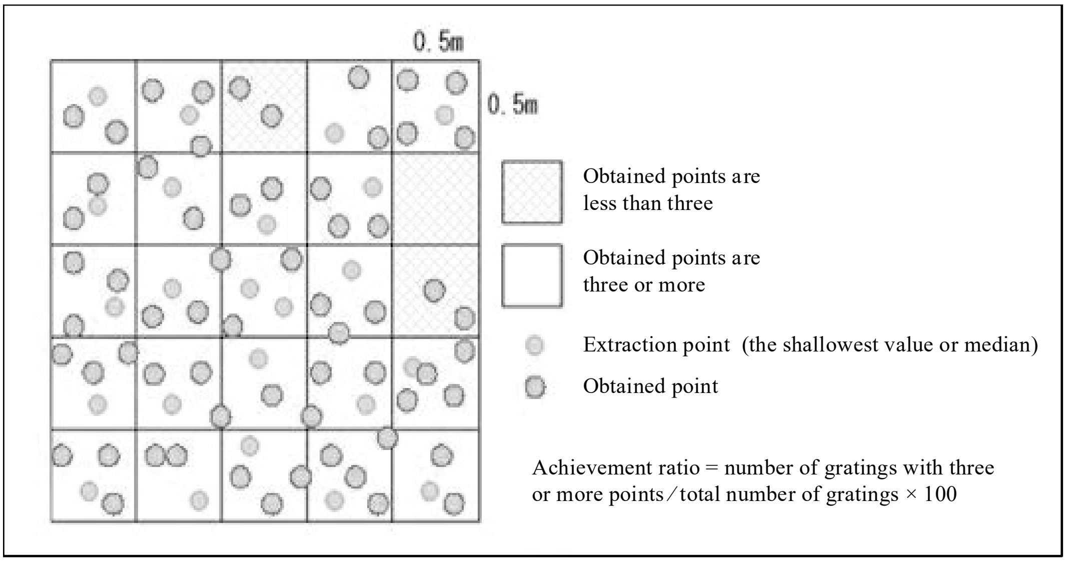


Fig. 2.7.17 Concept of the Data Density

The principal data (random point group data) shall be saved in a space or comma separated text format recording the plane position (X, Y) and the depth (Z) from the datum level as a readable format by software generally used for work process control and volume calculation.

(3) Conversion to 3D Data and the Deliverables

Deliverables shall be judged by the purpose of business and the future use and created. The point group data shall be processed in several ways shown below to be used for creating mesh data as a basis of deliverables. When creating a resulting mesh, it is necessary to examine a method suitable for the purpose because the result may take a different value by the method.

① **Nearest neighbor method**

A method to employ the nearest point to the interpolation point. The process is rapid but subject to density.

② **TIN method**

A method to generate a triangular group from the point group and obtain an interpolation point from the triangular plane by linear approximation. The model reflects the measurement data.

③ **IDW method**

A method to obtain by weighing according to distance to the interpolation point for each point group at a certain distance (or a certain number) from the interpolation point. The water depth value varies according to the weighing range.

④ **Kriging method**

A method to interpolate with weight considering spatial correlation. The processing takes longer as the point density becomes higher.

⑤ **Mean method**

A method to average each point group within a certain distance from the interpolation point. The reliability of the result lessens when the point density is low or the search range is wide.

⑥ **Shallowest value method**

A method to take the shallowest point group within a certain distance from the interpolation point. The reliability of the water depth value lessens when the point density is low or the search range is wide.

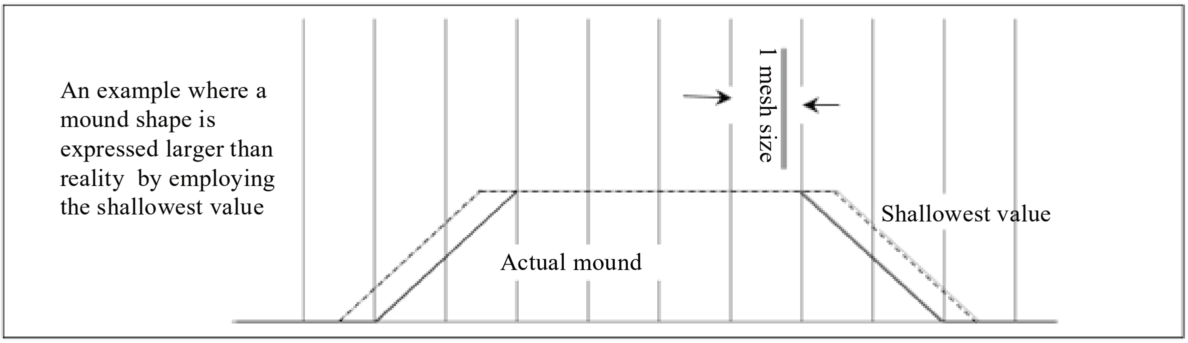


Fig. 2.7.18 Image of an Error Example in Meshing

Among the abovementioned methods, the TIN method is the most general digital data structure to express the surface profile, such as topography or work process profile, and the point group data is processed in this method[51)].

TIN is an acronym of Triangular Irregular Network, which expresses the topography of seabed as a collection of triangles so that many points are linked by straight lines on three dimensions to become the apex of apexes of triangles.

General results of bathymetric surveys are track chart, water depth chart, contour line chart, etc. The interval of contour lines in water depth chart is generally determined in the discussion with the ordering party as it is difficult to display all the created resulting mesh data points on the space of the chart.

It becomes possible to create a point group drawing (**Fig. 2.7.19**), bird's eye view (whale's eye view) (**Fig. 2.7.20**), contour line chart (**Fig. 2.7.21**), and others by using the created resulting mesh data and understand the details of sea bottom condition visually and quantitatively. There are many expression methods for them.

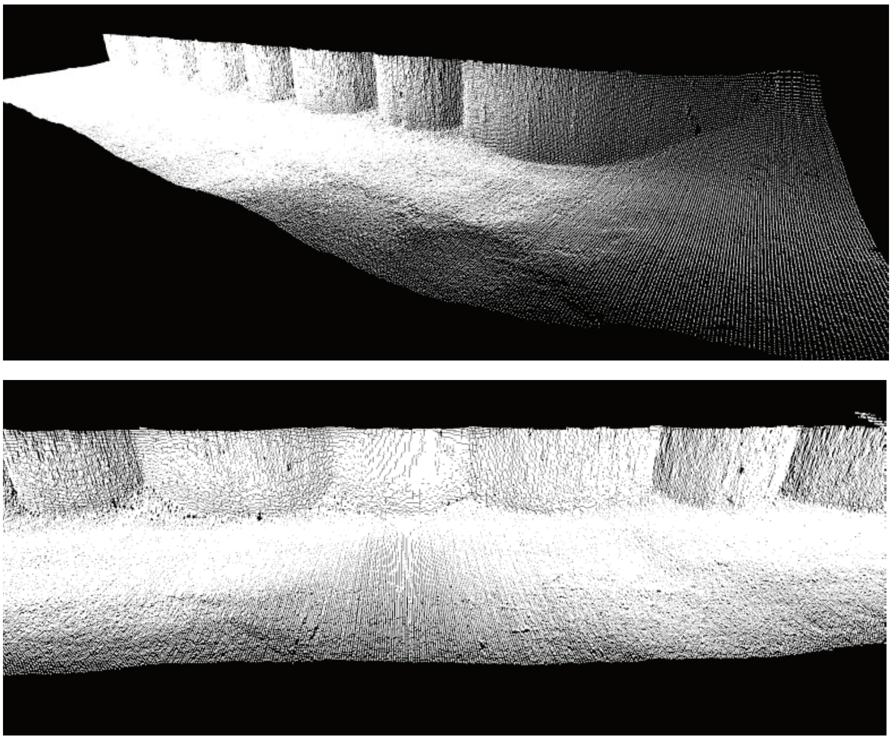


Fig. 2.7.19 Example of a Created Point Group Drawing

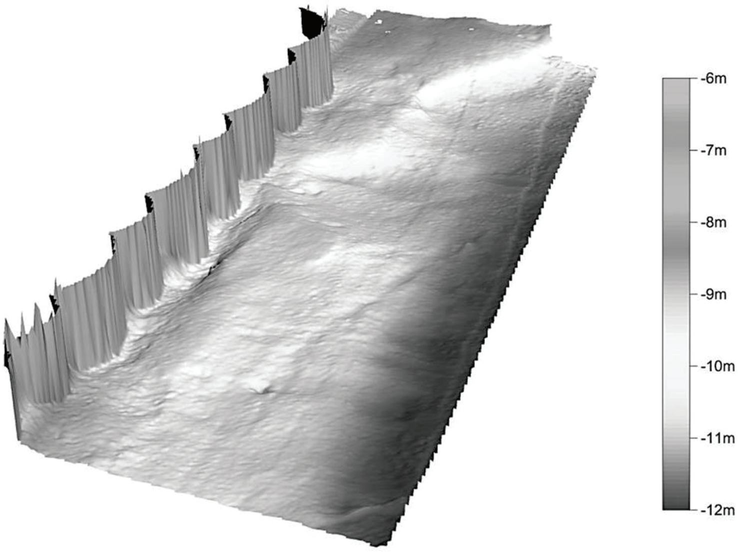


Fig. 2.7.20 Example of a Created Bird's Eye View (Whale's Eye View)

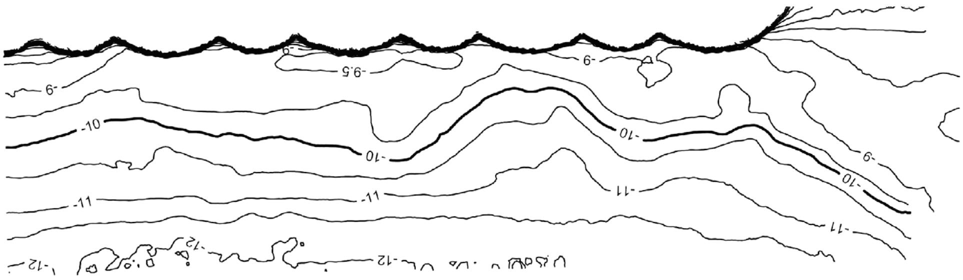


Fig. 2.7.21 Example of a Created Contour Line Chart

2.8 Observation and Investigation of the Flow of Water, etc.

2.8.1 Overview

(1) Purpose of the Flow Condition Investigation

The flow of water in the sea is caused by overlapping phenomena from different factors, such as ocean current, tidal current, wind-drive current, density current, and nearshore current. It may move the bottom sediment of the sea bed and influence the maintenance of port facilities, such as burial of navigation channels and basins and scouring around the facilities. Moreover, change in the flow of the sea by coastal development influences the natural environment, such as changes in water quality, bottom sediment, and biota. For this reason, it becomes important to clarify the flow condition of seawater and other aspects of the coastal zone, including ports and navigation channels, with a flow velocimeter, etc., in advance.

Table 2.8.1 shows the purpose of flow condition observation in harbor improvement.

Table 2.8.1 Purpose of the flow condition observation

Port Maintenance stage	Purpose of investigation	Flow condition observation					Flow condition analysis	
		Tidal current	Ocean current	Wind-drive current	Density current	Nearshore current	Data analysis	Numerical calculation
Planning of the facilities	① Grasp an environmental property	◎		○		◎	◎	◎
Environmental impact assessment	② Future prediction of flow condition and water quality	◎		○	○	◎	◎	◎
Design of structures	③ Face line plan, etc.	○	○			○	○	
Plan of construction work Supervision of construction work	④ Prediction of diffusion of muddiness, etc.	○	○	○		○	○	

Note 1: About flow condition observation: ◎ means an observation item which exerts important influence, and ○ means other observation items.

Note 2: About flow condition analysis: ◎ means an analysis method for item which exerts important influence, and ○ means other analysis method.

(2) Seawater Flow in the Coastal Zone

The flow of nearshore waters is caused by various natural factors, as stated above. Generally, the flow of the ocean is classified into the ocean current, tidal current, wind-drive current, density current, etc.

① **Ocean current**

Ocean current is known as general circulation of the ocean, and the factor is a flow governed by the wind blowing regularly on a global scale, the density effect in the ocean, and the Coriolis force. In Japanese waters, the Japan Current (Kuroshio), the Tsushima Current, the Chishima Current (Oyashio), etc. are known. Although the ocean current basically flows to one direction and does not change in a short time, the sphere of influence varies with the seasons. Also, a channel and the flow velocity may change significantly, as in the large meandering of Kuroshio. When these ocean currents come near the shore, a raid tide phenomenon happens in coastal zones.

② **Tidal current**

A tidal current is a phenomenon caused by the attraction (tide producing force) of heavenly bodies. It changes periodically, like a tide, and primarily comprises the reciprocating current with a half-day or one-day period. Generally, it becomes strong at a spring tide period and weak at a neap tide period. From the viewpoint of the region, the tidal current is strong in narrow channels at straits of Seto Inland Sea, etc., while it tends to be weak in the Sea of Japan or other areas with small tidal variation. Moreover, on the coast of an open sea, a reciprocating current often occurs during a one-day period. When the reciprocating tidal current is subject to the influence of topography, such as at a point behind an island in the sea area where the reciprocating current is abundant, the current does not reciprocate, but there is a phenomenon in which only one way tidal current becomes alternately stronger and weaker. Thus, the one way flow in the sea area where tidal current excels is called the tidal residual current.

③ **Wind-drive current**

The wind-drive current is a phenomenon in which the stress, according to wind strength, is transmitted from a sea surface to undersea, and the leeward flow is generated. It strongly influences the surface layer, and gradually spreads to the lower layer if the wind continues to blow in the same direction. The wind-drive current is known to grow in a season when monsoon blows and to bring blue tide to Tokyo Bay, etc.

④ **Density current**

The density difference in inland water and seawater may cause the river water to slide and spread on the seawater at the river mouth into which river water flows. This is called the density current at the river mouth. Although this flow mainly occurs on a surface layer, the impacted depth changes according to the increase and decrease of the river flow rate or the flux and reflux.

⑤ **Nearshore current**

A nearshore current is a flow generated by the coastal topography and wave transformation. The seawater surged by propagated waves to the shore cannot return offshore due to the waves coming one after another. It becomes longshore currents along the shore line or rip currents where longshore currents converge and return offshore.

⑥ **Other flows**

Inside the bay, the flow by characteristic vibration, according to the flow accompanying a tide, may occur. It is also called seiche or harbor resonance, which often lasts several minutes to several tens of minutes in Japan.

(3) Observation Method of the Flow Condition

The flow condition is generally observed continuously at a fixed point, with the flow velocimeter moored or installed on the sea bed and on a ship using the ultrasonic type flow velocity profiler to clarify the sea area's horizontal flow. Moreover, it is also observed with the multilayer type or point type ultrasonic flow velocimeter installed on the sea bed for turbulent flow measurement of the ocean.

Data continuously observed at a fixed point is used to analyze the frequency characteristic of a flow, periodicity, tidal current harmonic decomposition, diffusion coefficient and others, clarify the flow property in the target sea area and calculate a harmonic constant required for verification of a simulation, etc.

Data observed on a ship is used to understand the flow pattern in sea area in place of flow observation using the Lagrange method with a conventional flow measuring plate.

Table 2.8.2 Systematic Summary of Flow Condition Observation Equipment

Flow condition observation method	Observation equipment	Observation items, etc.
Fixed point continuous observation	Electromagnetic flow velocimeter (moored)	Multilayer possible
	Electromagnetic flow velocimeter (installed on the sea bed)	Near the sea bed
	Doppler flow velocimeter (installed on the sea bed)	Multilayer possible
Sailing type observation	Doppler flow velocimeter	Understanding the flow condition properties in the whole sea area
Ocean shortwave radar	HF Radar	

(4) Flowchart of the Flow Condition Investigation

The flowchart for each stage of ① planning, ② preparation for investigation, ③ field survey and ④ data analysis of the flow condition investigation is shown below.

① Planning

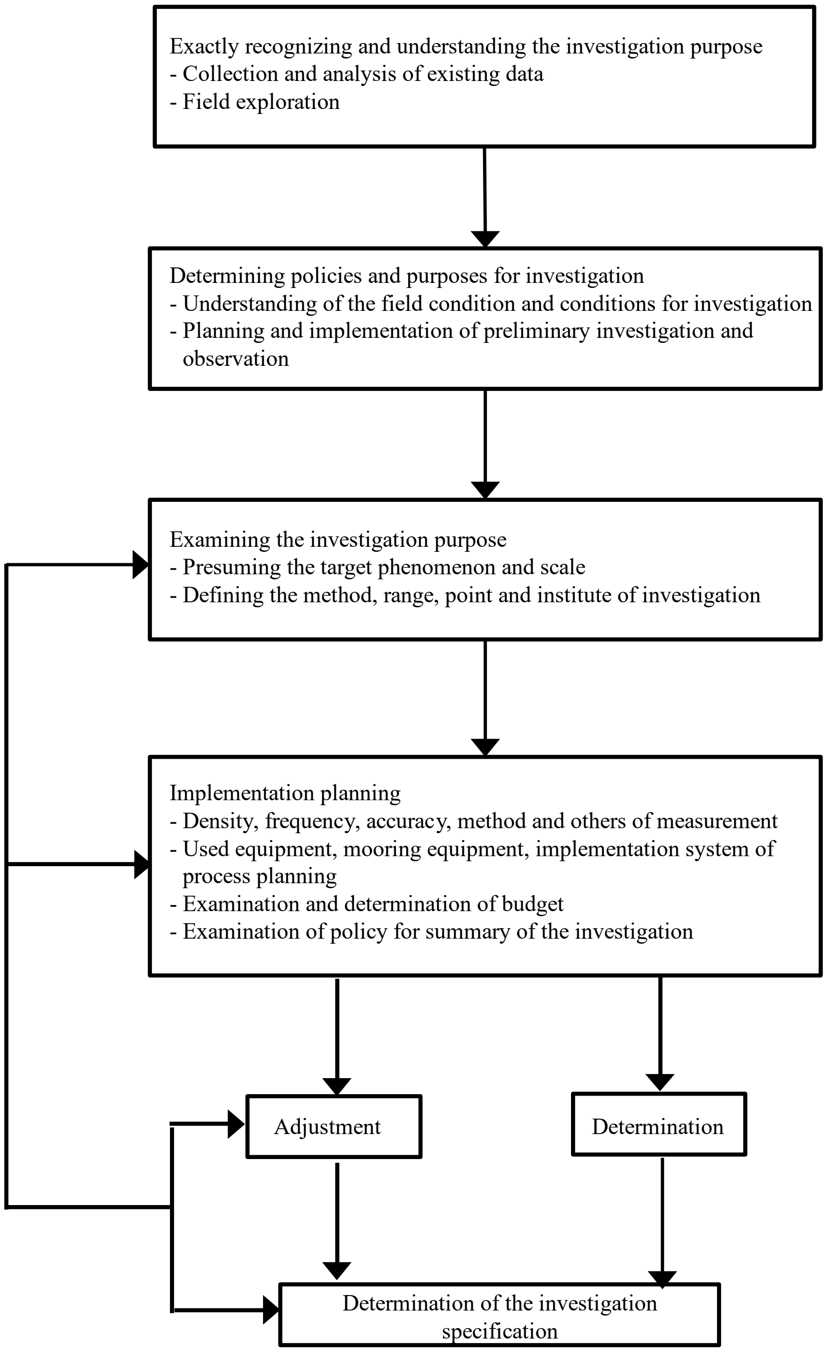


Fig. 2.8.1 Flowchart of Planning and Examination

② Preparation for investigation

- Explanation to stakeholders and agreement
- Approval and authorization from government and municipal offices
  (Water area occupancy application → port authorities)
  (Work license application and registration → competent Coast Guard Office)

③ Field survey

(Fixed-point investigation)

(Preparation)
- Equipment preparation, apparatus setup
- Equipment conveyance
- Risk assessment
- Hazard prevision activities

(Field work)
- Installation of flow velocimeters
- Inspection of flow velocimeters
- Recovery of flow velocimeters
- Clearing up

(Sailing investigation)

(Preparation)
- Equipment preparation, apparatus adjustment
- Equipment conveyance
- Risk assessment
- Hazard prevision activities

(Field work)
- Rigging of flow velocimeters
- GPS test
- Sailing measurement
- Clearing up

Fig. 2.8.2 Flowchart of Preparation for Investigation and Field Survey

④ Data analysis

(Fixed-point investigation)

1. Collection of pertinent information
(1) Weather (wind direction, wind velocity, rainfall, etc.)
(2) Hydrometeor (river flow rate, tide level)

2. Analysis of survey data
(Preliminary processing)
- Definite values of raw data except equipment noise and outliers at the time of inspection, etc.

(Output of a result)
(1) Raw data and statistics analysis
Vector diagram, decomposed velocity curve diagram, flow direction and velocity appearance frequency diagram, etc.
(2) Flow periodicity
Autocorrelation, power spectrum diagram, etc.
(3) Tidal current harmonic decomposition
Tidal current harmonic decomposition result table, tidal current elliptical diagram, flow condition diagram, etc.
(4) Flow disorder component
Diffusion coefficient diagram, intensity of turbulence, etc.
(5) Long period component
25-hour running mean diagram, mean flow diagram, etc.

3. Analysis and consideration
(1) Understanding of the flow condition properties of the sea area concerned
(2) Consideration of relations with weather conditions or external factors
(3) Comparison with the existing results of investigation
(4) Verification using a numerical simulation

(Sailing investigation)

1. Collection of pertinent information
(1) Weather (wind direction, wind velocity, rainfall, etc.)
(2) Hydrometeor (river flow rate, tide level)

2. Analysis of survey data
(Preliminary processing)
(1) Definite values of raw data except outliers such as noise
(2) After removing outliers in position data based on the track, combination of position data (latitude and longitude) and flow data

(Output of results)
(1) Horizontal distribution of flow velocity vectors, cross-sectional distribution map
(2) Time series vector diagram on survey lines
(3) Calculation and others of cross-sectional flow rate, etc.

3. Analysis and consideration
(1) Understanding of the flow condition patterns of the sea area concerned
(2) Consideration of relations with weather and hydrometeor
(3) Comparison with the existing results of investigation, etc.
(4) Verification using a numerical simulation

Fig. 2.8.3 Flowchart of Data Analysis

(5) Explanation of the Terms Concerning Flow Condition Observation

① Harmonic decomposition of a tidal current

The phenomena of tidal current and tide are closely related to the revolution of the moon and the sun. The tide theory expresses the phenomena as a sum of many tidal constituents calculated from the combination of astronomical specifications. A calculation of the maximum flow velocity and the lag (time between the passing on the meridian line at the location by a virtual heavenly body and the high tide, expressed in angle) per each tidal constituent from an actual measurement is called the harmonic analysis, or harmonic decomposition, of a tidal current. These maximum flow velocity and the lag are called the harmonic constants.

The flow is considered the synthesized flow of some simple harmonic motion flows that regularly change according to the cosine function and non-periodic flows and is expressed by the following equation.

$$V_t = U_0 + \sum_{\mathrm{i}=1}^{\mathrm{n}} \mathrm{f_i V_i} \cos\{\sigma_i \cdot t - K_i + (V_0 + u)_i\} \quad \textbf{(2.8.1)}$$

where

V_t	:	flow velocity at time t (northwardly or eastwardly decomposed velocity)
U_0	:	mean flow within a period (permanent current)
$\mathrm{V_i}$	:	amplitude of each tidal constituent
K_i	:	lag of each tidal constituent
σ_i	:	angular velocity of each tidal constituent
$\mathrm{f_i}$	:	intersection coefficient of each tidal constituent
$(V_0 + u)_\mathrm{i}$	:	astronomical argument of each tidal constituent
t	:	time
n	:	number of tidal constituents
Intersection coefficient	:	related to the position of ascending node of the moon's orbit and changes with the period of 18.6 years. The ratio of the amplitude by the equilibrium theory of tides to its mean value. That is, this corrects the amplitude.
Astronomical argument	:	a value from the time when a virtual heavenly body super transits the meridian line at an observation point to the time of an epoch, a correction value of time.

② 10 tidal constituents

These are the tidal constituents obtained from the tidal current observation for 15 days and nights (table below).

List of 10 Tidal Constituents

Classification	Sign	Name	Period	Velocity
Semi-diurnal tide	M_2	Principal lunar semi-diurnal tide	12.42h	28.984°/h
	S_2	Principal solar semi-diurnal tide	12.00	30.000
	K_2	Luni-solar semi-diurnal tide	11.97	30.082
	N_2	Principal lunar elliptic rate semi-diurnal tide	12.66	28.440
Diurnal tide	K_1	Luni-solar diurnal tide	23.93	15.041
	O_1	Principal lunar diurnal tide	25.82	13.943
	P_1	Principal solar diurnal tide	24.07	14.959
	Q_1	Principal lunar elliptic rate diurnal tide	26.87	13.399
Double tide	M_4	Lunar 1/4-diurnal tide	6.21	57.968
Compound tide	MS_4	Shallow sea tide	6.10	58.984

③ **Four principal tidal constituents**

These are four tidal constituents with large amplitude, i.e., M_2 (principal lunar semi-diurnal tide), S_2 (principal solar semi-diurnal tide), K_1 (luni-solar diurnal tide) and O_1 (principal lunar diurnal tide).

④ **15-day mean flow**

The mean flow within an observation period is called the mean flow (mean flow within a period), and the mean flow obtained from the flow condition observation result for 15 days is the 15-day mean flow.

⑤ **Tidal current ellipse**

When calculating the tidal current vector and drawing a change with time (expressed by a vector from one point, and the ends sequentially connected) by using the flow velocity amplitudes and lags per tidal constituent flow obtained from the tidal current harmonic analysis, the locus shows an elliptical shape. This diagram is called a tidal current ellipse. The tidal current element shown in an elliptical-shaped element is called a tidal current elliptical element. A tidal current element and an elliptical element correspond, as in the following table.

The tidal current ellipse is created by calculating the northward decomposed velocity and the eastward decomposed velocity by the following equation using the harmonic constant obtained from the harmonic decomposition.

Correspondence between tidal currents and elliptical elements

Tidal current	Ellipse	Tidal current	Ellipse
Flow direction at the strongest time	Direction of a long axis	Flow direction at the change of flow direction	Direction of a short axis
Flow velocity at the strongest time	Radius of a long axis	Flow velocity at the change of flow	Radius of a short axis
Strongest time	Time of a long axis	Time when flow changes the direction	Time of a short axis

$$V_{it} = \mathrm{V_i} \cos(\theta_i t - K_i) \tag{2.8.2}$$

where

V_{it} : flow velocity at time t of each tidal constituent

σ_i : 30°/h for semi-diurnal tide, 15°/h for diurnal tide

t : 0 to 12 h for semi-diurnal tide, 0 to 24 h for diurnal tide

The tidal current ellipse planarly indicates the change in time of the flow direction and velocity for each tidal constituent of the northward decomposed velocity and eastward decomposed velocity from their harmonic constants. The hodograph planarly indicates the change in time of the flow direction and velocity under specially designated conditions by synthesizing several tidal constituents. Therefore, the tidal current ellipse can be considered a hodograph for one tidal constituent.

⑥ **Flow accompanying seiche**

In a port and others, the sea surface may go up and down in a short period due to other reasons than a tide. Although the period is specific to a port, the rise and fall differ by daily weather and the state of the sea surface. These rise and fall are called seiche (harbor resonance). This is dialectally called by regions like “yota” in Shimoda in Izu Peninsula and “abiki” in Nagasaki in Kyushu. There is a flow accompanying seiche as the tidal current is the flow accompanying rise and fall of a tide.

2.8.2 Implementation of Flow Condition Observation

(1) Fixed Point Continuous Observation

Observation of a mooring or installation on the sea bed requires notification to concerned parties. That is, if the sea area to be observed is in a port area, it is necessary to apply for a water area occupancy license to a port authority and obtain permission. Moreover, offshore work requires applying for a work license or a work notification to the local Coast Guard Office (port manager if work is to be done in a port specially designated in the **Act on Port Regulations** or near its border), etc.

Although a mooring observation generally uses a two-point anchor method, as shown in **Fig. 2.8.4**, continuous observation may be done by fixing a flow velocimeter to a sea bed installation type mount, as shown in **Fig. 2.8.5**, in congested sea areas such as navigation channels. Where it is difficult for a diver to work, the flow velocimeter shall be installed on the sea bed by using a gimbal frame and a disconnecting device.

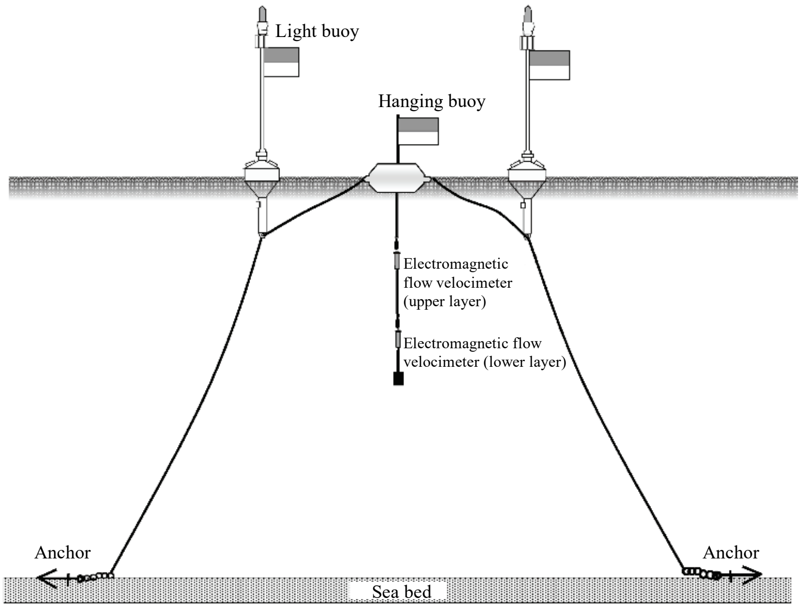


Fig. 2.8.4 Example of Mooring Observation by a Two-Point Anchor Method

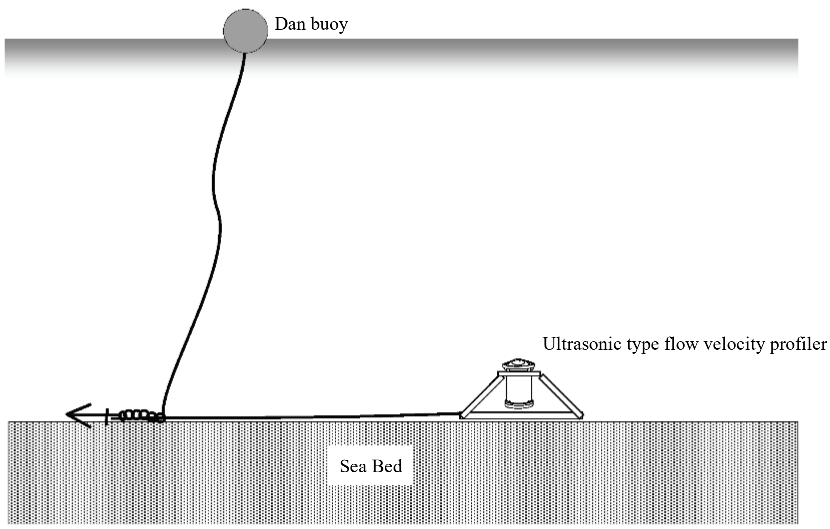


Fig. 2.8.5 Example of Observation Using a Flow Velocimeter Installed on the Sea Bed

Because of the risks of hazards or accidents, such as a collision or an outflow occurring by cruising vessels or operation of fishing boats, all parties and persons concerned must be notified.

Because of the analysis period necessary for tidal current harmonic decomposition, an inner bay sea area, like Tokyo Bay, generally requires 15 days for observation, while coasts facing the open sea require an observation period lasting one month or longer. It is preferable to determine the observation periods and times of nearshore current and others to understand conditions in both calm and severe weather, while taking seasonal variations into account.

Although measurement layers are generally surface, middle, and lower, they shall be set according to the water depth or the target phenomenon. Moreover, the measurement data of ultrasonic type flow velocity profilers, such as the Doppler flow velocimeter, has a layer thickness (for example, 1 m), and the flow direction and velocity in many layers from the measurement starting layer of equipment to the measureable layer near the sea surface may be measured at fixed intervals. In addition, in order to measure the flow near the sea bed or near the surface layer, which cannot be measured with the ultrasonic type flow velocity profiler, the ultrasonic type flow velocity profiler may be installed on the sea bed and, at the same time, an electromagnetic flow velocimeter may be used to measure near the sea bed and near the surface layer.

In a tidal current observation, the sampling interval for continuous observation data, based on the mooring or sea bed installation, has generally been 10 minutes. On the other hand, the nearshore current, turbulent flow, and others are often measured by observing at 0.5 second (or shorter, if necessary) intervals for 20 or so minutes every hour, as with wave investigation.

(2) Sailing Type Observation

On a coast with topographic changes, including the circumference of port facilities, the flow by tidal currents and others is not uniform. Complicated flows, influenced by topography, occur in many cases. Since the fixed point observation is discrete, it is preferable to grasp the flow pattern in the sea area by observing the whole area. For this reason, the horizontal distribution of flows shall be created by setting sailing survey lines according to the intended sea area, attaching the ultrasonic type flow velocity profiler to several or more work barges, and observing the intended sea area simultaneously for a short time. **Fig. 2.8.6** shows an example of such measurement. When semi-diurnal tidal current is intended, it is preferable to complete observation in about one hour, but 90 minutes shall be permitted. Flood tide times and falling tide times shall basically be observed, but if necessary, the times when the flow direction changes shall also be observed. When the tide level is used as a reference, the strongest time zone of the tidal current occurs with a time lag, and thus careful setting is needed.

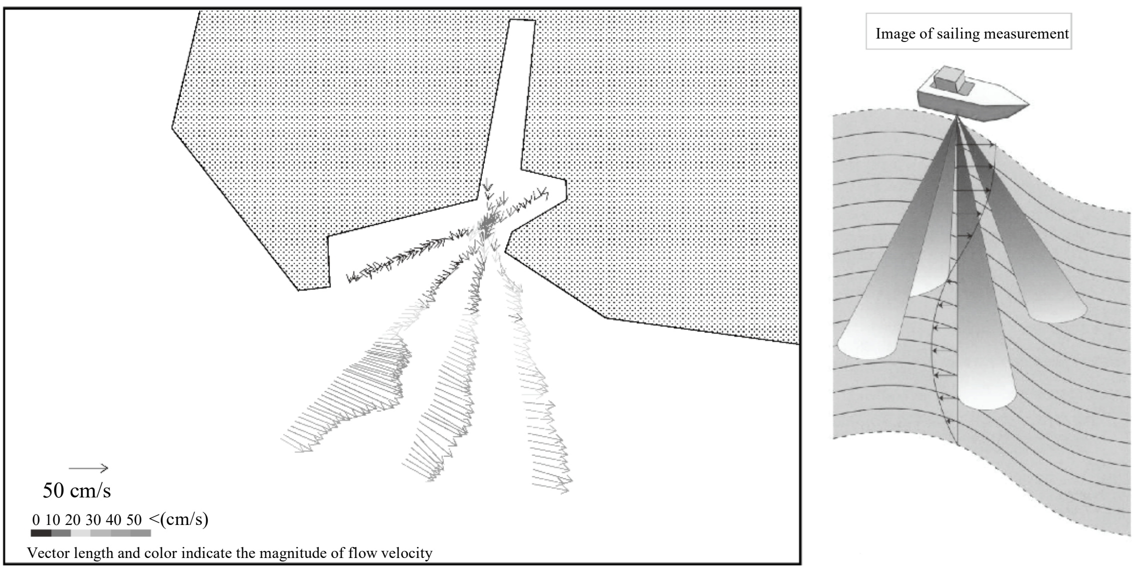


Fig. 2.8.6 Measurement Example of the Horizontal Distribution of Flow Velocity by Sailing Type Observation

2.8.3 Equipment for Flow Condition Observation

The electromagnetic flow velocimeter made by domestic manufacturers, the ultrasonic type flow velocity profiler, made by overseas manufacturers, and the single point type ultrasonic flow velocimeter are mainly used for observation now. The measurement principle and features of each equipment are outlined below.

(1) Electromagnetic Flow Velocimeter

The electromagnetic flow velocimeter (**Fig. 2.8.7**) applies Faraday's law of electromagnetic induction to measure the flow. According to the law, when the seawater, or water which is a conductor, crosses a magnetic field, an electromotive force proportional to the flow velocity of the fluid is generated, and the electromotive force and the flow velocity are linearly related. Practically, two perpendicular magnetic fields are created, and the flow velocity of the two perpendicular axes is measured by measuring the generated small voltages. The flow direction and velocity indicated clockwise from the north, which is the orientation obtained from the built-in compass, are measured from the flow velocity of the two components.

Fig. 2.8.7 Appearance of an Electromagnetic Flow Velocimeter

The electromagnetic flow velocimeter is often installed in the surface layer, and the momentary value contains the wave component. It is necessary to average by preparing the measurement time exceeding the wave period. Although domestic manufacturers recommend measuring, for example, for 30 seconds or more in one-second intervals, considering the period of big swell in the open sea and others, measuring for one minute is preferred. For nearshore current measurement, it is preferable to observe for 20 minutes or more during one or two hour intervals, as mentioned above.

When observing with an electromagnetic flow velocimeter of two horizontal components, care should be taken not to underestimate the flow velocity component when it inclines to a certain extent since there is no inclination sensor. A countermeasure to this shall be attaching a weight below the flow velocimeter.

(2) Ultrasonic Flow Velocimeter

The ultrasonic type flow velocimeter (**Fig. 2.8.8**) used in the ocean is roughly divided into the ultrasonic type flow velocity profiler and a single point type ultrasonic flow velocimeter.

Although the ultrasonic type flow velocity profiler, such as the ADCP, AWAC, and Aquadopp Profiler are available in the market, their measurement principle mostly uses the Doppler effect.

Fig. 2.8.8 Appearance of an Ultrasonic Type Flow Velocity Profiler

The ultrasonic type flow velocity profiler detects the Doppler shift of the ultrasonic obliquely beamed and reflected by the dispersion substance in the sea per each section on the measurement beam, converts the coordinate of the flow velocity on the beam in each section according to the coordinate system, consisting of four beams or three beams, together with the data obtained with a direction meter and a clinometer and measures the flow velocity of two horizontal components of a multilayer and the vertical direction.

In order to secure the measurement accuracy, a setting of_the layer thickness, the number of sampling, and others, according to the frequency of the flow velocimeter to be used, is important. Recently, models capable of measuring the specifications of waves, in addition to the multilayer flow velocity profile, have also become available.

On the other hand, the single point type ultrasonic flow velocimeter uses the Doppler shift of the flow, near the adjacent focus, or the phase difference in the ultrasonic pulse between the transducers, caused by the flow, to measure the three-dimensional flow velocity. Ultrasonic flow, such as 3D-ACM and vector, are on the market. Although these are still used for measurements in research fields, they are suitable for measuring the turbulent flow of the ocean.

The ultrasonic type flow velocity profiler can measure vertical distribution of the horizontal flow velocity independently. Living organisms do not adhere to it, unlike the electromagnetic flow velocimeter, so its accuracy degrades only slightly over many years. The single point type ultrasonic flow velocimeter can measure the three-dimensional flow velocity accurately at a short interval, such as 30 Hz.

2.8.4 Maintenance of Equipment for Flow Condition Observation

Maintenance by the models is described below.

(1) Electromagnetic Flow Velocimeter

Not only the electromagnetic flow velocimeter, but also all equipment approved within one year, should be used, although the approval interval depends on the manufacturer. Since living organisms often adhere to the electromagnetic flow velocimeter sensor in summer, regular cleaning is needed during the operation period. The standard cleaning frequency shall be about twice per week in summer in a eutrophic sea in the inner bay, and about once per week on the open sea coast. Moreover, it is effective to apply marine biofouling prevention paint to an electromagnetic sensor portion if needed. Since equipment may inevitability be washed away by a collision with a cruising vessel or other reasons, it is necessary to keep periodical data recovery from moored equipment in mind. Use of communication facilities such as COCOSECOM is an increasingly popular method for monitoring the position of moored equipment at a lower cost.

(2) Ultrasonic Flow Velocimeter

Unlike the electromagnetic flow velocimeter, the adhesion of living organisms to an ultrasonic flow velocimeter does not degrade the sensor. Moreover, it is generally difficult to directly certify the flow velocity of the ultrasonic type flow velocity profiler. The manufacturer's performance standard can be substituted for official approval.

ADCP installed on the sea bed may also be washed away by dragnet and others. One countermeasure is to collect data onboard, using a sound modem system, at the time of inspection while leaving the ADCP installed on the sea bed.

2.8.5 Arrangement and Summarization of the Tidal Current Observation Data

The first step treatment of the observed tidal current data, subsequent typical analysis items, and their deliverables are described below.

(1) Fixed Point Continuous Observation Data

For preparation, correct the measured flow direction in the magnetic north standard with the investigated magnetic deviation at the point, and convert it to the flow direction of the true north standard. Since the magnetic north is located on the west side in Japan, a -3° to -9° angle correction is required. Moreover, a time-sequence diagram (**Fig. 2.8.9**) shall be created of a decomposed velocity curve, which decomposes the flow direction and velocity data acquired every 10 minutes into northward and eastward component flow velocity, and others, and remove the abnormal data due to adhesion, the time of inspection, and others.

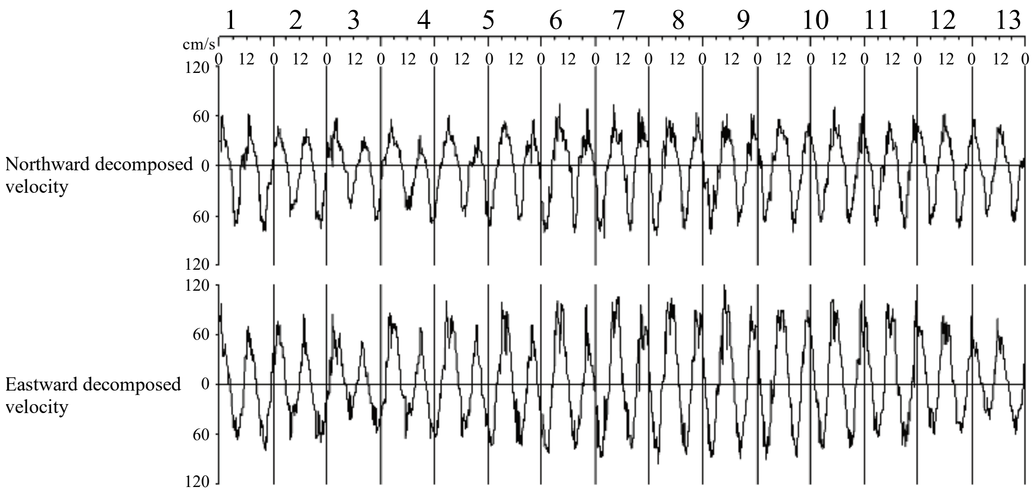


Fig. 2.8.9 Time-Sequence Diagram of the Decomposed Velocity Curve, etc.

First, a frequency distribution map (**Fig. 2.8.10**) shall be created to determine the frequency characteristic of the prevailing flow direction and others from the above-described pretreated flow direction and velocity data. That is, classify the flow direction by 16 orientations and classify the frequency of the flow velocity by orientation into classes by 10 cm/s, for example, and draw a diagram. In addition, a flow velocity frequency histogram (**Fig. 2.8.11**), by the flow velocity class, shall be created to clarify the flow velocity class of the field flow. Next, to grasp the periodic characteristic of a flow, use a spectral analysis, such as the MEM method (maximum entropy method) or the FFT method, and create an autocorrelation curvilinear diagram and a power spectrum diagram (**Fig. 2.8.12**) in the north-south and east-west direction, or mainstream direction, and its rectangular direction component. The prevailing period of the flow velocity variation shall be clarified from these analytical results. Also, to determine the main tidal current component in the longshore flow, perform a tidal current harmonic decomposition based on a least-squares method and, in the case of 15-day observation data, a harmonic decomposition result table (**Table 2.8.3**) of 10 tidal constituents shall be created. When there is observation data for one month or more, the harmonic decomposition of up to 13 tidal constituents is possible. A mean flow during the period, which is a mean value of all the data, shall also be published as an important evaluation index in the tidal current harmonic decomposition result table. A tidal current hodograph (**Fig. 2.8.13**), such as a tidal current ellipse of the main tidal constituent or a mean spring tide period, shall be created using harmonic constants by these harmonic decomposition. Furthermore, a horizontal diffusion coefficient (**Fig. 2.8.14**) as an index of the diffusion phenomenon accompanying various disorder in the sea area shall be calculated using the Taylor's theorem from the autocorrelation function of the flow velocity variation. The horizontal diffusion coefficient of the data, excluding periodic components such as the tidal current component, may be calculated considering the time scale of the target diffusion substance. In addition, in order to grasp the flow condition excluding the tidal current component, a running mean vector for 25 hours may be drawn.

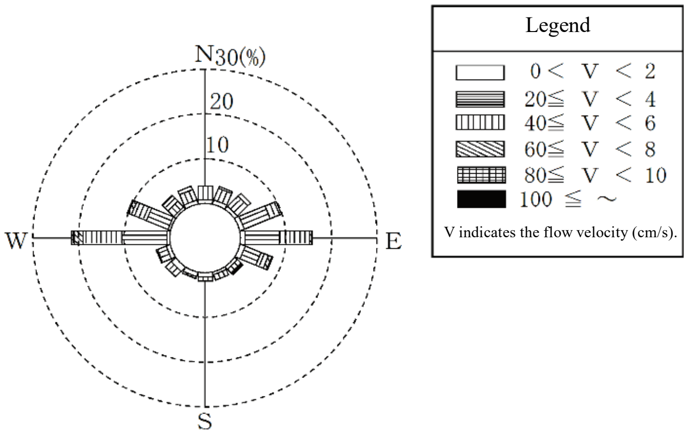


Fig. 2.8.10 Flow Direction and Velocity Frequency Distribution Diagram

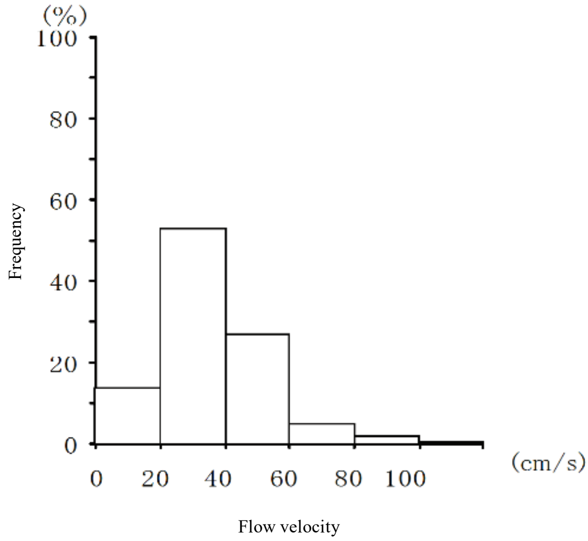


Fig. 2.8.11 Flow Velocity Frequency Histogram

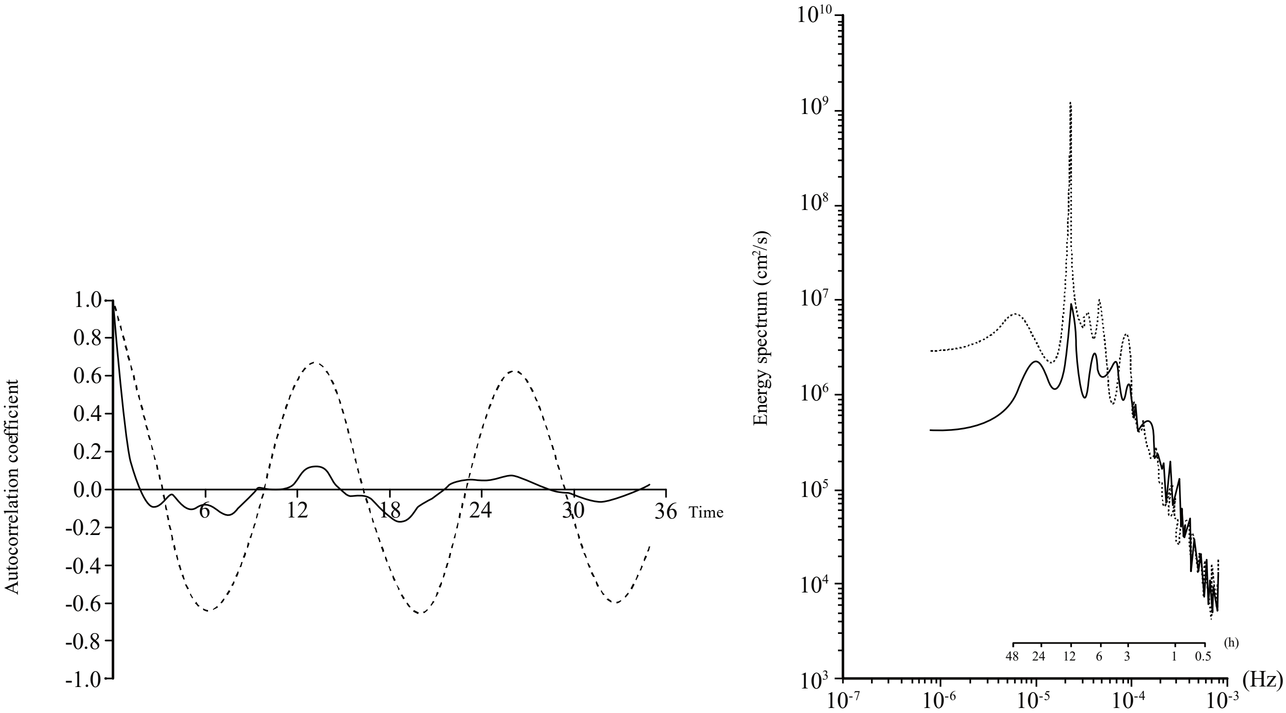


Fig. 2.8.12 Autocorrelation Curvilinear Diagram and Power-Spectrum Diagram

Table 2.8.3 Tidal Current Harmonic Decomposition Result Table

Tidal constituent	Northward component		Eastward component		Elliptical element						Main flow direction 275°	
					Long axis			Short axis				
	Flow velocity (cm/s)	Lag (°)	Flow velocity (cm/s)	Lag (°)	Direc-tion (°)	Flow velocity (cm/s)	Lag (°)	Direc-tion (°)	Flow velocity (cm/s)	Lag (°)	Flow velocity (cm/s)	Lag (°)
M_2	7.5	124	37.7	216	270	37.7	36	0	7.5	126	37.5	37
S_2	1.7	141	15.9	262	273	15.9	82	3	1.4	172	15.9	82
K_2	0.5	141	4.3	262	273	4.3	82	3	0.4	172	4.3	82
N_2	1.5	115	7.3	191	87	7.3	190	177	1.4	280	7.2	12
K_1	3.1	6	5.0	185	302	5.9	5	32	0.1	95	5.3	5
O_1	2.0	264	6.8	98	286	7.1	277	16	0.5	187	7.0	278
P_1	1.0	6	1.7	185	302	2.0	5	32	0.0	95	1.8	5
Q_1	0.4	32	1.3	55	75	1.4	54	165	0.2	144	1.3	236
M_4	4.2	151	8.0	347	297	9.0	163	27	1.0	73	8.4	166
MS_4	3.1	196	6.3	347	294	6.9	172	21	1.3	262	6.6	169
U_0	4.6 cm/s		-1.8 cm/s		4.9 cm/s			338°			2.3 cm/s	

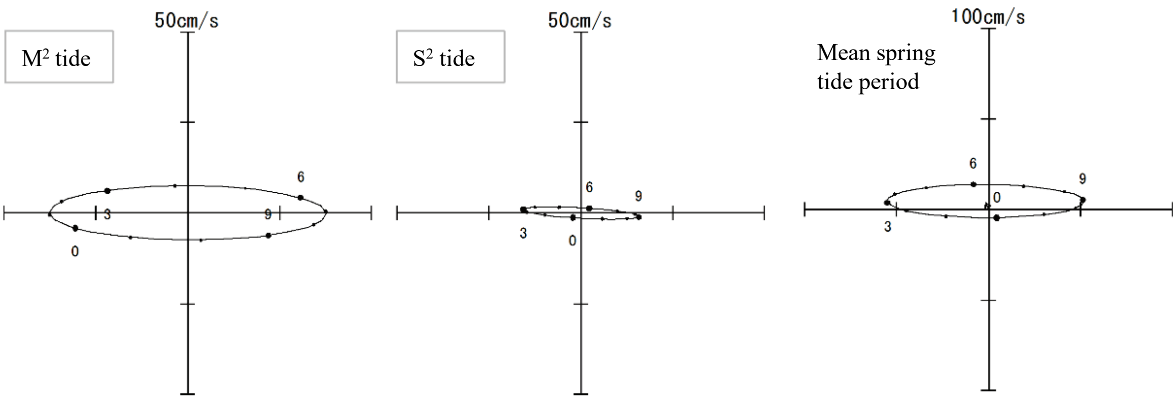


Fig. 2.8.13 Hodograph of the Tidal Current Ellipses of the Four Principal Tidal Constituents and of the Mean Spring Tide Period

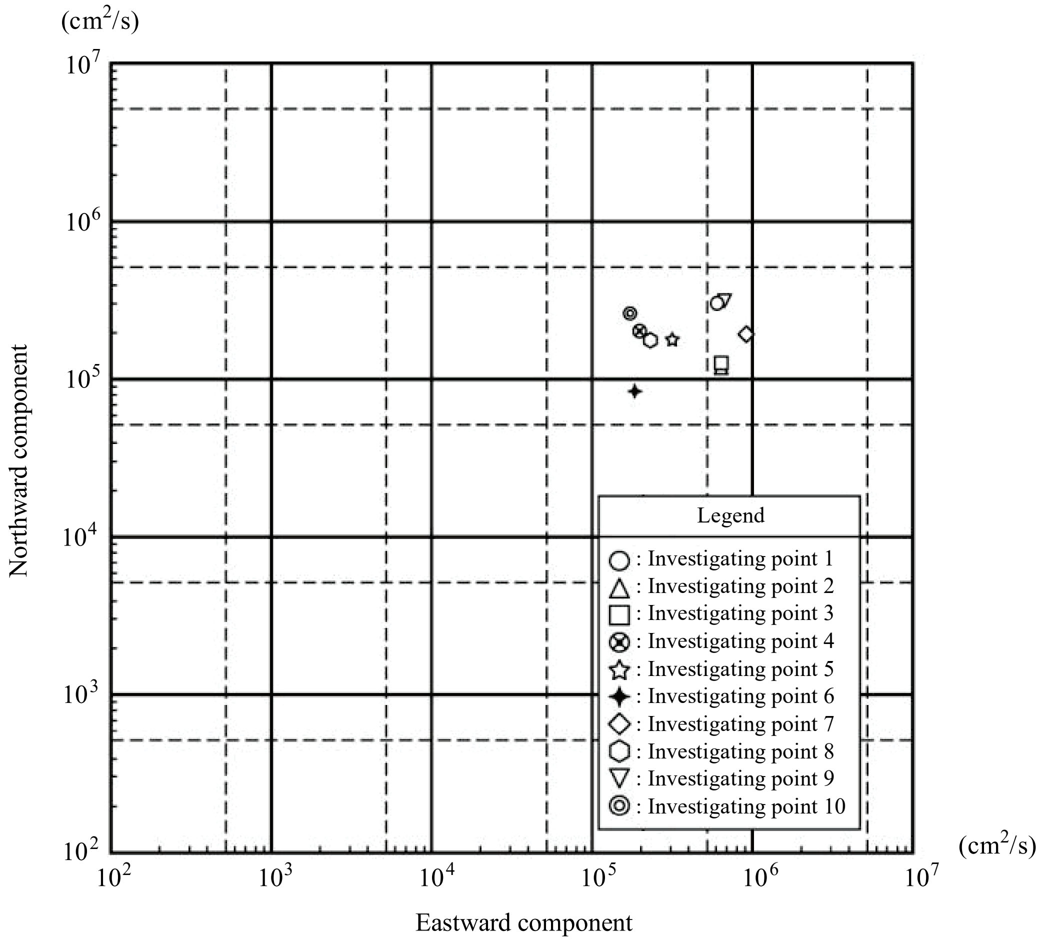


Fig. 2.8.14 Horizontal Diffusion Coefficient

(2) Sailing Observation Data

The sailing observation obtains the position data from GNSS and multilayer flow direction and velocity data from a flow velocimeter. The horizontal distribution of the flow velocity shall generally be created by plotting the flow direction and velocity data, which varies hour to hour based on time, on a map based on the position information by GNSS. When the interval between the start and the end of investigation becomes long, it is necessary to ensure a time change in a flow is included. The observation result is as shown in **Fig. 2.8.6** above.

(3) Examination of the Reproducibility of Numerical Simulation with the Observation Data

The tidal current observation data is used to examine the reproducibility of the tidal current field in a numerical simulation. That is, for the validity of a tidal current component, the actual measured tidal current ellipse (e. g., M_2 tide) and the calculated similar tidal current ellipse at the calculation lattice in the same place shall be compared, and the shape's validity shall be examined. Similarly, the mean flows on the calculation lattice shall be compared using the observed mean flow (mean flow in a period), and the validity of each flow field of the numerical simulation shall be checked.

2.8.6 Comparison of the Flow Velocimeters Based on the Example of Tidal Current Observation

As a helpful example of tidal current observation, an examination of comparison of flow velocimeters in the "Study committee regarding comparison of flow velocimeters," established in the Japan Marine Surveys Association, is described below.[53)]

(1) Field Measurement

The experimental sea areas were near the Chiba Prefectural oceanographical phenomena observation tower at the closed-off section of Tokyo Bay and near the Kanagawa Prefectural Hiratsuka observation tower on the open sea coast. The comparison tests were conducted in the sea area near the Chiba Prefectural oceanographical phenomena observation tower in February 2002 and in the sea area near the Hiratsuka observation tower in July of the same year.

The flow velocimeters used for comparison are an impeller type flow velocimeter (PU-2), an electromagnetic flow velocimeter (C-EM), two sorts of point type ultrasonic flow velocimeters (RCM11, 3DACM), and two sorts of ultrasonic type flow velocity profilers (ADCP, AWAC).

They were installed with the surface layer buoy hanging method with two-point anchors except for the ultrasonic type flow velocity profiler which was fixed to the mount on the sea bed (**Fig. 2.8.15**).

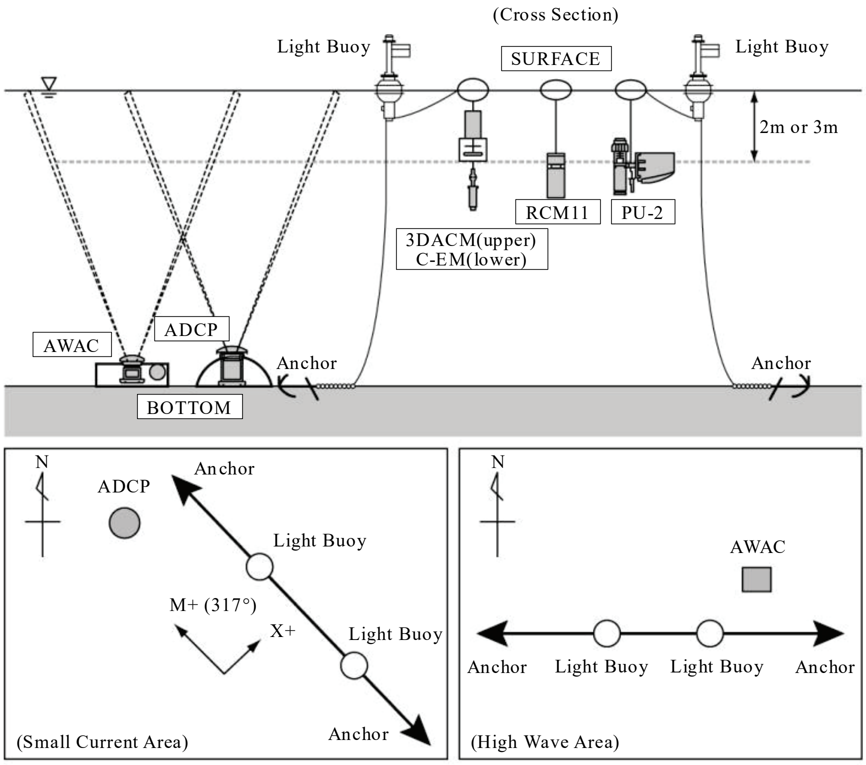


Fig. 2.8.15 Installation Method of Flow Velocimeters

(2) Observation Result

The outliers of the flow velocity values measured with various flow velocimeters were not processed by setting up a threshold value, but by using the measured raw data. To compare the flow velocity values of these various flow velocimeters, the flow velocity data of all models in each observation time (every 10 minutes) was averaged to make a temporary standard flow velocity. In the Chiba Prefectural oceanographical phenomena observation tower, the mainstream direction (317°), in consideration of the shore line at the investigation site and its rectangular direction, were compared. In the Hiratsuka observation tower, the east-west component, which is almost the shore line direction of the mainstream direction, and the south-north component as its rectangular direction were compared for the deviation component from the above-mentioned standard flow velocity.

In addition to these component flow velocity deviations, the flow velocity deviation based on the equation below, i.e., the deviation from the mean value (the standard flow velocity) of the scalar flow velocity value was also calculated and compared.

$$Di = \sqrt{Ui^2 + Vi^2} - \sqrt{\overline{Ui}^2 + \overline{Vi}^2} \qquad \textbf{(2.8.3)}$$

where

D_i : flow velocity deviation of various flow velocimeters

U_i, V_i : flow velocity in the mainstream direction and its rectangular direction

$\overline{U}_i, \overline{V}_i$: standard flow velocity value for the mainstream direction and its rectangular direction

① **Comparison result at the Chiba Prefectural meteorological tower (weak flow area)**

A about 70 cm-high significant wave (period: about 3 seconds), the highest in the observation period, was observed on February 19, 2002. Including this period, southeasterly flows were prominent on February 15 and 19, when significant waves of 50 cm or higher were seen. The weak periodic fluctuation of about 10 cm/s was dominant for the flow condition in other periods. No clear correlation is recognized for any flow velocimeter between the flow velocity deviation by each flow velocimeter and the change with time of wave height (**Fig. 2.8.16**).

Although the flow velocity deviation histograms of various flow velocimeters mostly indicated normal distribution, ADCP installed on a bottom tends to have larger flow velocity deviation than other ultrasonic models, and the impeller type flow velocimeter has the largest flow velocity deviation (**Fig. 2.8.17**).

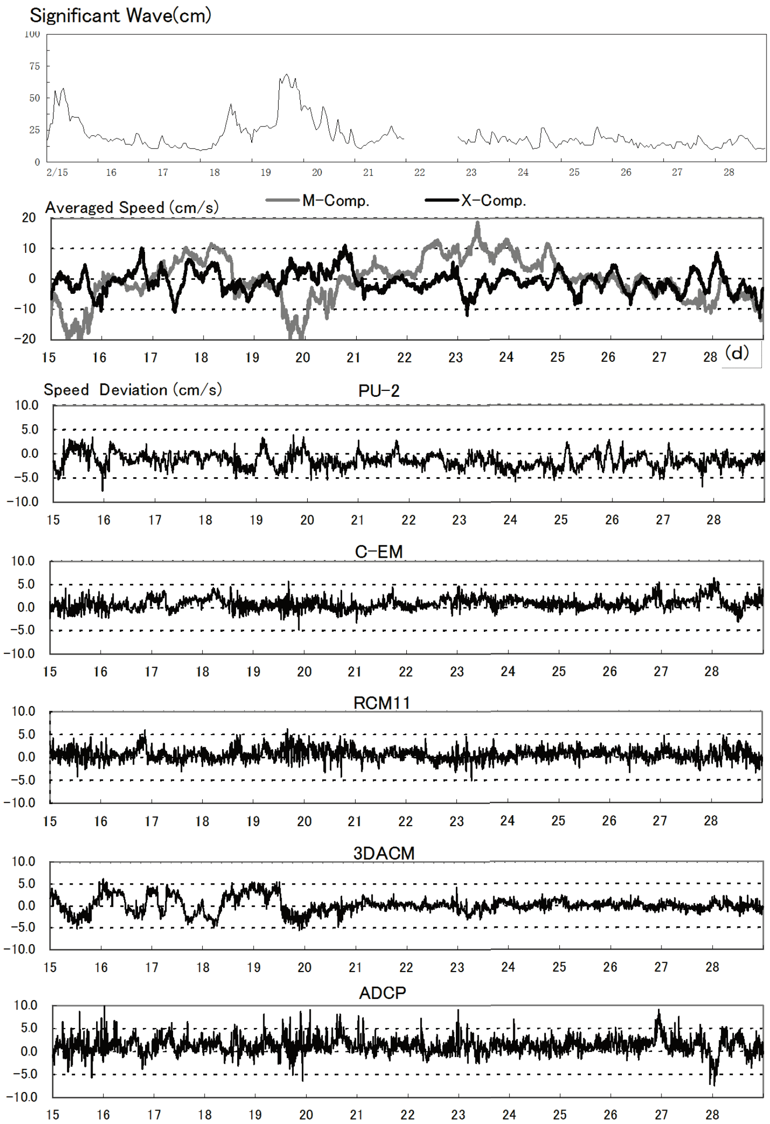


Fig. 2.8.16 Change with time of the Significant Wave Height, Standard Flow Velocity and Scalar Flow Velocity Deviation (Sea Area near the Chiba Meteorological Tower)

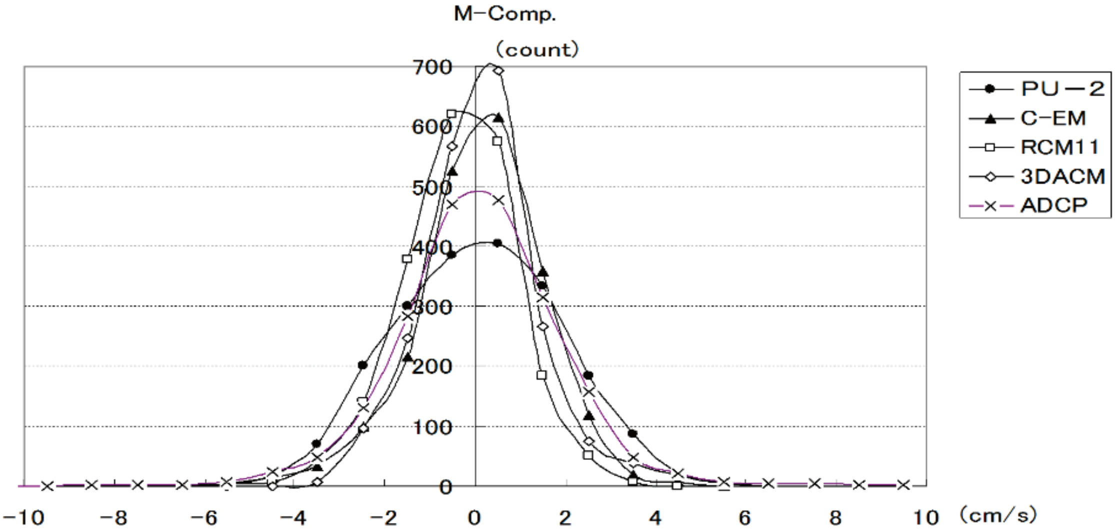


Fig. 2.8.17 Histogram of the Flow Velocity Deviation in the Mainstream Direction (Sea Area near the Chiba Meteorological Tower)

As another analysis item, the flow direction frequency by the orientation indicates the dominant distribution from almost northwest to a southeast direction. The flow direction frequency is similar, but the orientations of the most frequent flow velocity frequency in the mainstream direction differ with each flow velocimeter, and are far from the same. As for the flow velocity frequency, although only the impeller type flow velocimeter shows a peak in the weak flow velocity of 2 cm/s or less, the shapes of flow velocity frequency distribution of other electromagnetic flow velocimeter and ultrasonic flow velocimeter are almost the same.

As for the periodicity of the flow, although a peak is detected in every flow velocimeter result over a 12-hour period in the east-west and north-south component, ADCP shows higher energy density on high frequency side compared to other flow velocimeters.

The flow velocity value in the long axis direction of the M_2 tidal constituent flow of each flow velocimeter shows a weak flow of less than 3 cm/s, and the mean flow velocity in a period is less than 2 cm/s. The flow velocity of an impeller type flow velocimeter is smallest, but there is no clear difference in the flow direction by the flow velocimeter.

The orders of diffusion coefficients of the raw data and the data excluding periodic components of 12 hours or more are identical between every flow velocimeters.

② **Comparison result at the Hiratsuka observation tower (open sea wave region)**

During the period from July 23 to 25, 2002, Typhoon No. 9 passed the southern area of Sagami Bay westward. July 24 saw high significant waves of max. 3 m or more (period: about 15 seconds). In another period, the southern wind also dominated, due to the summer type pressure distribution, with waves as high as 1 m (period: about 4 seconds). An overview of the time series of mean flow velocity (standard flow velocity) shows that the eastward or westward flow parallel to the shore repeated alternatingly, and the flow on the 24th, when the typhoon passed, is small as a whole. The change with time of the flow velocity deviation from the standard flow velocity of each flow velocimeter shows that the flow velocity deviation width is generally larger than that of the Chiba Prefectural oceanographical phenomena observation tower, but no correlation is found between the wave height change and the flow velocity deviation (**Fig. 2.8.18**). Moreover, unlike other flow velocimeters, the flow velocity deviation histogram of the impeller type flow velocimeter does not have a normal distribution (**Fig. 2.8.19**).

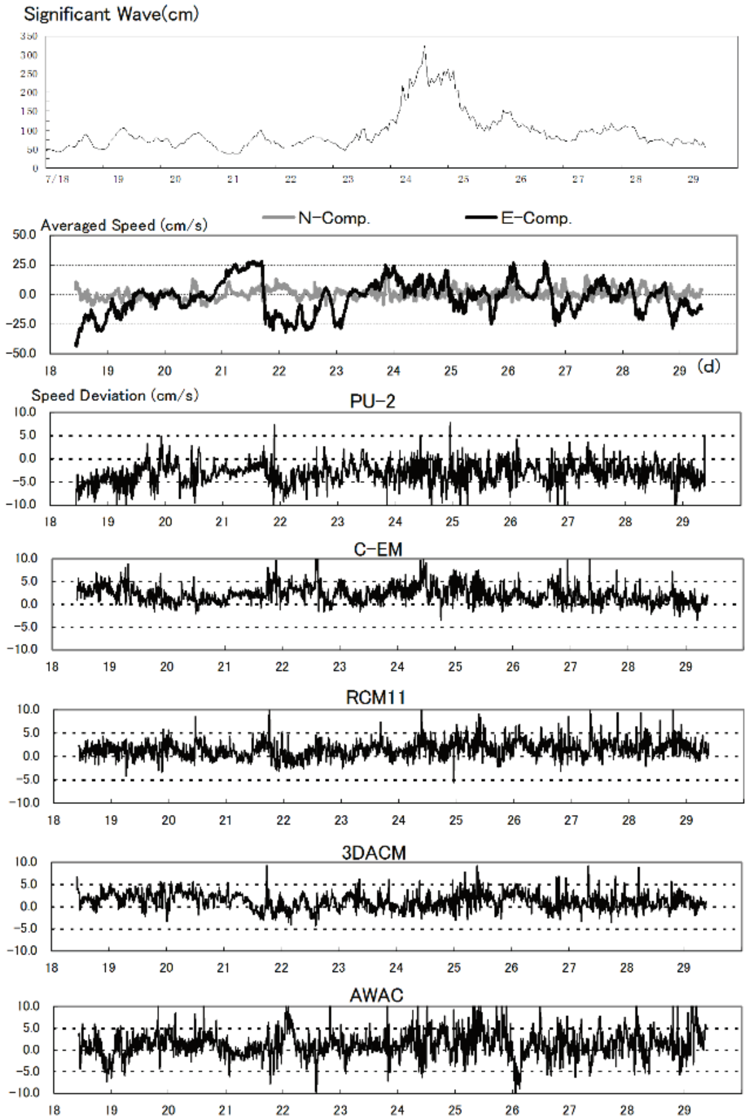


Fig. 2.8.18 Change with time of the Significant Wave Height, the Standard Flow Velocity and the Scalar Flow Velocity Deviation (Sea Area near the Hiratsuka Observation Tower)

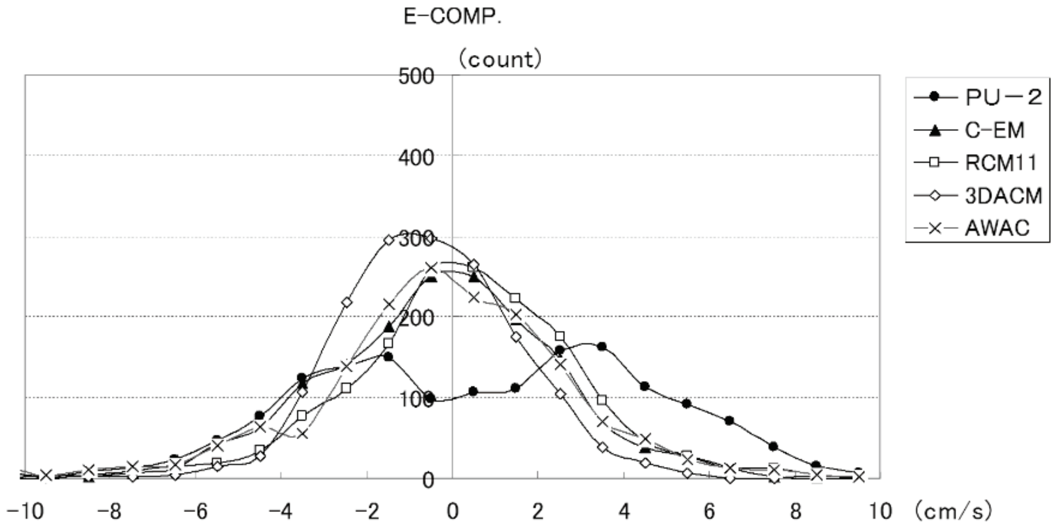


Fig. 2.8.19 Histogram of the Flow Velocity Deviation in the East-West Direction (Sea Area near the Hiratsuka Observation Tower)

As another analysis item, the flow direction frequency by the orientation indicates a clear dominance in east-west direction, with almost the same shape for every flow velocimeter. The shapes of the flow velocity frequency distribution of electromagnetic flow velocimeters and each ultrasonic flow velocimeter are almost identical except that the flow velocity frequency distribution of the impeller type flow velocimeter has a peak at the weak flow velocity side.

As for the periodicity of the flow, the spectrum of every flow velocimeter shows a peak at a period of about 12 hours in the east-west component, and the shape of the spectrum is almost the same. As for the long axis flow velocity value of four principal tidal constituents for each flow velocimeter, the values of the impeller type flow velocimeter are smaller than those of other flow velocimeters.

Both diffusion coefficients of the raw data and the data excluding periodic components of 12 hours or longer obtained from the impeller type flow velocimeter are smaller than those obtained from other flow velocimeter data, and the north-south component of raw data was smaller by one order.

③ **Comparison between the surface layer data hung and moored and installed on the sea bed**

The flow velocity in the surface layer is generally measured by the method of hanging from the moored buoy or installing the ultrasonic type flow velocity profiler on the sea bed.

This comparison result shows that the data of the ultrasonic type flow velocity profiler installed on the sea bed has a larger deviation compared to the various flow velocimeters except the hung and moored impeller type flow velocimeter.

The factors causing the deviation in the ultrasonic type flow velocity profiler are that the S/N ratio worsens as the distance from the sensor increases and lowers the distance resolution (**Fig. 2.8.20**).

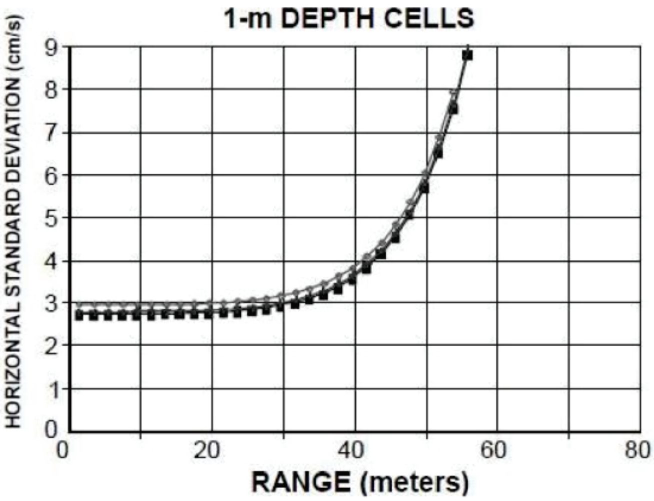


Fig. 2.8.20 Theoretical Value of WH-ADCP (600 kHz) Standard Deviation Error (ADCP Technical Manual)

That is, the error resulting from the lowered distance resolution by the spreading distance between beams near the surface layer far from the sensor is considered to affect it, since the S/N ratio near the surface layer becomes worst when installed on the sea bed, where the measuring beam is obliquely beamed (20°).

On the other hand, it seems that the point type flow velocimeter hung and moored by a two-point anchor method generates a measurement error by mooring system vibration, waves, and so on. However, the influence is considered mitigated by setting a measurement interval shorter than the wave period and by averaging the measurement data for a longer time than the wave period (for example, 1 minute) (**Fig. 2.8.21**).

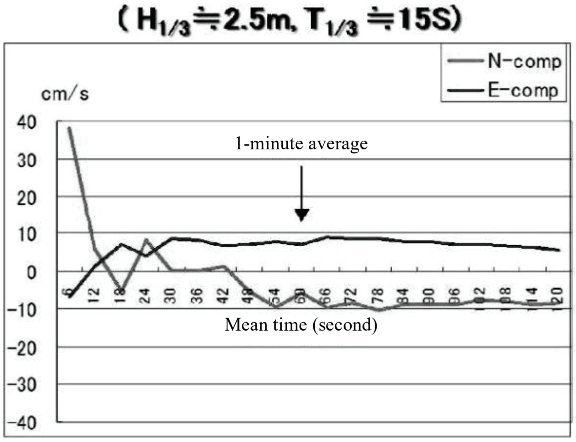


Fig. 2.8.21 Averaging the Electromagnetic Flow Velocimeter Data (Hiratsuka Observation Tower Data)

This result also shows that the electromagnetic flow velocimeter hung and moored to the surface layer and 3DACM acquire data with smaller deviation than the ultrasonic type flow velocity profiler installed on the sea bed.

In light of the foregoing, a flow velocity measurement that has a smaller deviation than the ultrasonic type flow velocity profiler installed on the sea bed is possible, even from the flow velocity data measured with the point type flow velocimeter hung and moored to the surface layer by a two-point anchor method.

(3) Points to Note When Comparing Flow Velocimeters, etc.

The summary of the observation and study done in this real sea area found the following noteworthy points on how to select and install flow velocimeters.

Table 2.8.3 Points to Note on How to Select and Install Flow Velocimeters

Challenge	Points to note
How to install in the flow velocity measurement targeting the tidal current phenomenon and others in the surface layer	The hung and moored point type flow velocimeter obtains smaller deviation data than the ultrasonic type flow velocity profiler installed on the sea bed.
Comparison of the electromagnetic flow velocimeter and various ultrasonic flow velocimeters	In observation of a weak flow area and at the time of high waves, the flow velocity and other general flow condition analysis items generally coincide.

2.9 Observation and Examination Concerning Littoral Drift[66) 67) 68) 69) 70) 89)]

2.9.1 Overview

(1) Overview of the Littoral Drift Phenomenon

The sediment which moves in the sea by wave or flow is called littoral drift. Littoral drift moves in the range from the threshold depth of sediment movement to the highest point reached by wave run-up. The sediment which moves in the sea contains silt or clay with a grain size of 0.074 mm or less, sand with a grain size of 0.074 to 2 mm, gravel with a grain size of 2 mm or more, and sediment called stone. Littoral drift usually refers to sand in many cases. The primary target of sediment described below is considered to be sand. **Fig. 2.9.1** schematically presents the change in the movement form of sand from the offing to the direction of the shore.[71)]

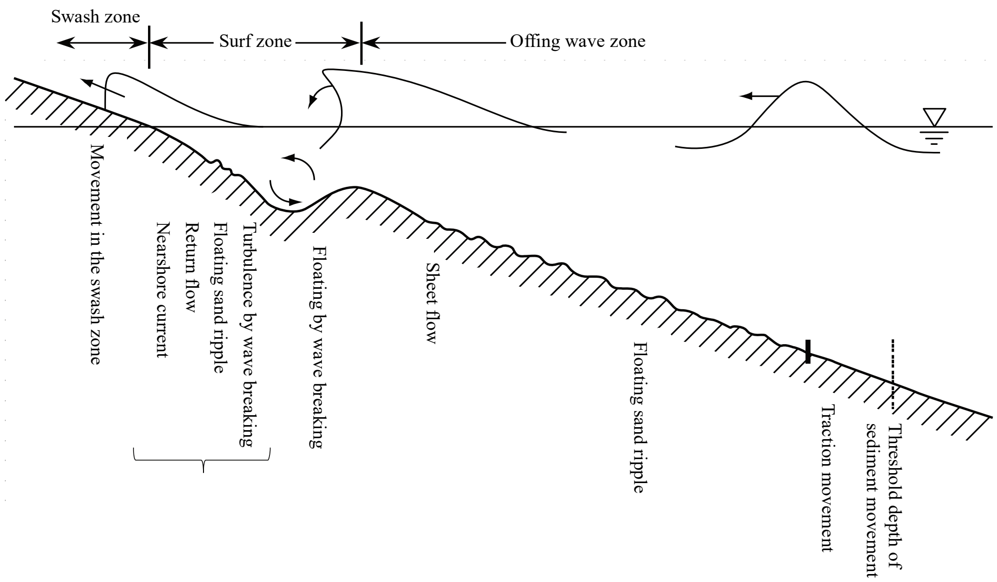


Fig. 2.9.1 Movement Form of Littoral Drift (Excerpted from Shore Environmental Engineering (supervisory editor: Hitoshi Homma, editor: Seiji Horikawa), p. 146, 1985)

(2) Purposes of Littoral Drift Examination

The purposes of littoral drift examination are to solve the problems caused by topographic changes due to sediment movement in the coastal area, to identify countermeasure works, to predict possible future topographic changes brought about by artificial influence or change in natural environment, and to propose countermeasures if needed. The target phenomena are shore erosion, siltation of navigation channel and basin, local scouring near the structures, etc. For the outline of each phenomenon, see **Part II, Chapter 2, 7.4 Littoral Drift**, **7.5 Scouring and Suction** and **Part III, Chapter 4, 18 Facilities for Siltation Prevention Countermeasures**.

2.9.2 System and Flowchart of Examination

Littoral drift is examined by collecting information, understanding the present condition, and proposing countermeasures (refer to the flowchart shown in **Fig. 2.9.2**).

(1) Collection of Information

Past and existing data and documents are collected, including past/existing examination report, aerial photographs, beach topographic maps, sound maps, satellite data, and bottom sediment data, as topographic information. In addition, data concerning the weather, oceanographic phenomena, and rivers shall be collected because the littoral drift phenomenon is influenced by waves, wind, flow, precipitation, and outflow sediments from rivers. Furthermore, construction records of shore and port structures, and if needed, work records of dams and records on river improvement and on the amount of accumulated gravel, shall also be collected.

Simplified examination (field exploration) in the field and interviews with persons concerned shall be conducted as needed. Even if field exploration is not necessary, it is desirable for the investigator to visit the field at least once.

Collected documents and data (including the additional data that will be mentioned later) are very valuable; hence, they should be managed properly and stored collectively to avoid scattering and loss.

(2) Understanding of the Present Condition

Besides understanding the characteristics of topographic change in the target shore through analysis of the collected data, the cause of topographic change (mechanism of the littoral drift) should also be determined. If the collected data is insufficient, then additional data shall be obtained by conducting new field measurements by topography and bathymetric surveys, examination of bottom sediment, observation of waves and flow condition, observation of the concentration and amount of suspended sediment, observation of tracer (fluorescent sand), observation of the amount of wind-blown sand, etc. Details are described in **2.9.3**.

Although reasonably valuable data can be obtained from a short-term observation, which is about two months, it may be impossible to achieve the purpose of the observation if the weather and oceanographic phenomena at the time of observation do not coincide with the natural characteristics of the target shore. For example, in a two-month observation of the shore which is affected by southerly littoral drift, if waves from the north dominate during the observation period, sufficient data might not be obtained in the field measurement. Therefore, it is preferable to conduct a short-term observation with a strict plan if some questions still remain after the analysis of the collected data.

Numerical simulation or model experiment may be done to confirm the solved mechanism of littoral drift. With the recent development of computers, numerical simulation is conducted in many cases. Numerical models currently used for this purpose are the shoreline modification model and the three-dimensional beach modification model.[72), 73), 74)]

The shoreline modification model predicts a long-term change in the shoreline by considering only the longshore sediment transport rate. In this model, the shoreline moves by the income and outgo of the longshore sediment transport rate caused by the coastal direction component of the incident wave energy flux. The shoreline direction changes and finally settles down so that waves eventually enter at right angle to the shoreline. This model is often used, and the prediction result is sufficiently practical, but in order to increase the accuracy of the prediction, sufficient accumulation of observation data and recurrence calculation using the data are paramount.

In contrast to the shoreline modification model, which treats the phenomenon in a surf zone as a black box, the three-dimensional beach modification model formulates and incorporates the phenomenon in a surf zone. This model calculates the wave field and the nearshore current field and then predicts the change in water depth in the calculation area. The phenomena in a surf zone have not yet been solved fully, and there is also a possibility of using wrong results if various assumptions in the modeling process are not fully understood.

Although numerical simulations and model experiments to understand the present condition are primarily intended to confirm the resolved mechanism of littoral drift, two or more countermeasure works are often assessed by numerical simulation or model experiment in the proposal of countermeasure works that will be mentioned later. Since numerical simulation about littoral drift simplifies and models the natural phenomenon, it includes coefficients that are experientially determined. On the other hand, no perfect similarity rule has been established for model experiment about littoral drift. Therefore, when conducting numerical simulation or model experiment in the examination of countermeasure works, it is necessary to confirm that they have reproduced changes in the local topography in sufficient accuracy at the stage of understanding the present condition.

(3) Proposal of Countermeasure Works

Countermeasure works shall be proposed taking into consideration the clarified topographic change characteristics and the mechanism of littoral drift. In the proposal of countermeasure works, the evaluation result of the effect of two or more planned countermeasure works by the numerical simulation model is often utilized.

Collection of information
- Collection of past/existing data and documents
- Performance of exploration examination and interviews, if needed

↓

Understanding of the present condition (understanding of the topographic change characteristics and resolution of the mechanism of littoral drift)
- Collection of additional data by field measurement
- Analysis of data
- Numerical simulation or model experiment, if needed

↓

Proposal of countermeasure works
- Performance of numerical simulation or model experiment

Fig. 2.9.2 Flowchart of Littoral Drift Examination

2.9.3 Observation Method

(1) Field Shore Exploration

Field shore exploration is an important examination to collect information for the validation of the established examination plan, i.e., determination of the examination range, examination items, timing and frequency of examination, observation equipment to be used, and arrangement of the equipment.

① **Items to check in the field exploration**

The general items to check during the exploration are as follows[75] ((a) through (e) shall be confirmed in the field referring to the outline estimated from maps and aerial photographs):

(a) Approach path for measuring equipment to be carried in and for examination staff to come in and out

(b) Reservation of the workspace for observation preparation

(c) Height of the hinterland for shore observation or the existence of tall buildings

(d) Existence of structures available for observation

(e) Whether temporary material storage and parking space can be reserved

(f) Whether commercial power can be used

(g) Contact with the field coast management bodies, fishermen’s cooperative association, Japan Coast Guard, etc.

② **Items to examine in the field shore**

The general items to check in the field shore are as follows:

(a) Shore observation from a height

(b) Beach width, foreshore slope, backshore slope

(c) Bottom sediment form and grain size distribution

(d) Planar shoreline form, topography near structures, existence and form of a river mouth bar

(e) Aerial photography taken by a drone

(f) Interviews with area residents

③ **Measurement equipment used for field exploration**

The measurement equipment used for field exploration is a conventional simple device as follows. Although this equipment is enough for the exploration, the use of optical simplified rangefinder, macrometer using laser beams, simplified position-measuring device using the Global Navigation Satellite System (GNSS), and others may be considered.

(a) Maps (the latest beach topographic map, sound map, and nautical chart of the target region)

(b) Expected tide level diagram of the day

(c) Compass

(d) Camera (a camera capable of identifying the filming location with the GNSS function)

(e) Hand level, tape measure, staff, clinometer, etc.

(f) Others (writing instrument, bottom sediment collection shovel, preservation bag, etc.)

④ **Examination method in field exploration**

(a) Observation of the target shore from a height[76) 77)]

If there are cliffs, buildings, towers, observation platforms, etc., near the beach, observe the shore from there so the width of the shore and the condition of the shoreline can be clearly seen. Depending on the wave condition, the position of the river mouth bar and the location where the rip current occurs may be observable. If the wave period is long, there is a possibility that a diffraction phenomenon may occur, and the outline of the seabed topography can be estimated.

(b) Two-dimensional shoreline condition (roughness of the shoreline)

The characteristics of two-dimensional shoreline form, shoreline form and topography near the river mouth and shore structures (jetty, detached breakwater, etc.) shall be observed, measured, and recorded by sketch, photography, and, if needed, by simple survey (GNSS survey). Existence check shall be done and the positions of the river mouth bar, reef, and others recorded.

(c) Beach width, slope of foreshore and outer beach

A pole, tape measure, hand level, and clinometer are enough to measure a slope. However, a simplified measurement using the GNSS function may be performed. The survey line interval is normally about 500 to 1000 m along the shoreline. The survey line shall be perpendicular to the shoreline. The elevation of the land shall be measured on the survey line in about 5 to 10 m interval, including sudden changing point of a slope and others. If using a survey instrument using GNSS and no referring point on the ground can be used, an error of several meters in position and several tens of centimeters in elevation of land may be observed. It is necessary to check in advance if it is used. Slopes shall be measured from a mean water level surface (shoreline) to the land side edge of the backshore, seawall, or root of a dune.

(d) Bottom sediment form and grain size

The bottom sediment shall be collected near the shoreline, with a collection interval of about 500 to 1000 m, depending on the characteristics of the shore. The bottom sediment shall be collected in a large interval on the shore with a monotonous shoreline form. In an inflectional shoreline, the bottom sediment shall be collected at essential points of the inflection. The collection points shall be properly selected considering the topographic form near the structures.

(e) Aerial photography by UAV

Aerial photography enables easy understanding of the condition of the beach in the target shore. Using an unmanned aerial vehicle (UAV) is recommended for the exploration. Whether a UAV is used or not shall be determined by considering the observation purpose and cost.

(f) Interviews with residents

Knowledge on the following items and so on shall be asked from the fishermen and residents living in the target examination area. It is preferable to have interviews with as many people as possible, without sticking to the statement of specific persons.

1) Features such as waves, longshore currents, and weather by four seasons and months

2) Change in shoreline (advance, retreat)

3) Change in the river mouth bar by seasons

⑤ **Arrangement of data**

An exploration plan shall be prepared, and photographs shall be digitalized based on the data obtained from the exploration.

The exploration date, bottom sediment collection points, beach slope measurement points (sketch of a slope), condition of the shoreline, sites where photos are taken, grain size, condition of the structures, and others shall be entered in the plan to have a better understanding of the present condition of the shore.

Photographs taken shall be arranged and digitalized. The use of photograph management software is recommended.

(2) External Force Observation

The following items shall be observed as external force factors causing littoral drift movement. If wave and flow conditions are observed relating to the littoral drift examination, they may be observed for a short period at the time of severe weather. The observation equipment to be used and how to install the equipment and arrange the data should be taken into consideration.

① Weather survey (mainly wind; temperature, precipitation, and others in some cases)

② Wave observation (wave height, period, wave direction, rise of mean water level near the shoreline due to waves, run-up of waves, long-period wave)

③ Flow condition observation (nearshore current (longshore currents, rip current), tidal current, ocean current: distribution in time and space of the flow velocity and direction)

④ Tide level observation

(3) Beach Topographic Survey and Bathymetric Survey

The bathymetric and topographic surveys of the beach portion shall be conducted to estimate the trend and amount of littoral drift from the topographic change. The range of beach topography and bathymetric surveys is determined in the examination plan. Frequency shall be based on **Table 2.9.1**.

Table 2.9.1 Range and Frequency of Beach Topographic and Bathymetric Surveys

Items \ Classification	Broad survey (The nautical chart shall be used to know the topography in high accuracy.)	Survey for change in topography near the structures	Shoreline survey	Remarks
Range in the direction of the coast	A vast area extending from the supply source of the littoral drift to where it outflows	b + 2l (The outside of the range is complemented by the shoreline survey.)	Covers the range of broad survey in principle.	b: coastal distance of a structure
Range in the direction of offing	From the offshore edge of a structure to be fixed within five years from now to 4 or 5 wavelengths to offing	l + (2 or 3 wavelengths)	From the shoreline at low tide to the back end of the backshore	l: onshore–offshore distance to the tip of a structure
Survey line, survey point interval	Survey line interval: 200 m or less Survey point interval: 50 m or less Surf zone depth or shallower: 5 m or less	Survey line interval: 100 m or less 10 to 20 m is preferable in the extreme vicinity of structures.	Survey line interval: 100 m or less 10 to 20 m is preferable at points where topography is complicated.	

Items \ Classification		Broad survey (The nautical chart shall be used to know the topography in high accuracy.)	Survey for change in topography near the structures	Shoreline survey	Remarks
Survey frequency	Planning	Once in the early stage of the planning	Substituted by the broad survey.	Done in the broad survey.	
	Design	Once if no survey was done in the planning	Twice a year	Same as above	
	Construction	Once in five years	Same as above	One time each after the calm standard severe-weather term A total of two times	
	Administration	Same as above	Once a year	Once after the severe-weather term	

Excerpted from *Port Examination Guideline (Revised edition)* editorially supervised by Ports and Harbours Bureau of the Ministry of Transport, published by the Ports and Harbours Association of Japan, pp. 1–202, 1987

① Estimation of the trend and amount of littoral drift by using the beach topographic map and sound map

Although recently surveyed topographic maps and sound maps are computerized as three-dimensional numeric data, only the maps of the survey result are left for previous topographic maps and sound maps in many cases. In such a case, the topographic maps and sound maps shall be converted into numeric data. Here, several topographic maps and sound maps surveyed at different times and scaled at the same scale are assumed to exist.

When examining the topographic change characteristics and mechanism of the littoral drift from the topographic maps and sound maps, the three kinds of maps shown below shall be created, or statistical analysis[89] shall be conducted, and they shall be comprehensively judged based on their results. The amount of littoral drift is classified into the gross amount and the net amount (precise amount) of the littoral drift. When considering the sediment movement to a certain direction, the amount of the littoral drift to the positive direction and the negative direction are called the gross amount of the littoral drift in the positive direction and the negative direction, respectively. The difference is called the net amount of the littoral drift. The topographic change and the net amount of the littoral drift are proportional.

(a) Secular change map of the shoreline

If the shoreline positions are plotted on the same plane from among the beach topography and bathymetric survey data obtained at different times, the shoreline change in the period can be seen. If several pieces of survey data are plotted, shoreline changes in such periods can be seen. The cross section of the shoreline can be assumed to be accumulative if the shoreline moves forward to the sea side, erosive if retreats to the land side, and in an equilibrium situation if it moves a little.

(b) Sectional change map in the direction of onshore–offshore

This is a map plotting the vertical section perpendicular to a shoreline at a certain survey point. If survey data obtained at several times are plotted, the condition of the topographic change (deposition, erosion) of the section during the term can be seen. If topography and bathymetric surveys are conducted several times in a short period of time (e.g., a year), seasonal seabed change in the onshore–offshore direction can be observed.

(c) Topography (seabed foundation surface) change map

A map on which isopleth lines are drawn by determining the difference in the water depth at the same points in the whole survey area of two topography and bathymetric survey data obtained at different times and plotting these values on the plane is called the topography (seabed foundation surface) change map. If the water depth is indicated by (–), the before and after values are represented by {(–)–(–)}. The + value indicates deposition, whereas the – value indicates erosion. The condition of the deposition and erosion in the whole survey area can be observed.

② **Measurement of the amount of sea bottom change at the time of severe weather**

It is difficult to conduct a bathymetric survey at the time of severe weather. Although a sea bottom variation gauge[78] which can continuously measure the water depth using the ultrasonic wave had been developed, there is a problem with regard to the measurement accuracy when the littoral drift moves. Thus, observation of the seabed change at the time of severe weather is impossible. However, when local topographic change analysis is necessary, the deepest foundation height with littoral drift movement at the time of specific severe weather can be observed by using the scouring ring.[79] The use of the scouring ring together with the sea bottom variation gauge shall be considered.

(4) Bottom Sediment Examination

The bottom sediment examination for littoral drift examination is conducted for the following purposes:

(a) Resolution of the bottom sediment characteristics of the target sea area

(b) Estimation of the moving direction of the littoral drift

(c) Examination of the relevance of the bottom sediment characteristics to oceanographic phenomena (mainly waves, flow condition)

(d) Resolution of the narrow-range (near structures) bottom sediment characteristics

(e) Resolution of the characteristics of the river sediment load considered as the source of littoral drift and estimation of the outflow range.

① **Setting of the examination range and period**

The examination range shall be set according to the purpose. It is preferable to have at least three data collection points onshore including the root of a dune (land shore edge of the backshore), critical wave run-up point, and the shoreline position at the time of the mean tide level. On the seabed, every 2 to 5 m of water depth is preferable. The examination period shall be determined considering the seasonal variation of natural conditions. If data on the bottom sediment is already available and when investigating supplementarily,??? the examination site and time shall be determined according to the purpose.

② **Main examination equipment to be used**

The examination equipment and others used for bottom sediment examination are shown in **Table 2.9.2**.

Table 2.9.2 Examination Equipment and Others Used for Bottom Sediment Examination

Classification		Examination equipment	
Equipment for bottom sediment collection	On land	Collection equipment	- Shovel
		Measurement of examination points	- Measuring tape, specially designed surveying equipment at the collection point (using the GNSS function)
		Others	- Sample preservation bag, writing instrument, field notebook
	In the sea	Collection equipment	- Bottom sediment collection device (properly select according to the purpose from the glove type bottom sampler, columnar bottom sampler, and dredger)
		Measurement of examination points	- Ship position-measuring device (using the GNSS function), sounding equipment
		Others	- Sample preservation bag, writing instrument, field notebook
Grading analyzer	Sieve analysis	Sieve, drier, precision balance, balance, sieve shaker (vibrator type, electric type)	
	Dispersing device analysis	Dispersing device, hydrometer, thermometer	
	Analysis using the Emery tube	Emery tube, thermometer, stopwatch, scripter	
Equipment for specific gravity analysis		Drier, precision balance, pycnometer, water thermometer	
Equipment for ignition loss analysis		Drier, precision balance, evaporating dish, crucible, sieve (420 μ)	

③ Examination method

(a) Collection method

The bottom sediment onshore shall be collected after removing surface layer dust, etc. The grain size distribution on the surface layer near the shoreline may significantly differ when 20–30 m apart due to the wave's sieving action. When the shoreline varies much or has a cusp shape, it is necessary to choose a collection point considering the fine topographic characteristics of a sandy beach. When determining the grain size distribution in the direction of the coast, collect the bottom sediment by unifying the collection points to the same elevation of land or collect the bottom sediment from points having the same beach vertical section form.

The bottom sediment on the seabed shall be collected from onboard using the bottom sediment collector. The amount to be collected is about 400 g but shall be increased to the amount required for the other planned analyses (specific gravity, silt and clay whole amount measurement, measurement of shell weight). The collected sample shall be stored in a sample preservation bag. The sample number, time and day of collection, and other necessary information shall be written on the preservation bag. Simultaneously, a sample control chart, which is presented in **Table 2.9.3**, shall be created and recorded.

Table 2.9.3 Field Notebook for Bottom Sediment Collection Record

Item / Survey line number	Collection number	Water depth	Collection time	Remarks

(Excerpted from Port Examination Guideline (Revised edition) editorially supervised by Ports and Harbours Bureau of the Ministry of Transport, published by the Ports and Harbours Association of Japan, 1987)

(b) Determination of the collection point

The bottom sediment collection point shall be measured using a surveying equipment. The simplified surveying equipment using the GNSS function is enough for the measurement. Planar errors of about several tens of centimeters are not considered problems. For onshore collection, the collection point may be measured before or after the collection. At sea (in the case of the collection of seabed bottom sediment), the ship's position and water depth shall be measured simultaneously at the collection time.

(c) Analysis of a sample

The following analysis shall be conducted according to the purpose by using the collected sample. Grading analysis shall conform to the **Japan Industrial Standards (JIS 1102)** and the **Japanese Geotechnical Society (JGS 0131)**.

1) Grading analysis
2) Specific gravity measurement
3) Other tests: ignition loss analysis, measurement of the content of heavy minerals, measurement of the content of feebly (weakly) magnetic minerals, mineral analysis, silt and clay whole amount measurement examination, measurement test of shell weight

(d) Arrangement of data

The required chart shall be created based on the analysis of the collected samples. Simultaneously, the information about all the collected samples shall be summarized (create a bottom sediment examination record note). Items shown below shall be at least required. All data shall be digitized in a database for later use.

1) Creation of the grain size accumulation curve

Median diameter, sorting coefficient, degree of distortion, and others shall be determined.

2) Creation of a bottom sediment distribution map

A bottom sediment distribution map shall be created on the sound map closest to the time of collection, in which the median diameter, sorting coefficient, degree of distortion, and other required items obtained from the examination analysis are entered. When input items become too many to distinguish, items to enter in the map shall be selected. Simultaneously, distribution maps of the onshore–offshore directional section and in the coastal direction shall be created.

3) Creation of a bottom sediment examination record note

Records of collection date, collection point, waves at the time of collection, information on flow, analysis method, analysis time and day, and others shall be summarized for all samples collected so that the examination results are smoothly utilized later on.

④ **Use of data**

The obtained bottom sediment distribution map and other data shall be used to estimate the amount and moving direction of the littoral drift together with the secular change map of shoreline, onshore–offshore directional section change map, and topography (seabed foundation surface) change map obtained from the topography and bathymetric surveys.

(5) Sand Catching Examination

This is an examination to directly measure the amount and moving direction of the littoral drift. The measurement methods are roughly classified into (a) the use of a sand catcher and (b) the electrical measurement of the amount of littoral drift (littoral drift measuring equipment). The outline of these measurement equipment and methods is described below. Either method has currently a limit in the accuracy of the measured value. It is necessary to use a method suitable for the purpose of the examination.

Measurement of the amount of littoral drift by sand catcher determines the concentration, assumes that the littoral drift moves at the same speed as the seawater, and calculates as the product of concentration and the flow velocity (concentration × flow velocity). Therefore, it is necessary to measure the flow velocity at the position of the sand catcher. Moreover, since the moving direction is determined by the installation direction of the sand catcher, it does not necessarily show the moving direction of the littoral drift at the sand catcher installation position.

① **The sand catching functional principle and the form of sand catcher**

The sand catcher is a device used to directly catch littoral drift, and there are various sand catchers according to the purpose. The features are classified in **Table 2.9.4**.

Table 2.9.4 Classification of the Features of Sand Catcher

Classification method	Classification of sand catcher
Object to be measured	① Suspended sediment only, ② bed load sediment only ③ Whole amount (suspended sediment + bed load sediment + sheet flow sand) Note: Observation of the bed load sediment is seldom conducted recently, since it does not necessarily give much amount of information
Area to be measured	① Inside the surf zone (including the swash zone), ② outside the surf zone
Principle to catch sand	① Suction type (water sampling), ② precipitation type and net type
Installation method of sand catcher	① Hang the sand catcher from a piled pier ② Fix to an underwater mount ③ Leave on the sea bottom surface (sink to the bottom type) ④ Embed under the sea bottom surface

② **Equipment used for the sand catching examination**

The equipment used for sand catching examination is outlined in **Table 2.9.5**.

Table 2.9.5 Outline of the Equipment Used for Sand Catching Examination

	Suspended sediment sand catcher		Littoral drift measuring equipment (suspended sediment measuring equipment)
Classification	Precipitation-type suspended sediment sand catcher Net-type suspended sediment sand catcher	Suction-type suspended sediment sand catcher Water sampling-type suspended sediment sand catcher	Optical suspended sediment densimeter Ultrasonic-type littoral drift measuring equipment Laser littoral drift measuring equipment (laser diffraction grain size and granularity distribution measuring equipment)
Outline of equipment	■ A hole punch pipe or box-type container is installed in the sea. This is a sand catcher to allow the littoral drift to flow in from the hole to settle in the container ■ The net type is a sand catcher to catch littoral drift by using nets (wire net, plankton net, etc.). Sand catchers using a bamboo pole or vinyl chloride pipe is generally used.	■ This is a sand catcher to sample a certain amount of seawater and measure littoral drift contained in the seawater ■ Seawater is sampled with the natural suction system using water pressure only or with the compulsive suction system using a pump.	■ This is the equipment developed to measure the amount of littoral drift in uninterrupted hours. ① Optical suspended sediment densimeter: a measuring device to electrically convert the rate at which the light volume penetrating underwater is intercepted by the sand particle and to measure the suspended sediment concentration quantitatively ② Ultrasonic-type littoral drift measuring equipment: a measuring device to measure the concentration and velocity of the suspended sediment by using the reflective characteristics and the amount of penetrated ultrasonic wave ③ Laser littoral drift measuring equipment[85]: a measuring device to measure the grain size and concentration of floating bottom sediment by using the diffraction phenomenon of laser beams
Points to note in the examination	■ The size of the sand-catching pipe shall have enough margin to the predicted amount of sand caught. A vinyl chloride pipe which is 5 to 7 cm wide and 30 to 40 cm long is generally used. ■ The size of the sand-catching holes shall be basically 5 cm long and 1 cm wide. They shall be made symmetrically. ■ The top float shall have buoyancy to keep the sand catcher perpendicular to the water surface and always be applied tension with a chain, etc. The chain and others shall be strong enough to bear waves and flows. ■ The lower anchor shall be heavy enough to stay against		■ As the output from every equipment varies according to the grain size and concentration of sand and the flow condition (flow velocity), it is difficult to calibrate and acquire the true value. ■ However, the change tendency in the amount of the littoral drift shows a qualitative rationality. Combined use with the sand catcher is recommended.

	waves and flows.		
Reference	Literature 80)	Literature 80)	Literatures 81), 82)

③ **Field examination**

(a) Examination period and duration

The period and duration of the field examination shall be determined considering the purpose, observation equipment to be used, all-year change in the oceanographic phenomena and weather, and cost. About one week-long examination shall be the standard considering the preparation and withdrawal.

(b) Examination method

1) Installation of the sand catcher and flow velocimeter

The sand catcher and flow velocimeter shall be installed promptly under waves and weather that allow installation by making preparation scrupulously and rehearsing installation as needed. Skilled divers should be used to conduct the work. Simultaneously measure the position of the installation point, the water depth, the water depth of the sand catcher and flow velocimeter, and the direction of the sand catcher. It is preferable to collect the bottom sediment at the installation point.

2) Measurement

When a precipitation-type or net-type sand catcher is used, the installation finish time shall be recorded as the measurement start time. When a water sampling-type sand catcher is used, the water sampling start time and the duration of the water sampling shall be recorded.

3) Raising of the sand catcher

When the collection (measurement) is completed, the sand catcher shall be recovered as promptly as the waves and weather allow. When a precipitation-type or net-type sand catcher is used, the recovery time shall be recorded as the measurement finish time. Check the direction of the sand catcher simultaneously. Collection of the bottom sediment at the time of recovery is preferable.

4) Analysis of the collected sample

The collected sample shall be dried and measured. The grading shall be analyzed after measurement.

5) Calculation of the concentration and conversion to the amount of the littoral drift

The concentration shall be calculated. Next, the amount of the littoral drift shall be calculated by using the measured flow velocity if it has been measured or the estimated flow velocity if otherwise.

6) Summarization of the observation record

When a precipitation-type or net-type sand catcher has been used, a list including the installation location, water depth of the installation point, water depth of the sand catcher, installation finish time (measurement start time), direction of the sand catcher at installation time, time and direction at the time of sand catcher recovery, and water depth at the time of recovery shall be created.

When a water sampling sand catcher has been used, a list including the installation location, water depth of the installation point, water depth of the sand catcher, direction of the sand catcher at installation time, water sampling time, water sampling duration, and wave condition at the observation time or during the observation period shall be created.

④ **Arrangement of data**

A list of data in which the analysis result of the samples is added to the list of the observation record shall be created. All data shall be recorded in computer.

Data shall be analyzed according to the purpose. Examples of diagrams to be created are as follows:

(a) The vertical distribution of the amount of the littoral drift and the distribution of median diameters by directions

(b) The planar distribution of the amount of the littoral drift and the distribution of median diameters

(6) Tracer Examination (Fluorescent Sand Examination)

This is the examination to estimate the moving direction and amount of the littoral drift by putting a tracer (follower) on the seabed, which moves in the same fashion as the bottom sediment (sand) on the seabed and by tracing the movement of the tracer. The fluorescent sand, which is the sand collected from the field shore and dyed with the fluorescent dye, is generally used as the tracer.

In recent years, such methods as using a microcapsule with a built-in IC tag as a tracer and detecting the tracer by the GNSS survey,[83] estimating the moving direction and amount of littoral drift by measuring the amount of fluorescent X-ray which the littoral drift itself has, without using a tracer[84] and using the colored sand containing no fluorescence as a tracer and detecting the tracer by an image analysis[85] have been tried. These methods are meaningful in labor-saving examinations; thus, they are ideal to be used in the future. Practical examples of these applications are still limited, and their evaluation has not been established.

Here, the tracer (fluorescent sand) examination which uses fluorescent sand as a tracer is described.

① **Principle of the fluorescent sand examination**

Fluorescent sand examination is a method used to grasp the migration pathway and amount of the littoral drift qualitatively by throwing in a certain amount of fluorescent sand to a certain point on the shore, collecting (sampling) the seabed sand at several points after a certain period of time, and calculating the fluorescent sand contained in the collected sand. It is also possible to estimate the moving velocity and amount of the littoral drift by core sampling with denser sampling points under certain wave conditions.[86]

Fluorescent sand often remains in the same sea area for a long time. To newly conduct a fluorescent sand examination, it is necessary to check whether another fluorescent sand examination has been done in the same sea area before. If so, then it is necessary to check if the fluorescent sand used in the examination still remains or not.

② **Equipment**

(a) Fluorescent sand, (b) mercury ultraviolet lamp, and (c) seabed sand collector (sampler) shall be used.

③ **Setting of an examination area**

Although the area determined in the field examination plan fundamentally becomes the examination area, the area may be restricted considering the purpose of the examination and the seabed topography if the area is extensive.

④ **Examination method**

The outline of the examination procedure is as follows:

(a) Fabrication of the fluorescent sand: dry the sand collected from the field shore and then mix with the fluorescent paint.

(b) Throwing in of the fluorescent sand: when throwing in, record the throwing day and time, survey the throwing location, and measure the water depth at the throwing location.

(c) Collection of the fluorescent sand: collect from the plane using the sand collector (which was used for bottom sediment examination) in principle.

(d) Detection of fluorescent sand: irradiate with an ultraviolet lamp in a darkroom and count the number of coloring fluorescent sand particles of each color.

In addition, weather, waves, flow, tide, degree of muddiness, and others shall be observed continuously in the examination period.

⑤ **Arrangement of data**

(a) Planar distribution of the numbers of fluorescent sand for each color and for the total fluorescent sand by collection days shall be illustrated, and a uniform distribution line shall be drawn. The scale of the diagram, orientation, throwing date, collection date, legend of fluorescent sand, and others shall be input in the diagram.

(b) The vertical distribution is indicated by a histogram showing the detection number in each layer of each color.

(c) The records of weather and oceanographic phenomena of the collection day and the observation period shall be added as another diagram.

(7) Observation of the Amount of Wind-blown Sand

The wind-blown sand trespasses the important facilities adjacent to the beach to impede the function of the facilities. The wind-blown sand causes a spill of littoral nourishment sand from the littoral nourishment beach. In order to cope with such a situation, wind-blown sand should be controlled. The wind-blown sand shall be examined to obtain information necessary for the wind-blown sand control.[87) 88)]

① How to estimate the amount of wind-blown sand

The movement of wind-blown sand can be visually observed, and it is a one-way movement. Measurement of the amount of wind-blown sand is easier than the measurement of the sediment amount in a river or the littoral drift on the shore. Moreover, measurement results are accurate. The amount of wind-blown sand can be estimated by utilizing the beach topographic survey or by directly measuring the amount of wind-blown sand using the sand catcher.

Table 2.9.6 Estimation Method of the Amount of Wind-Blown Sand

	Utilize the beach topographic survey	Directly estimate the amount of wind-blown sand using the sand catcher
Estimation method	■ Topographic change caused by the wind-blown sand can be distinguished from the difference between two beach topographic surveys.	■ The amount of wind-blown sand is directly measured by using the sand catcher. ■ The amount of wind-blown sand passing a unit area or unit width can be measured.
Points to note, etc.	■ The amount of wind-blown sand cannot be calculated. The amount of deposition or erosion in a certain area calculated by comparing the topographic map is the difference between the inflow and outflow in that area, and not the amount of wind-blown sand. ■ The moving direction of the wind-blown sand is the same as the wind direction. ■ The information acquired from a beach topographic map is limited.	■ The amount of wind-blown sand on the dry flat sand surface may be assumed to be proportional to the third power of wind velocity. ■ The floating wreckage washed up on the beach decreases the amount of wind-blown sand. ■ The existence of vegetation decreases the amount of wind-blown sand. ■ The water content in the sand surface greatly impacts the amount of wind-blown sand. ■ 80% or more of the amount of wind-blown sand is moving in the 20 cm or less high space near the sand surface. ■ The windward sand surface distance where wind-blown sand can be generated relates to the amount of wind-blown sand. ■ If the mean wind velocity exceeds about 12 m/s, field work becomes difficult.

(a) Used equipment

The equipment required for the examination of wind-blown sand using the sand catcher is as follows: i. sand catcher, ii. vane anemometer, iii. measurement equipment for the water content on the sand surface, iv. sample preservation bag, v. writing instrument, etc.

(b) Sand catcher

This is a device to directly catch the wind-blown sand. The sand catcher of proper size and type shall be made considering the purpose of the examination and difficulty of the work. Various sand catchers have been devised, prototyped, and attempted according to the examination purpose. There are two catching principles, namely, the compulsive catch and the free fall.

Table 2.9.7 Outline of Sand Catcher and Others for Sand Catching Examination

Classification	Compulsive catch-type sand catcher	Free fall sand catcher	Measuring equipment of the amount of wind-blown sand
Outline of equipment	■ This is a device to catch wind-blown sand with a box-like or a cylindrical container with an opening to the windward side in the direction perpendicular (vertical) to the sand surface. ■ Wire gauze or plankton net of small mesh (about 80 micron or less) is often stretched on the lee-side edge to escape the air flow. ■ This model is called a vertical-type sand catcher. The whole amount-type sand catcher which catches the wind-blown sand that passes one space area of a certain width (about 10–30 cm) and a certain height (about 2 m) at one time and the distribution type sand catcher which catches the wind-blown sand by dividing into some portions to the height direction are available.	■ This is a device to measure the amount of wind-blown sand fallen in the box embedded in the same surface as the sand surface based on the fact that the one-time level flight distance of a particle of the wind-blown sand is short. ■ The rough standard size of the box is 20–100 cm wide, 50–200 cm long, and 20–100 cm deep. ■ The horizontal whole amount type which uses a single box and the horizontal distribution type which has some partitions in the lee direction and measures the amount of wind-blown sand by its flight distances are available. ■ The wind-blown sand catching channel (trench) can expand the horizontal whole amount-type sand catcher, excavate a channel (trench) of about 1 m or more deep and about 5–8 m wide in the field beach, and also measure the amount of wind-blown sand which drops in the trench. This is one good method of measuring the amount of all wind-blown sands.	■ A prototype of wind-blown sand measuring equipment to measure the amount of wind-blown sand using the same optical principle and ultrasonic wave as the littoral drift measuring equipment has been made. ■ In addition, wind-blown sand measuring equipment which makes use of the shock pressure of the wind-blown sand particle colliding with the sensing part is also fabricated. ■ As the wind-blown sand measuring equipment has a problem in certification (calibration), independent use is not recommended. Combined use with another sand catcher is recommended.

② **Field examination**

(a) Selection of an examination point

An examination point shall be determined based on the examination purpose and field shore exploration. Examination items for the field shore exploration shall be the beach width, foreshore slope, backshore slope, and bottom sediment collection (grain size distribution).

(b) Examination period

The period when sand is blown by the wind differs by districts. Since the amount of wind-blown sand is proportional to the third power of the wind velocity, strong wind brings it in large amount. The examination period and duration shall be determined considering when sand is blown by the wind in the examination area concerned and the purpose.

(c) Observation

1) Install the measurement equipment on a workable day and wait until the sand is blown by the wind. The sand catcher shall be covered with a “lid” to prevent sand from mixing into the sand catcher.

2) Judge the measurement start time from the condition. The sand catching measurement period is about 5 to 60 min. The measurement period may be adjusted properly.

3) Cover the sand catcher with a "lid" when the measurement period is over so that the sand being blown does not mix, and collect the caught wind-blown sand. The collected wind-blown sand shall be stored in a preservation bag, and the place and time of sand catching shall be written on it.

4) When collection is completed, restart the measurement.

5) If determination of the relation between the wetness of a sand surface and the amount of wind-blown sand is one of the examination purposes, measure the water content on the surface of the sand layer.

③ **Arrangement of data**

(a) List of raw data

A list of raw data in any format shall be created by measuring the caught sample.

1) When measured by a whole amount-type sand catcher

The list of raw data shall include every measured data, including the observation place, position, observation date, measurement start time, measurement finish time, measurement period interval (measurement start time – measurement finish time), mass of measured sample, and others.

2) When measured by a distributed-type sand catcher

The list of raw data shall be created for every measurement. It shall include the distance to the center of the divided partition (segment), the height from the sand surface to the center of the divided partition in the case of vertical distribution, and horizontal distance from the windward edge of the sand catcher to the center of divided partition in the case of horizontal distribution, in addition to the items measured by the whole amount-type sand catcher.

(b) List of data for analysis

A list of data for analysis shall be created based on the list of raw data. The list of data for analysis shall include the observation place, position, observation date, measurement time (measurement finish time – measurement start time), amount of wind-blown sand, mean wind velocity, converted mean friction velocity, and other required items.

The amount of wind-blown sand is as follows, and the mean wind velocity is the mean wind velocity in a measurement period interval, the converted mean friction velocity is the value converted from the mean wind velocity to the friction velocity.

- In the case of whole amount-type sand catcher: unit width, the value converted into unit time
- In the case of distributed-type sand catcher: unit area, the value converted into unit time

(c) Illustration of data

1) The amount of wind-blown sand and the friction velocity of data obtained by the whole amount-type sand catcher shall be plotted on the logarithm paper.

2) The amount of wind-blown sand and the height (or horizontal distance) of data obtained by the distributed-type sand catcher shall be plotted on a logarithm or semi-logarithm paper.

2.10 Hydraulic Model Experiment

2.10.1 General

(1) Aim

Almost all evaluation equations and design drawings including wave force formulas and calculation charts for a wave-overtopping discharge that describe the performance of structures, are derived or verified not only by theoretical examinations, but also hydraulic model experiments. The reason is that waves, as an external force, exhibit nonlinearity. In particular, in the case of shallow water and high waves with a long period, waves often display significantly nonlinear behavior. Wave-breaking phenomenon is a typical example. So far, hydraulic model experiments are the only methods that can model such wave phenomena. In recent years, it has become possible to examine wave transformation in detail using numerical analysis. However, in the event that numerical analyses cannot accurately reproduce the phenomena of interest, especially for strong nonlinear behavior such as wave transformation including the strong eddies caused by wave breaking on a reef topography, evaluation using a hydraulic model experiment is vital.

If waves and structures interact with each other, the phenomenon becomes more complicated, and evaluation by numerical analysis requires modeling of the structure or the phenomenon, itself. However, to evaluate performance design with respect to the stability of structures, it is necessary to evaluate failure mode with the movement or deformation of these structures. In such cases, it is extremely difficult to conduct an evaluation using only theoretical treatment and numerical analysis.

As for interaction between waves and structures, although it is desirable to conduct field observations and evaluation, it is difficult to evaluate the performance of structures by field observation. This is because it is almost impossible to anticipate conditions that are similar to the design conditions given that external force conditions cannot be controlled, observation with different structure conditions cannot be conducted, and considerable effort, including cost, is needed for field observations.

Even for past design formulas and design diagrams, their application is often limited to certain specific ranges depending on the external force conditions such as wave height and period, and natural conditions, such as water depth. If conditions are out of the range of application, confirmation via a hydraulic model experiment is required.

Under the present circumstance in which numerical analysis is becoming the mainstream technique, hydraulic model experiments are necessary, and the aims of the experiment are to better understand, to confirm the hydraulic phenomena associated with waves, evaluation of the failure modes of structures that are required for performance verification, construction of performance verification method for structures, and verification of numerical analysis models.

(2) Flow of experiment

The general flow of the experiment, including preparation, is stated below:

① Setting of problems (if possible, including survey of the target port or coast)

② Collection and organization of information concerning weather, oceanographic phenomena, seabed topography, etc.

③ Setting of field conditions (tide level, wave, seabed topography)

④ Selection of experiment area and experiment structure or layout plan

⑤ Maintenance of experimental facilities and preparation of measurement equipment

⑥ Setting of experimental cases

⑦ Determination of model scale

⑧ Production of models

⑨ Examination of experimental waves

⑩ Execution of model experiments

⑪ Data analysis

⑫ Study of experimental results and execution of additional experiments

⑬ Preparation of report

(3) Systematic organization of hydraulic model experiment

Figure 2.10.1 presents the overall hydraulic model experiment for waves. Experiments involving waves can be classified into three types based on their interactions or propagation characteristics: wave transformation experiments involving wave propagation and transformation, experiments involving interaction between waves and structures, and movable bed experiments that involve interaction between waves and the bottom sediments such as sand and littoral drift issues.

Wave transformation experiments mainly include wave propagation experiments dealing with wave transformations due to changes in water depth, harbor calmness experiments that involve wave transformation behind structures such as breakwaters, and experiments concerning the nearshore current that accompany wave breaking. Plane experiments are required, except for experiments involving transformations due to changes in water depth of sections. However, given that numerical analysis technology has been advancing due to recent improvements in computer performance, it has become possible to replace wave transformation experiments with numerical analysis.

Experiments involving waves and structures include wave-resistant stability studies that deal with the effects on structures that result from the interaction between the structure and a wave, reflection and transmitted wave experiments that involve wave transformation due to interaction between structures and waves, and wave-overtopping experiments. Mutual interaction between waves and structures is a phenomenon in which the nonlinear characteristics of waves and the nonlinearity of the interaction between waves and structures are intertwined in a complicated manner. Based on recent progress in numerical analysis technology, the examination of phenomena without deformation of structures is possible using this approach. However, examinations based on hydraulic model experiments are still necessary in the case of major deformation of structures including failure, which is important to design. In addition, although stability experiments against tsunamis are included in stability experiments against waves, given that the damage to harbor structures by the tsunami caused by the Great East Japan earthquake was so severe that the evaluation of the failure modes or the mechanism to generate damage was difficult without hydraulic model experiments, tsunami-resistant stability experiments are classified separately in this report.

The movable bed experiments include experiments on the beach profile change or beach deformation targeting sand to examine the interactions between waves and the bottom sediment, and the siltation experiments involving silts. In addition, scouring experiments around structures also address the interactions among waves, structures, and bottom sediment. In movable bed experiments, if the similarity of an external force is achieved, the particle size of the sediment is significantly small. This is because given that the physical properties and underwater behavior of the sediment depend on the particle size, it is difficult to conduct experiments that perfectly satisfy the similarity rule. Therefore, it is necessary to model and conduct experiments considering the external forces and physical properties of the bottom sediment.

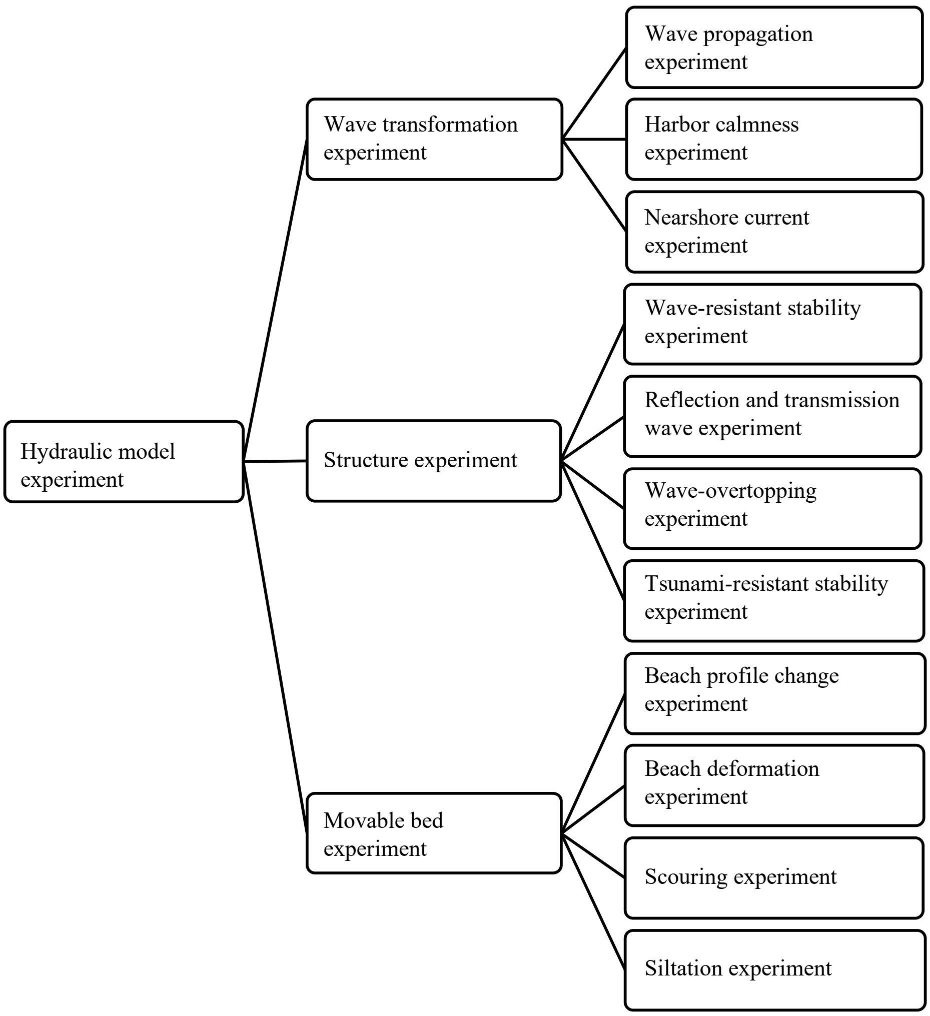


Fig. 2.10.1 Systematic organization of hydraulic model experiment

2.10.2 Classification of hydraulic model experiments for structures

Hydraulic model experiments for structures are classified as follows:

① Hydraulic model experiment on wave-resistant stability of structures

② Hydraulic model experiment on reflection and transmission performance of structures

③ Hydraulic model experiment on countermeasures against wave overtopping of structures

④ Hydraulic model experiment on tsunami-resistant performance of structures

Although the aforementioned experiments are mostly performed as section model experiments, performance verification may be performed using plane model experiments in the case that the characteristics of the plane waves, the effects of complicated seabed topography, and the effects of the plane layout of structures need to be considered (refer to **Reference (Part I), Chapter 1, 2.10.10 Hydraulic model experiment on plane wave field**).

2.10.3 Experimental equipment and measurement instrument

(1) Wave generator

Wave generator should exhibit the appropriate performance required to achieve the aims of the experiments, and it is desirable to use this equipment, which can generate not only regular waves, but also random waves. In addition, given that reflected waves from structures are generated in wave tanks in the experiments to investigate interaction between waves and structures, most of the recent wave generator serves the function of absorption and control of reflected waves. However, given that this function changes according to the period, it is important to closely examine the effects of reflected waves in experimental data, considering incident wave characteristics.

(2) Measurement instrument

In hydraulic model experiments on performance verification of structures, it is necessary to understand ① wave characteristics in the wave tank and ② wave force characteristics acting on the structures.

To understand wave characteristics in the wave tank, the equipment to measure changes in the water surface elevation in the tank, such as capacitance-type wave gauge and servo-type wave gauge, and the equipment to measure velocity such as electromagnetic current meters and ultrasonic current meters, are used. The methods to measure the wave force acting on the structures include wave pressure measurement using a wave pressure gauge attached to a structure model, wave force measurement to measure the force acting on a model by supporting the entire model, or components using a force meter or a load cell. In addition, the distortions of the members that support a model can be measured with a distortion gauge using the Rosette method, and the result can be converted into a wave force.

In using such measurement equipment, it is necessary to consider the scale of the physical quantities to be measured and the measurement accuracy required for the equipment in concern, and the achievement of adequate measurement range and resolution.

2.10.4 Setting of natural conditions for a model experiment

(1) Tide level

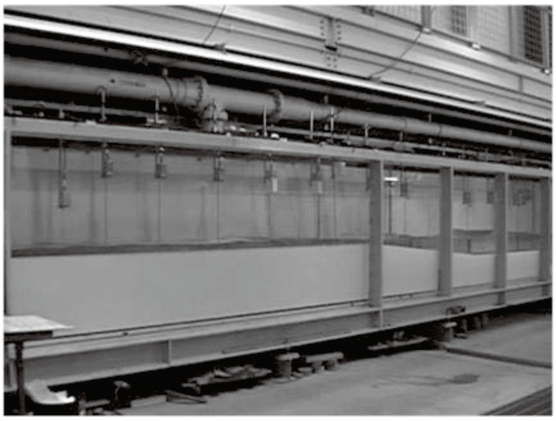
The tide level shall be set to provide severe conditions for structures as surveyed and evaluated based on the method described in **Reference (Part I), Chapter 1, 2.3 Observation and survey of tide level**. In general, the high water level shall be used, but it is necessary to note that the low tide level may be dangerous in terms of the stability of riprap.

(2) Wave

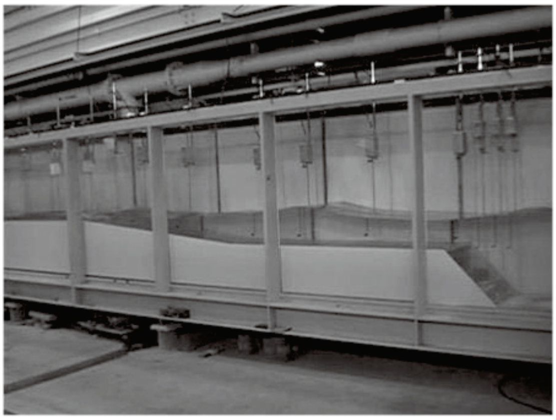
The waves shall be set and used to provide severe conditions for structures for combinations of wave height and a period, as surveyed and evaluated based on the method described in **Reference (Part I), Chapter 1, 2.4 Observation and survey of wave**. For wave-resistant stability and wave-overtopping prevention function of structures, the most severe wave that attacks the location is generally used as the design wave. However, it should be noted that waves with a wave height lower than the design wave may cause the most dangerous situation when the period is long. In addition, it is necessary to ensure that the breaker depth might occur around structures depending on the tide level.

(3) Seabed topography

For the seabed topography, in the case that waves are concentrated due to the refraction phenomenon in a plane experiment or when the topography is peculiar and the changes in topography are large, the field site topography may be faithfully reproduced in the wave tank. However, topography is usually modeled so that the field site phenomena can be reproduced in the experimental wave tank. In general, a slope with a uniform gradient is used, whereas some models have slopes with multiple steps depending on the conditions. The effective topography gradient for wave transformation is usually assumed to be an average gradient in the range of approximately a wavelength. This is because wave transformation is affected more by topographic changes on a wavelength scale than by topographic changes in a smaller scale compared to the wavelength. In reef topography, if the water depth on the reef is shallow, the wave-breaking line may be fixed around the reef edge, and it is necessary to appropriately model this topography. **Fig. 2.10.2** presents an example of modeling experiment setup for reef topography.



(a) Experiment situation on a reef



(b) Experiment situation of a reef front

Fig. 2.10.2 Example of an experiment modeling setup for reef topography including the layout of the wave gauges

2.10.5 Points of attention in a hydraulic model experiment

(1) Similarity rule and model scale

When a hydraulic model experiment using waves is conducted, similarity in terms of shape, motion, and force is required between the model and the actual field. The restoring force that acts on waves is gravity, and given that this controls wave motion, it is common to use the Froude similarity rule in which the inertial force and gravity are scaled down using the same scale. If the Froude similarity rule is used, similarity cannot be achieved for the force associate with water viscosity, and the effects of viscosity appear. Therefore, it is possible that the experiment is affected by the scale for wave-overtopping or wave-dissipating experiments at an extremely small scale. This is called the scale effect, and the outline of the effect can be generally predicted using Reynolds number, which describes the ratio of inertial force and viscosity force evaluated using the representative scale of the experiment. For wave-overtopping, conducting the experiment at a scale larger than the 10^5 order of the Reynolds number is considered to be desirable.

In experiments of long-period waves such as tides in which vertical motion can be ignored, distorted models that adapt different model scales for the vertical direction and the horizontal direction may be used, but such distorted models shall not be used in experiments for the performance check of structures.

(2) Characteristics of waves in wave tanks

In hydraulic model experiments, waves generated in a wave tank may cause unexpected transformation as they propagate. In addition, waves that are newly generated in the wave tank must be carefully considered.

① **Attenuation due to friction and dirt on the water surface**

First, attenuation of waves in a wave tank occurs because of the effects of friction on the wall surface and bottom surface, and the effects of the surface tension due to dirt on the water surface. However, the effect is normally small, except for experiments with an extremely long and relatively narrow wave flume. Given that wave attenuation caused by the effects of a wave tank is unavoidable, the effects are commonly eliminated by measuring the wave at the location where a structure is constructed and by calibrating the experimental waves so that the measured wave agrees with the incident wave to the structure.

② **Effect of the reflected wave**

Given that a wave tank is a closed space surrounded by sidewalls in an experiment with a structure, partial standing waves are formed due to reflected waves from the structure. Therefore, the wave height changes with time and space even in an experiment with regular waves. In the case that partial standing waves are formed, it is necessary to evaluate the waves based on methods that include separating and estimating incident waves and reflected waves using an array of wave gauges. In addition, in the case when the attenuation of the reflected waves that accompany propagation is small, reflected waves are reflected repeatedly by the wave-generating paddle, and the so-called multiple reflection state may occur. When multiple reflection states occur, the wave height level in the wave tank increases with time, and it is difficult to conduct a stable experiment. To solve this problem, an experiment using wave generator with a function to absorb and control reflected waves is desirable.

③ **Wave around the front part of the wave train**

In the case that regular waves are generated by wave generator, if wave generation is started abruptly, energy changes discontinuously from a state without wave energy to a state with wave energy in the front part of a wave train. In practice, in the discontinuous part of the energy, in addition to the diffraction phenomenon in which energy moves from a high-energy place to a low-energy place, the scattering phenomenon, in which the new waves are generated from the discontinuous part of the energy, occurs. Namely, the discontinuous part at the front of a wave train immediately after wave generation shall be smoothed out and accompanied by oscillations as the wave train propagates. Therefore, in the case that waves are generated abruptly by wave generator, even if regular waves are generated, the wave height would not be stable but would fluctuate around the front part of the wave train. This oscillation is normally suppressed not by generating waves abruptly, but by increasing the energy level gradually to a certain level using a gradual startup device. Although it is possible to suppress this oscillation further by extending the time to complete gradual startup, the startup time is normally limited to several waves because the effects of reflected waves in the wave tank appear before reaching a stable wave train. For this reason, it is difficult to suppress the oscillation at the front part of a wave train completely, and the observed wave height would not remain constant, but would show some fluctuation even in an experiment with regular waves. **Fig. 2.10.3** presents the calculation result of CADMAS-SURF at the front part of a wave train at the location about 10 wavelengths from the wave-generating point with 20 m water depth, 10 s period, and 4 m wave height (refer to **Reference (Part I), Chapter 1, 2.11.7 3-dimensional numerical analysis** for the analytical method using CADMAS-SURF). The wave train increases gradually at the front part due to the effect of smoothing, and the following wave trains show oscillation of the wave amplitude.

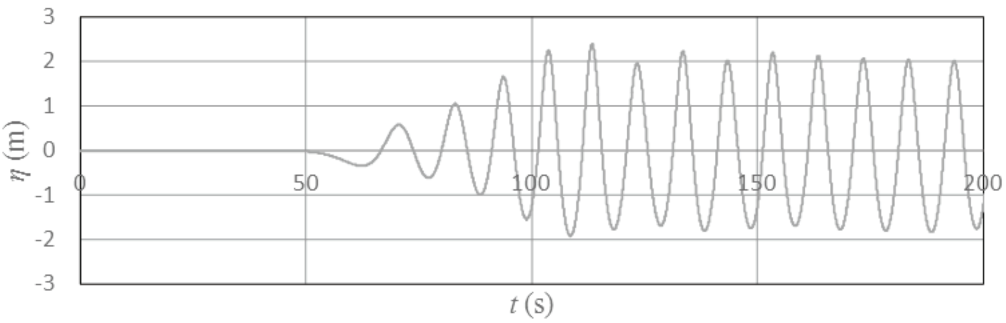


Fig. 2.10.3 Smoothing of discontinuous part and oscillation of the wave profile at the front part of the wave train (Calculation example by CADMAS-SURF; water depth 20 m, period 10 s, wave height 4 m)

④ **Modulation instability**

In the case that regular waves are generated in a wave tank on the condition that the water depth is sufficiently deeper than the water length, a phenomenon called modulation instability caused by wave nonlinearity occurs, in which water trains fluctuate with time as they propagate, and regular waves may not be generated (**Fig. 2.10.4**) [90), 91)]. The condition in which the modulation instability occurs is $kh > 1.363$, with k referring to the wavenumber and h is the water depth. This condition is achieved by stable analysis based on the nonlinear Schrödinger equation for weakly nonlinear wave train amplitudes, such as Stokes waves. Given that it is difficult to conduct an experiment with regular waves in this condition, it is necessary to change the experimental conditions such as water depth. In addition, given that the modulation instability can be regarded as an effect of irregularity, it is possible to conduct an experiment with random waves without considering the modulation instability.

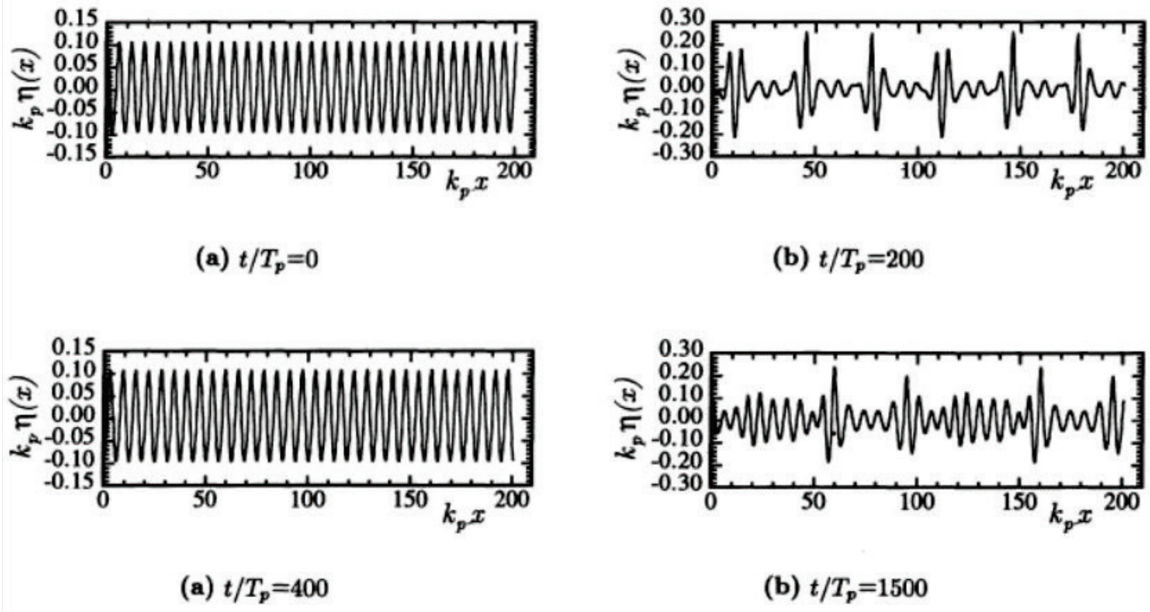


Fig. 2.10.4 Example of changes in wave shape during the propagation process of Stokes waves accompanied by initial disturbance (Yasuda and Mori [91)])
(Calculation under the condition of $k_p a_p$ = 0.1; η is the water surface elevation, k_p is the wave number, T_p is the wave period, a_p is the amplitude of the regular waves)

⑤ **Generation of waves by resonance with a wave tank**

In some cases of experiments in wave tanks, transverse oscillation occurs as component waves with the same wavelength in the width direction of the experimental flume, and the waves used for experiments are amplified by the resonance phenomenon. This would not normally cause significant effects, but in case there are effects, these can be decreased by using a flow straightening plate in the wave flume.

⑥ **Generation of secondary wave crest** [92) 93)]

In the case that regular waves are generated with wave generator, such as pistons, on the condition that wave shape gradient is large to a certain extent and relative water depth is small to a certain extent, a small secondary wave crest appears between two wave crests accompanying wave propagation. This phenomenon is caused by the nonlinearity of waves and is known as the secondary wave crest. It is generated by the secondary mutual interaction of waves and by the interaction between waves and the displacement of a wave-generating paddle. Specifically, whereas waves exhibit sharp wave crests and flat wave troughs due to nonlinearity, displacement of a wave-generating paddle normally moves in a sine curve, and the difference between the two generates new waves. However, although the wave-generating data is originally created so that the speed of a wave-generating paddle agrees with the velocity at the initial position of the wave-generating paddle, as the wave-generating paddle is displaced, a difference between the velocity at the position of the wave-generating paddle and the displacement velocity of the wave-generating paddle occurs, and new waves are generated. Although the same kinds of waves are generated when random waves are generated, it is not apparent because of the irregularity of the waves. Power spectrum analysis of water surface elevation data at the front surface of a wave-generating paddle confirms the existence of ripples with increased energy at the frequencies of an integer time of the peak frequency of the spectrum. Although the energy of this secondary wave crest is extremely small and seldom causes problems in typical experiments, when the effects of secondary wave crests are eliminated, it is necessary to prepare signals without these effects as wave generation data or for data analysis assuming the existence of secondary wave crests. As the wave generation data from which the effects of secondary wave crests are eliminated, the displacement velocity of a wave-generating paddle that agrees with the values obtained for the holizontal velocity based on nonlinear wave theories including Stokes wave theory and cnoidal wave theory, shall be given [94)].

⑦ **Generation of long-period waves**

In the case that random waves are generated with wave generator such as pistons, slow fluctuations in the mean water level are generated due to nonlinearity of waves and operate together with changes in momentum. This fluctuation in the mean water level is called the bound waves because the change is connected to and bounded by the wave group. The occurrence of bound waves is an unavoidable phenomenon due to the characteristics of the waves. As presented in **Fig. 2.10.5**, given that these bound waves that are not normally considered as waves are generated, other new waves, which are as large as the bound waves with a phase shift of 180º from that of the bound waves, are generated at the front surface of a wave-generating paddle at the same time as if to cancel the bound waves. The fluctuation of the mean water level propagates in the group velocity of the wave group. The waves with the 180º phase shift are called free waves because they propagate with a wave velocity that is unrelated to the wave group, and they are generated unintentionally as new waves. Given that these long-period waves are normally small, they do not cause major problems. In addition, similar long-period fluctuations can be observed on the actual coast. Therefore, experiments are usually conducted without taking special measures. In the case that these bound waves cause problems, it is necessary either to apply a wave-generating method considering the effects these waves, or in the case that the water is deep and the generated bound waves are small, to make the front surface of the wave generator deeper and to use a gentle slope so that the water depth gradually reaches a certain value. As for the wave-generating method with consideration to bound waves, it is suggested to use the signal with consideration to bound waves at the front surface of the wave-generating paddle and to add a correction for the displacement of the wave-generating paddle as in the case of the secondary wave crest [95)].

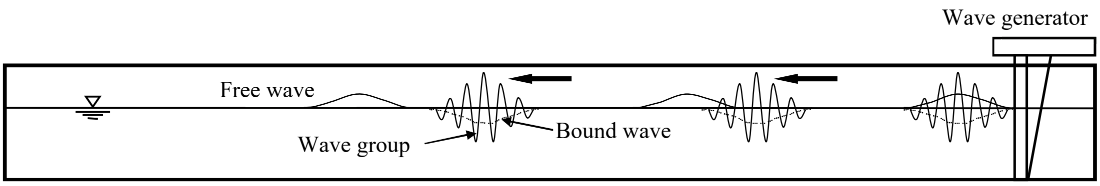


Fig. 2.10.5 Image of occurrence of long-period wave accompanying wave generation and propagation of bound waves and free waves

(3) Measurement time

① **Experiment with regular waves**

A feature on the propagation of waves is that there are roughly two types of velocities at which waves propagate. One is called the wave velocity or phase velocity: the velocity at which a wave profile progresses and travels or a phase at which the wave travels. The other is called the group velocity, which is the velocity of the entire group of waves, which propagate in a line. It is known that group velocity is in agreement with the velocity at which wave energy transmits.

Regular waves can be regarded as a wave group of an infinite collection of waves with the same wave height and period. Therefore, each wave in regular waves propagates at a wave velocity, whereas the front part of a wave train of regular waves propagates at the group velocity, as it is the beginning of a range of waves. Waves that progress at the wave velocity are drastically attenuated at the front part of the wave train of regular waves so that it appears as if the front progresses at the group velocity. In physical interpretation, although each individual wave attempts to propagate at the wave velocity, given that the wave energy is provided at a group velocity smaller than the wave velocity, each wave that attempts to progress before the front part of the wave train at the group velocity, is not supplied with energy and is attenuated in succession.

As presented in **Fig. 2.10.6**, if the distance from a wave-generating paddle to a structure is l and the group velocity is C_g, as the front of the wave train propagates at the group velocity, the period of time t_i from the time when wave generation starts in the wave flume to the time when the wave group arrives, can be expressed as

l/C_g. Therefore, in the case of experiments with regular waves, valid data as a wave cannot be obtained from the time when wave generation starts to the time t_i, the arrival time of the wave group. However, in the case of experiments on structures, reflected waves are generated when incident waves arrive at the structure, propagate towards the wave-generating paddle, reflect again at the plate, propagate again towards the structure, and then reach the structure. Therefore, the period of time from the point at which the waves are generated to the point when the re-reflected waves from the wave-generating paddle reach the structure, is the time during which waves propagate three times as long as the distance l between the wave-generating paddle and the structure. As such, it can be considered that the effects of reflected waves from structures are negligible until the time $t_r = 3l/C_g$. Therefore, in experiments with regular waves, the time between t_i and t_r is the period during which incident waves are stable and there are no effects of reflected waves.

However, given that there are effects of diffraction and scattering at the front of the wave train, it is desirable to perform measurements that are longer than this period (from earlier time to later time) and to select the part in which wave profiles are stable to determine the time period for data analysis.

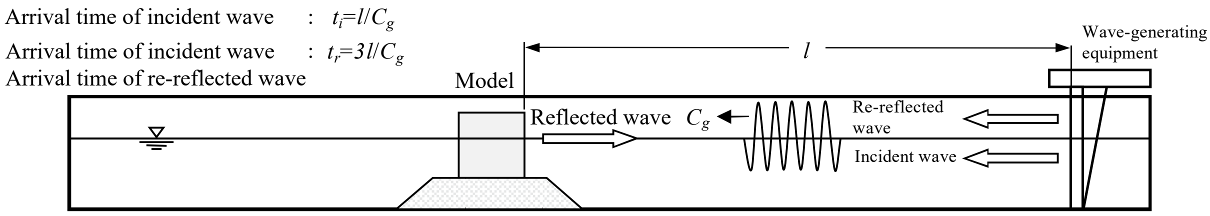


Fig. 2.10.6 Explanation of data measurement time in experiments with regular waves
(Time from the period of arrival of the incident wave to the time taken for the re-reflected wave to arrive)

② **Experiments with random waves**

In experiments with random waves, it is desirable that the measurement time is as long as possible to improve the statistical stability of the measurement data. According to Goda [96)], it is necessary that three or more sets of wave groups consisting of at least 200 waves should be used for this process. This is because the standard deviation of the significant wave height from the record of 200 waves is 4 %, and therefore, it is probable that the estimate would deviate from the target significant wave height by 4 % or more once in three attempts.

The following is a discussion of wave height distribution based on a statistical viewpoint. Normally, when a harbor structure is designed, 1/250 maximum wave height is adopted as the highest wave height in a group of random waves. Even if the distribution of wave height changes little, the expected value of the highest wave height in 400 generated waves is consistent with the 1/250 maximum wave. Therefore, the use of three or more sets of wave groups consisting of 400 waves in action may also be proposed. Based on the assumption that the wave frequency is approximately 10 s, 200 waves correspond to a duration time of approximately 30 min, and 400 waves correspond to a duration time of approximately 1 h.

Based on these considerations, with respect to the measurement time in experiments with random waves, in terms of the number of waves, actions by three or more sets of wave groups consisting of 200 to 400 waves can be regarded as a standard. In reality, the duration time of waves is also included. The measurement time shall be determined considering field site wave conditions, the incident wave examination result, wave height distribution, values of the highest wave height, and experiment cost.

(4) Measures against reflected waves

As previously noted, the effects of reflected waves are observed in wave tanks, and it is necessary to implement appropriate measures. However, even though the equipment to absorb and control reflected waves is used, it is impossible to totally eliminate their effects. Therefore, it is important to conduct an evaluation of reflected waves in experiments and to confirm if the waves that are incident on the structures fulfill the determined conditions.

It is usually the case that two units of wave gauges are used, and incident and reflected waves are separated based on the incident/reflected wave separation estimate method by Goda [96)]. In this approach, the water surface elevation recorded by the wave gauges are analyzed using the Fourier transform, and the amplitude of the incident and reflected waves are determined using the relationship between their amplitude and phase. In the case of random waves, the energy of incident and reflected waves of the entire wave group is determined by the amplitude of these waves for individual component waves. The reflection rate is determined by extracting the square root of this ratio. In this case, if the distance between the two wave gauges is Δl, the separation of the incident and reflected waves is possible for components with wavelengths in the range of (2.2–20)Δl. Δl is determined so that the main part of the wave spectrum used for experiments, for which the effects of nonlinearity are eliminated (normally within the range of 0.5 to 1.8 times of peak frequency) satisfies this wavelength condition.

In the area where the water depth is relatively shallow, if the water surface fluctuation and the velocity fluctuation at the same locations are measured using a wave gauge and a current meter, it is possible to separate incident waves and reflected waves using a pseudo-nonlinear long-wave model. Pseudo-nonlinear long-wave models [97] have a feature of calculating water surface fluctuation of time history of incident waves and reflected waves. If the water depth is h, the water surface elevation is η, and the velocity is u, the time series of the water surface fluctuation of the incident waves η_i and the reflected waves η_r are expressed as follows.

$$\eta_i = \frac{1}{2}\left[\eta + u\sqrt{\frac{h}{g}}\cdot\frac{h}{h-\eta}\right] \tag{2.10.1}$$

$$\eta_r = \frac{1}{2}\left[\eta - u\sqrt{\frac{h}{g}}\cdot\frac{h}{h-\eta}\right] \tag{2.10.2}$$

(5) Examination of experiment waves

In experiments with regular waves, a sine wave is used as the input signal. At that time, the relationship between the gain of the input signal and the wave height value obtained from the water surface fluctuation at the location where the model will be positioned to determine the gain of the input signal to the target wave height. Attention shall be focused on the aforementioned variability of the regular wave height, and the wave height value is calculated by averaging approximately 5 to 10 waves with stable wave heights.

In the case of random waves, the water surface fluctuation at the point where the model will be positioned is measured, the significant wave height values and the power spectrum forms are analyzed, and the difference between the observed shape of wave spectra and the shape of target spectra is corrected to match the spectral shape. Moreover, the gain is corrected so that the significant wave height is in agreement within the error by several percent. These operations are repeated several times until the measured value and the target value agree within an error of several percent. Subsequently, the significant wave period, the maximum wave height, etc. shall be compared with those of the target values. If the error is large, the initial random numbers are changed to generate signals of random waves and the preceding operations are repeated to obtain the prescribed input signal. The changing of the initial value of the random numbers of the input wave signals is repeated to generate three-wave groups using the same procedure.

In the case of multi-directional random waves, it is necessary to achieve an agreement with the target spectrum within an error of several percent not only for the power spectrum, but also for the directional wave spectrum. In multi-directional random waves, it is common to use the single summation method because the randomness of frequency and wave direction is introduced as random numbers. The idea of this method is not that one frequency consists of component waves with various wave directions, but that randomly selected one wave direction is allocated to one frequency as component waves. Specifically, the frequency spectrum is divided into several hundreds of component waves so that they all have equal energy. Then one wave direction is randomly selected and allocated to one frequency using random numbers. After a frequency is selected in this way, component waves with one wave direction are calculated for all the frequencies, and multi-directional random waves are calculated by adding all the component waves. This method is called the single summation method because the water surface fluctuation in a multi-directional wave field is given as the sum of a single series. With respect to the detailed explanation of methods to generate multi-directional random waves, refer to Hiraguchi et al. [98] and Hiraishi [99].

2.10.6 Hydraulic model experiment on wave-resistant stability of structures

(1) Wave-resistant stability

Hydraulic model experiments on wave-resistant stability of structures are conducted to assess the wave force used in the design of structures, to confirm stability by evaluation of the failure limit conditions of sliding of gravity structures and the margins in the wave-resistant design (hereinafter, wave-resistant margins), and to evaluate of damage rates and stability numbers used for wave-resistant design of covering stones, and wave-dissipating blocks.

Margins are often defined as the ratio of design conditions to failure limit conditions, and agree with so-called safety factors in design. However, it is seldom the case that margins and safety factors actually agree, and it is often the case that systematic gaps are generated.

Target structures are breakwaters (caisson type, sloping type, upright wave-dissipating type, and floating body type breakwater), in addition to the members attached to breakwaters such as armor units as well as wave-dissipating blocks placed on the front surfaces of breakwaters, revetments, and the like.

Concerning wave-resistant stability, given that it is necessary to understand the failure modes of structures, experimental plans including the confirmation of the behavior of structures against waves, need to be developed.

The phenomena that should be considered while evaluating wave-resistant stability of structures include ① the wave-pressure or wave-force acting on structures, ② the wave-pressure distribution or wave-force acting on structure members, ③ sliding and overturning of upright section of gravity type structures, ④ the stability of riprap and wave-dissipating blocks of sloping breakwaters, ⑤ the stability of armor units and foot protection work, and ⑥ the stability while mooring floating body type structures.

(2) Specifications of wave and tide level for the experiment

With respect to the external force required to evaluate wave-resistant stability, a range of wave height from low to high (exceeding design wave) must be covered, centered on the design waves of the structures. In particular, in experiments to confirm the wave-resistant margins and in the sliding and overturning of bodies of gravity-type structures, it is necessary to perform an examination from the waves height lower than the design condition, in which structures shows still stable, to the wave height exceeds the design waves, in which devastating damage occurs. It is known that wave-dissipating blocks are structures with relatively large wave-resistant margins, and devastating failure does not occur, even for conditions exceed the design waves. Based on this, the wave height used for the experiments shall be set within the range of 0.7 to 2 times the design wave height. Although wave periods are normally set in combination with the wave height by considering the maritime weather conditions at a target location, waves with periods longer than those of the design waves should be included.

Concerning the tide level, it is usually the case that if it is higher, the wave height also becomes higher and provides more severe conditions for the stability of structures. When there is a condition in which strong wave breaking occurs in the relationship between water depth and wave height, this tide level must also be added to the conditions. With respect to the armor blocks and foot protection blocks that are susceptible to the effects of the flow, the wave velocity may be higher in the low tide level, and it is necessary to set adequate conditions within the range between LWL (mean monthly lowest water level) and HHWL (highest water level on record) based on field site conditions.

(3) Model scale and model production

Although it is desirable that models for wave-resistant experiments should be as large as possible, the model scale shall be determined by considering the size of the experimental wave tank and the maximum height of the waves that can be generated. A model scale of approximately 1/10 to 1/50 is generally used, and model scales between 1/20 and 1/30 are most commonly adopted.

The models shall be scaled down based on the Froude similarity rule. Therefore, in experiments on sliding and overturning of caissons, the mass of a caisson model shall be reduced to the third power of the model scale, and the position of the center of gravity shall be adjusted so that it is in the same position as the field conditions. In the case of sliding experiments, the friction coefficient between caissons and the riprap needs to be matched to the design conditions based on friction experiments. Although models for caissons are usually produced by forming the outer box with mortar or concrete, it is now common to produce models with acryl, which is relatively easy to work with and has sufficient strength. In this case, some measures are necessary, such as the construction of the bottom plate with concrete material so that the friction is the same as the field situation.

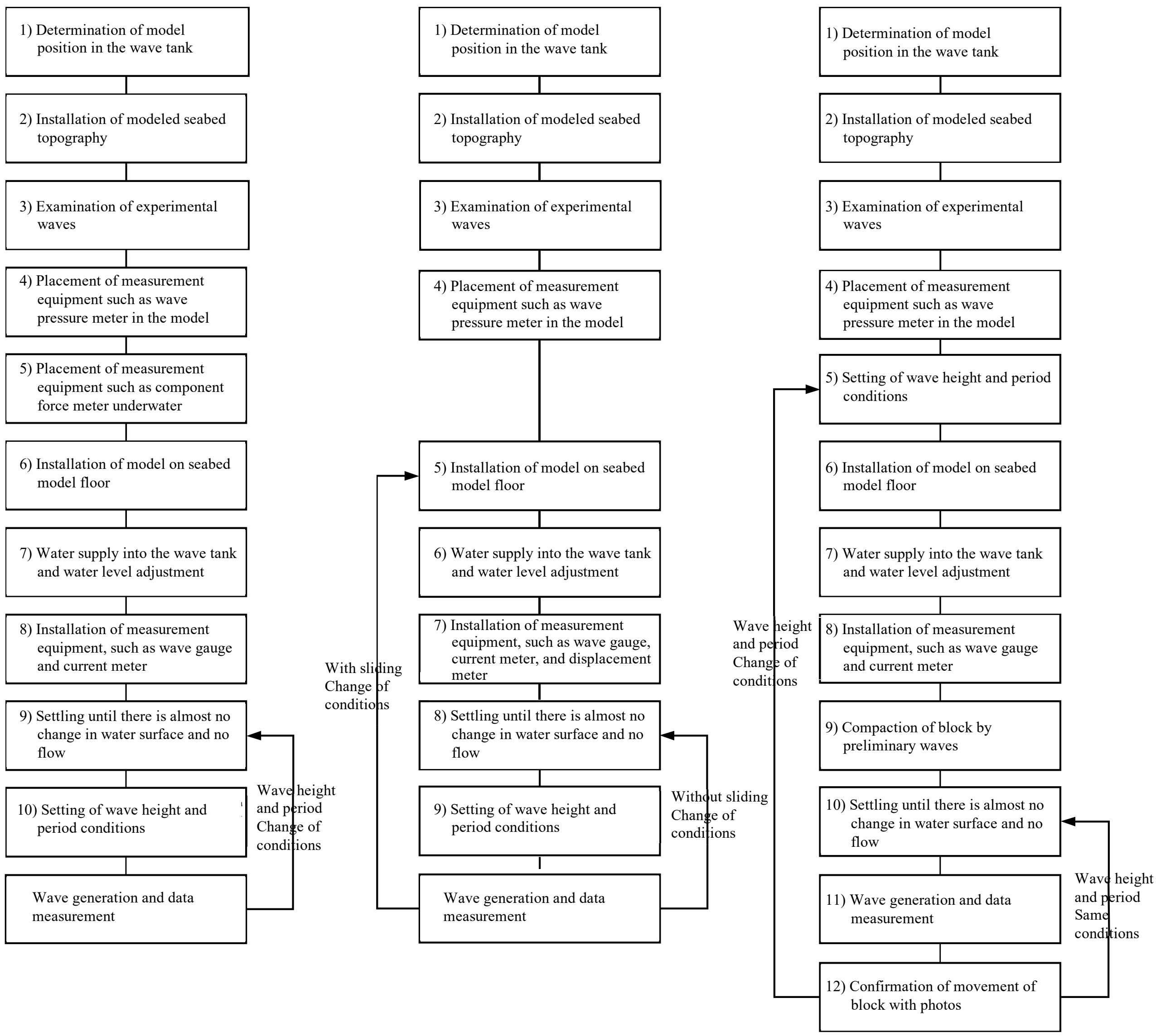
If wave-pressure and wave-force acting on the breakwater body and members are measured, wave-pressure or wave-force cannot be measured accurately when the breakwater body moves. Therefore, measures that involve making breakwater bodies heavy or fixing the bodies to seabed models are considered to prevent rocking due to large external forces.

Wave-dissipating block models are produced using mortar or concrete with small-diameter aggregates. Weights made of small pieces of iron or lead are embedded in blocks to adjust the position of the center of gravity and the weight.

In the case that mooring experiments of floating breakwaters are conducted, if the shape and the weight of the floating body are matched based on the Froude similarity rule, and the position of the center of gravity is adjusted, the center of buoyancy and the metacenter height are reproduced in the model scale. As for mooring ropes, considering the spring properties of actual mooring ropes, chain models, or strings made from adequate material shall be used to reproduce the properties in the model scale.

The seabed topography shall be modeled to reproduce wave transformation at the front side of structures. In the case that the field site seabed slope can be considered to be uniform, the model seabed topography is set to have the same seabed slope. In the case that the topography changes in a complicated manner, the field site topography could be reproduced with sand and mortar. Otherwise, given that the wave transformation is most strongly affected by the average seabed slope of approximately one wavelength, the local topography can be smoothed to form a seabed slope with one step or several steps as an approximation.

The border between a seabed topography model and the sidewalls of a wave tank shall be filled with silicon or similar material to prevent water leakage. If this measure is not taken, the pressure shall decrease due to water leakage, the water surface fluctuation shall be affected, and wave transformation such as wave shoaling cannot be accurately reproduced. This is also true for the border between a model and the sidewalls of a wave tank. If water comes in and out, the wave force cannot be measured accurately because the pressure decreases. In this case, water must be stopped by using a hard sponge or similar structure positioned along the outer surface around the model on the side of the model, as well as material that does not prevent wave transformation.



(1) Experiment on wave force and wave pressure acting on structures and members

(2) Sliding and overturning experiment of structures

(3) Stability experiment of wave-dissipating blocks

Fig. 2.10.7 Procedures of hydraulic model experiment on wave-resistant stability

(4) Experiment method

① Experiment procedure

Experiments shall be conducted according to the procedure presented in **Fig. 2.10.7**.

(a) Experiment on wave force and wave pressure acting on structures and members

1) Determine the position of the model in a wave tank considering the arrival time of reflected waves and the effects of water level rise in the back side water area due to wave-overtopping. 2) Reproduce the modeled seabed topography in the wave tank and 3) examine the experimental waves. Subsequently, 4) place the measurement equipment such as the wave pressure gauge in the model. 5) Place the measurement equipment such as the component force meter underwater. 6) Install the model on the seabed floor. 7) Adjust the water level according to the tide level setting. 8) Install the measurement equipment such as the wave gauge and current meter. 9) Allow the model to settle until there is neither disturbance on the water surface nor water flow. 10) Set the conditions such as the wave height and period, start wave generation, and obtain various data using the measurement equipment. Then repeat 9) and 10) for different conditions. For procedure 10), it should be noted that if the experiment is conducted while the water surface is moving, reproducibility of experiment may not be achieved. The measurement of the wave force acting on floating body type structures shall be conducted in the same manner, but more measurement parameters such as the mooring rope tension and floating buoy motion will be included.

(b) Sliding and overturning experiment of structures

After procedures 1) to 4) are conducted, the following procedures are carried out: 5) Install the model on the seabed floor. 6) Adjust the water level according to the tide level setting. 7) Install measurement equipment, such as a wave gauge, a current meter, and a displacement meter, to measure the position of the breakwater body. 8) Allow the model to settle until there is neither disturbance on the water surface nor water flow. 9) Set conditions, such as wave height and period, start wave generation, and obtain various data using measurement equipment. Then, in the case that no sliding or overturning of the breakwater body occurs after the start of the wave generation, repeat procedures 8) and 9) for different conditions. As for the wave conditions, the wave height shall be increased gradually from low to high. If sliding or overturning occurs, repeat from procedure 5).

(c) Stability experiment of wave-dissipating blocks

With regard to the stability of wave-dissipating blocks, armor stones, and foot protection blocks, it is known that damage advances almost in proportion to the square root of the length of wave action time. Therefore, it is necessary that waves should act as long as the expected wave duration or longer. The standard number of waves in action shall be 1,000, and the standard number of wave groups in action shall be 3. However, depending on the stability of the wave-dissipating blocks and the field site wave conditions, the number of waves may increase. In addition, as the movement of wave-dissipating blocks gradually progresses, it is difficult to judge whether damage occurs or not, unless the development of this process is continuously monitored. Therefore, it is often the case that the following procedures are repeated in the experiment until the final wave action time: stop wave generation appropriately, check the development of the damage situation by acquiring images, and continue the next wave generation under the same wave conditions. In addition, wave-pressure acting on the wave-dissipating blocks may be measured using a wave pressure gauge. The experimental procedure is the same as the "Sliding and overturning experiment of structures" from 1) to 7), but the "Stability experiment of wave-dissipating blocks" 8) or later shall be as follows. Then 8) set the conditions such that the wave action time to check wave height, period, development of the damage situation and the like, and the final wave action time. 9) To suppress the unevenness in piling up wave-dissipating blocks and to promote compaction, preliminary waves with a wave height lower than the waves used in the experiment are generated. 10) Allow the water to settle until there is neither movement in the water surface nor water flow. 11) Start wave generation and continue generating until the wave action time to check the development of the damage situation. 12) Acquire images of the damage situation. Repeat procedures 10) to 12) until the final action time. When the final action time is reached, go back to 5) and repeat the experiment in the next conditions.

② Placement of measurement equipment and measurement methods

One or two units of wave gauge shall be placed on the front side of the wave generation equipment to examine the wave generation situation. In the area on the front side of the model breakwater body with uniform water depth, approximately 3 wave gauges shall be placed to separate the incident waves and the reflected waves. In addition, a wave gauge may be placed on the front side of the breakwater body to examine the correspondence with the wave pressure acting on the breakwater body for use as the reference data for the comparison with numerical simulations. In the case that the transmitted waves of wave-overtopping occur, a wave gauge may also be placed on the back side of the breakwater body.

To measure the wave force acting on the breakwater body, wave pressure gauges shall be placed not only on the front side of the breakwater body and the bottom plate part, but also on the back side. Furthermore, in the case that the breakwater body sinks in the water or the overtopping flow is significant, wave pressure gauges shall also be placed on the upper side of the breakwater body at appropriate intervals. It is desirable that the interval for the installation of the wave pressure gauges should be short enough to capture the target phenomenon. It is common to use one measurement line at the center of the breakwater body model or two measurement lines around the center of the breakwater body model. When a wave pressure gauge is installed in the model, a jig shall be used to fix the wave pressure gauge. However, it is necessary to take appropriate measures, such as engraving the surface of the model to install a wave pressure gauge so, that the pressure-receiving surface of the wave pressure gauge or the fixing jig is on the same level as the model surface.

Given that there are different types of current meters with different measurement mechanisms, such as electromagnetic current meters, ultrasound current meters, propeller current meters, and laser current meters, it is necessary to select the appropriate equipment to match the experimental conditions. In hydraulic model experiments for performance verification of structures, current meters are commonly installed around the structures, and electromagnetic current meters are often used because they are relatively easy to handle. In the evaluation of the stability of the mound armor units, the velocity near the bottom surface is measured because the velocity at the position where the armor units is damaged is important. However, as velocity values near the bottom surface vary depending on the measurement height, it is necessary to pay attention not only to the installation position, but also to analyze the measurement data considering these effects.

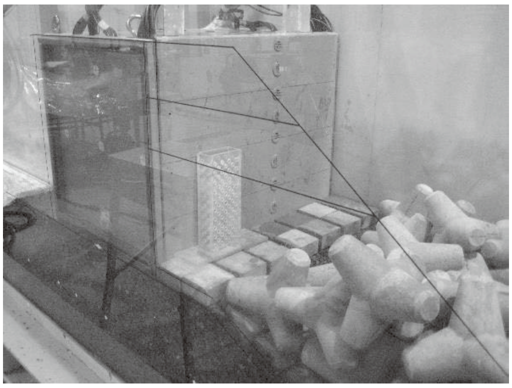
As for wave-dissipating blocks, mound armor units, and foot protection blocks, to facilitate the ease of recognition of the movement of blocks in acquired images, the blocks are often colored at an arbitrary width.

In experiments related to the stability of floating body structures, the tension of a mooring rope is often measured at the mooring point using a tension meter. However, it is difficult to directly measure tension underwater because the tension meter or the load meter experience a fluid force and this affects the tension of the mooring rope.

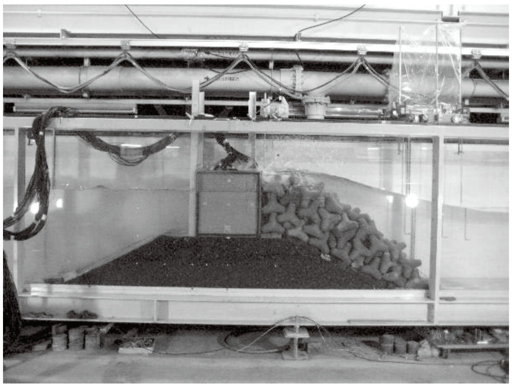
In addition, in the case of a floating body structure, it is necessary to measure the extent of vibration. A three-dimensional accelerometer and clinometer shall be installed on the floating body to measure acceleration and angles of inclination. The measured acceleration shall be integrated twice off-line to be transformed into a displacement value, to obtain the vibration data consisting of displacement and inclination angles. In the case that displacement is obtained by the second integration of an accelerometer, the measurement accuracy varies depending on the frequency. It is necessary to be aware that error in displacement, especially on the low-frequency side, becomes large. Due to recent advancements in image analysis technology, equipment has been developed applied to the measurement of the time series of three-dimensional displacement and inclination angles by installing several targets with sufficiently smaller weights compared to the model on the floating body, and by monitoring the change of the positions of the targets over time using multiple Charge-Coupled Device (CCD) cameras. Given that the displacement of structures can be measured without contact, high-accuracy vibration data may be obtained by utilizing this technology.

Experimental data are obtained using a personal computer or a data logger equipped with an Analog to Digital (AD) conversion board. It is necessary to visualize the measurement data using a monitor and to examine the circumstance under which data are obtained, such as the noise level during the experiment. The sampling interval of the experimental data needs to be properly set according to the phenomenon. For phenomena such as wave data that have a similar time scale as the wave period, a resolution of approximately 20 Hz is sufficient. However, for phenomena such as wave pressure data acquired at the time when wave-breaking effects are in action, data may be obtained at a sampling frequency of 1,000 Hz. In the case that transitional phenomena such as wave breaking are examined, the sampling frequency may be determined after performing preliminary experiments to determine if the characteristics of the target phenomenon are sufficiently reflected in the data.

Video recording of experiments from multiple angles is desirable to monitor the progress of the experiments. When experimental data are examined, it is most important to understand what kind of phenomena occurs in the wave tank. An example of a wave pressure experiment is presented in **Fig. 2.10.8**.



(a) Installment of a wave pressure meter



(b) Situation of wave pressure experiment

Fig. 2.10.8 Example of wave pressure experiment

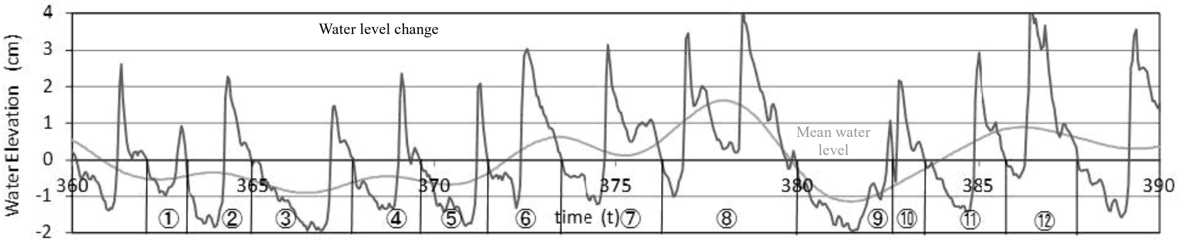
(5) Organization of experimental data

① Primary processing of experimental data

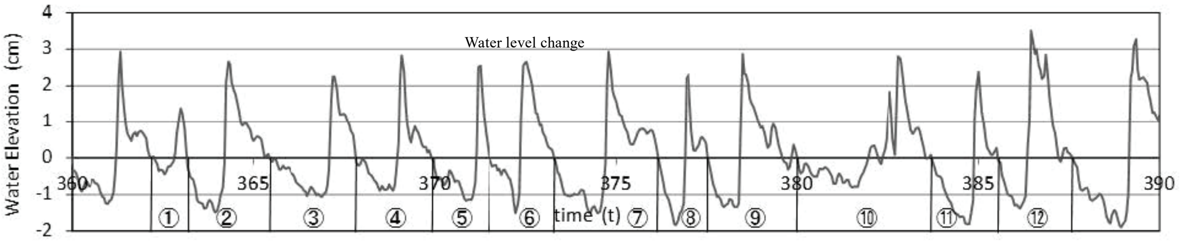
First, it shall be confirmed that the data do not contain noise. Although the time-series wave profile may be printed and visually checked, time-series data are usually processed numerically, and in the case that spike noise is found, an adequate threshold shall be set, and a judgment shall be made based on the amount or rate of change of the data values before and behind the noise, and the data shall be adjusted if necessary. If exist high-frequency white noise, it may be possible to remove the noise using a numerical low pass filter. However, in the case that the data resolution at the time of measurement is low compared to the noise frequency, removal of the noise is difficult. Therefore, either noise shall be removed at the time of the experiment so that the high-frequency noise level becomes small, or the sampling interval shall be set as small as possible when data are logged, and the noise shall be processed using a numerical low pass filter.

In the experiment with random waves, in the case that the water depth at the measurement point is shallow, the amplitude of components of long-period waves and normal waves may be the same order. As presented in **Fig. 2.10.9**, if long-period waves and normal waves coexist, the definition of wave height by the zero-up-crossing method or the zero-down-crossing method may not be able to adequately describe the individual waves. **Fig. 2.10.9** also shows that removing the average water level change and then applying the zero-down-crossing method (especially concerning ⑧, ⑨, ⑩) can produce more appropriate results.

If the water depth is shallow, it is necessary to obtain the wave height by extracting only wave components using a numerical high pass filter. In this case, a spectrum analysis shall be conducted in advance. Based on the fact that the power spectrum has two peaks of long-period waves and normal waves in general, the frequency at the dividing lines between the two peaks shall be set as the frequency to divide long-period waves and normal waves.



(1) Application of zero-down-crossing method without removing average water level



(2) Application of zero-down-crossing method after removing average water level

Fig. 2.10.9 Example of difference in average water level processing in the analysis of the wave

② **Analysis of experimental data**

The wave height of individual waves, the wave amplitude, the velocity amplitude, similar parameters shall be calculated based on the water surface elevation data, the velocity data, the wave pressure/wave force data, and the displacement data of structures using the zero-up-crossing method. Moreover, the average quantities, such as the mean wave height shall be defined using adequate statistical processing. In the case of regular waves, as data varies due to oscillation at the front part of the wave train, data averaging operation shall be conducted for the stable part of the data within the time period until re-reflected waves return and the average of the wave height, the wave amplitude, the velocity amplitude, the wave pressure and wave force amplitude are calculated. In the case of random waves, statistical values, such as the maximum value, the significant value, and the mean value, shall be obtained. When data are analyzed, these statistical values are normally used for discussion.

As for the wave data, in experiments with regular waves, the mean wave height obtained by averaging operation shall be used as the wave height. In the case of random waves, the maximum wave height, the significant wave height, the mean wave height, and their corresponding periods are calculated. If the effects of reflected waves are significant in the case of random waves, the representative reflection rate of a wave group shall be calculated using the approach by Goda [96]. The significant wave height and the mean wave height of the incident wave component calculated based on the representative reflection rate may be used for analysis.

Although the wave pressure data of structures shall also be statistically processed as in the case of wave height (regarding wave force, it is necessary to extract the most severe conditions for the structures), it is typical for the maximum value to be used in experiments with random waves. To calculate the wave force from wave pressure experiments, an adequate width of wave pressure act shall be allocated to each wave pressure gauge, and the wave pressure value at every moment shall be multiplied by the allocated wave pressure width for each wave force evaluation range (e.g., the front side of the breakwater body, the floor plate, the back side of the breakwater body, and the upper side of the breakwater body) to calculate the time series of the wave force for each evaluation range. By statistically processing the time series data, similar to the analysis based on the wave that was previously discussed, the representative values of the wave force shall be calculated.

In experiments related to the sliding of structures, the amount of sliding is defined as the amount of movement from the initial position of the breakwater body when a displacement meter is used. By using the data obtained from a displacement meter, it is possible to capture the time series of the sliding situation, but the analysis is normally conducted based on the total sliding amount after wave attack.

As for the movement of wave-dissipating blocks, the armor units, and foot protection blocks, the movement distance reached to the size of one or a half block or riprap shall be defined as "movement" and it is normal to count movement based on the number of blocks or riprap moved. The number of blocks or riprap counted to move in a certain duration of wave attacks shall be divided by the total number of blocks or riprap in the target region to calculate the damage rate.

③ **Analysis and evaluation of experimental data**

When comparing the measured values of wave force, it is usual to organize and analyze the results using the wave height measured at the time of the inspection of the incident waves. The incident wave component of the wave height measured during the experiment shall be compared to the wave height at the time of the inspection of the incident waves. It shall be confirmed that there is not a significant difference between the two and that the data shall be used.

As for the wave force, in the case of regular waves, the wave force shall be associated with the wave height evaluated as described above. In the case of random waves, the wave force value at the occurrence of the maximum value and the distribution of the wave pressure around the breakwater body shall be determined based on the time series of the entire wave force obtained as described in the preceding section. These wave force and wave pressure values shall be related to the highest wave height to determine the relationship with the wave height. For the present design idea, the maximum value of the wave force shall be evaluated as the wave height corresponding to the highest wave height. However, in the case of random waves in general, the maximum value of the wave force is not necessarily generated by the wave with the highest wave height. Therefore, when the experimental result is organized, the maximum value of the wave force and the highest wave height shall be regarded as representatives of the wave groups, and the wave force shall be evaluated. The data for the wave force of members shall be analyzed and evaluated using the same ideas. As for the wave force, given that the sliding of the breakwater body is the principal failure mode, the wave force index called the equivalent sliding wave force may be used for evaluation. The equivalent sliding wave force is calculated by multiplying the vertical wave force by the friction coefficient of the breakwater body and the mound, as the horizontal contribution of the upward vertical wave force (upward is positive), and the addition of the horizontal wave force (**Fig. 2.10.10**).

As for the entire wave force, the member wave force, the equivalent sliding wave force, the sliding amount, and similar forces, the relationships with the highest wave height shall be investigated. When the damage rate or the stability number is evaluated, the relationships shall be organized using the significant wave height. These relationships shall be compared with the existing design formulas and the design formulas shall be revised if necessary, and used for design verification.

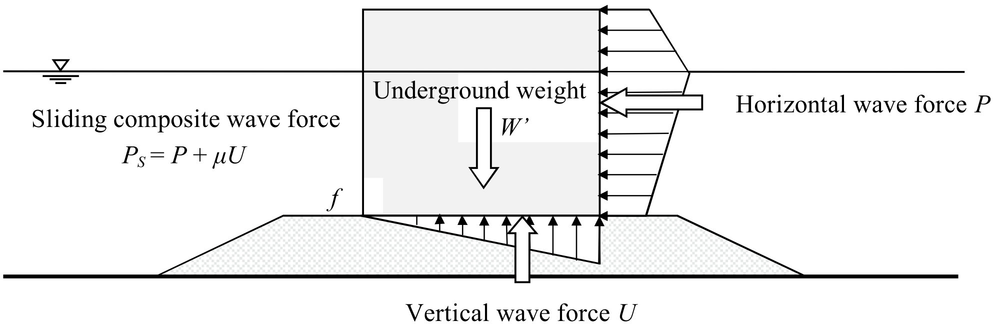


Fig. 2.10.10 Calculation of equivalent sliding wave force

2.10.7 Hydraulic model experiment on the reflection and transmission performance of structures

(1) Reflection and transmission performance

The evaluation standards for the main performance of structures include the reflection and the transmission rate of wave-dissipating structures, including sloping breakwaters and perforated structures, and the transmission rate to the back side by wave-overtopping of breakwaters and similar phenomenon. The evaluation of the performance of these structures is important to achieve harbor calmness and to evaluate the effects of waves on the surrounding ocean areas.

(2) Specifications of wave and tide level used for the experiments

Harbor calmness must be achieved against 97.5 % of the waves that normally arrive. However, wave-dissipating structures that enhance harbor calmness are often constructed in sea areas shielded by breakwaters. Therefore, the conditions that affect cargo handling are the wave conditions, as the target of wave dissipation. However, consider the effects on the surrounding sea areas, the navigation conditions and the departure-limiting conditions of small vessels such as fishing boats, which are most susceptible to waves, become the target. In general, the performance of reflected and transmitted waves with respect to normal waves with the wave height between 1.5 m and 2.5 m, is investigated. Wave-dissipating structures often have a period dependence on wave-dissipating performance. In general structures, the reflection rate tends to increase as the wave steepness becomes smaller. However, they may exhibit period dependence and have higher reflection rates under certain periods according to their structural forms. Therefore, it is desirable to set a relatively wide range of target periods considering the field site conditions and the characteristics of structures.

Given that structures show variable wave-dissipating performance depending on water depth, it is necessary to set the tide level at a certain range centered around the MSL (mean sea level), taking their characteristics into consideration.

(3) Model scale and model production

When the wave-dissipating performance and transmission performance are evaluated, the effects of wave attenuation due to propagation in the wave tank are problematic. Given that the target wave height is relatively low, it is necessary to prepare, so that no effect of the surface tension appears due to dirt on the water surface in the experimental wave tank. However, to reproduce the energy dissipation process to a certain extent, it is necessary to properly evaluate the effects of eddies within the structures. To realize this objective, it is necessary that the model scale that is used for the experiments is as large as possible. Considering this requirement, models with a scale of 1/50 to 1/20 is normally used for the experiments, as in the case of the wave pressure experiments.

Given that the forms of structures are extremely important for the accurate evaluation of wave dissipation and transmission, it is necessary to produce models that are as accurate as possible. In addition, given that observations inside the model breakwater body are required to investigate the mechanism of wave dissipation and transmission, transparent acryl models are used, and measures are acquired so that the structures can be conveniently observed during the experiment. To confirm the wave dissipation and transmission performance, it is necessary to change the specifications of the structure bodies. Therefore, it is necessary to prepare more than one model.

Seabed topography models can be produced by appropriate modeling in the same way as in **Reference (Part I), Chapter 1, 2.10.6 Hydraulic model experiment on wave-resistant stability of structures**. The gap between the sidewall of the tank and the seabed topography model or the experiment model also needs to be filled so that water leaks will not occur as in the case of "Hydraulic model experiment on wave-resistant stability of structures."

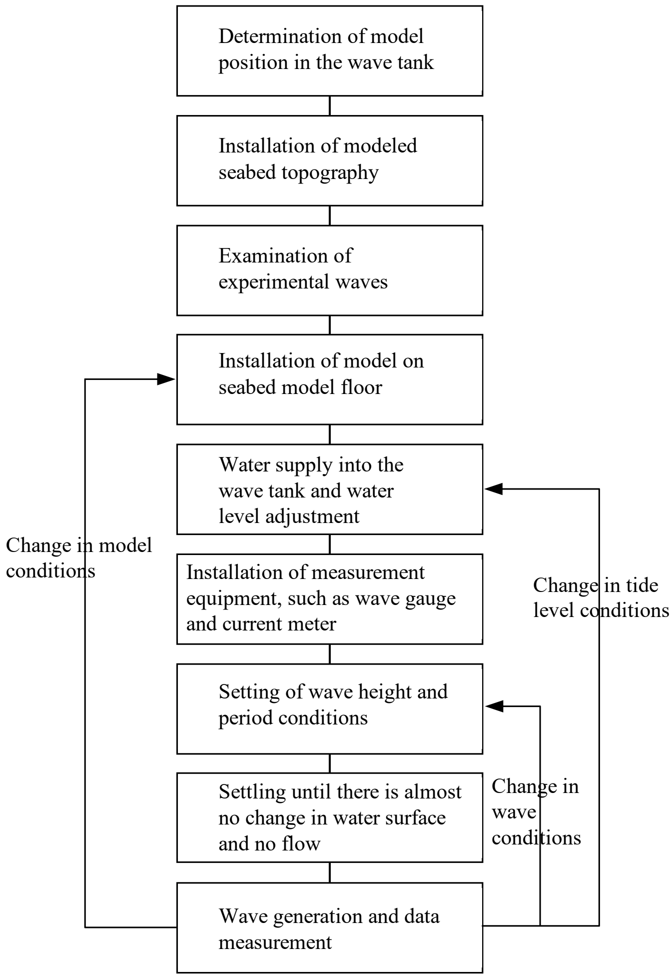


Fig. 2.10.11 Procedures involved in hydraulic model experiments on wave reflection, transmission, and overtopping

(4) Experimental methods

The experiments shall be conducted according to the processes presented in **Fig. 2.10.11**. 1) Determine the position of the model in the wave tank considering the wave action on the structure, the securement of the water area necessary for transmitted waves on the back side to become stable. 2) After installation of the seabed topography, 3) examine the waves and 4) install the structure model. 5) Supply water into the wave tank and set the tide level. 6) Position wave gauges and, if necessary, current meters. 7) Set the wave conditions, and 8) leave the equipment until oscillations in the wave tank settle. 9) Generate waves, and measure the water surface elevation and the velocity fluctuation. Next, change the wave conditions, and repeat procedures 7) to 9). If the experiments for all wave conditions at the same water depth are completed, change the water depth if necessary and repeat procedures 5) to 9). If the specifications of the model need to be changed, install a new model and repeat procedures 4) to 9).

The wave gauges for the separation of the incident waves and the reflected waves shall be positioned in the same manner as in **Reference (Part I), Chapter 1, 2.10.6 Hydraulic model experiment on wave-resistant stability of structures**. In the case that transmitted waves are measured at the back of a breakwater body, a wave gauge shall be installed for measurement at approximately one wavelength apart from the back of the breakwater body because some distance is necessary for the overtopping waves or the flow to become stable.

With respect to the current meters, in the case that the water depth is shallow to a certain extent, given that it is possible to perform measurement of the reflective ratio using a wave gauge placed directly above, a current meter may be used if necessary.

In the case that the mechanisms for the generation of wave dissipation or transmitted waves shall also be investigated, the fluctuation in the water surface or velocity inside the structures may be measured if necessary. **Fig. 2.10.12** presents an example of an experiment on reflected waves and transmitted waves.

(a) Experiment on the front side of a sloping bank (b) Experiment on the back side of a sloping bank

Fig. 2.10.12 Example of experiment on reflected waves and transmitted waves

(5) Organization of experimental data

① **Primary processing of experimental data**

Given that only the wave data shall be organized in principle, the data shall be processed in the same way as in **Reference (Part I), Chapter 1, 2.10.6 Hydraulic model experiment on wave-resistant stability of structures**.

② **Analysis of experimental data**

Similar to the approach in "Hydraulic model experiment on wave-resistant stability of structures," the zero-up-crossing method and related methods shall be applied to the water surface elevation data and the velocity data to obtain the wave height of individual waves, the wave amplitude, the water velocity amplitude, and other related parameters. In addition, the data are statistically processed so that the mean wave height shall be defined. In the case of regular waves, the data shall be averaged over the stable period from the duration up to the time when re-reflected waves from the wave-generating paddle return to obtain the average wave height, and its corresponding period. In the case of random waves, analysis based on the zero-up-crossing method of the water surface elevation shall be conducted, and the significant wave height and wave period obtained from the statistic values of wave height shall be used to organize the data.

With respect to the calculation of the reflection rates, the incident/reflected wave separation that was estimated using the method by Goda shall be used to obtain the representative reflection rate of the wave group. As for the transmission rate, the significant wave height shall be obtained from the statistical processing of the water surface fluctuation of transmitted waves, and the transmission rate shall be defined based on the ratio with respect to the incident wave height used for the examination of experimental waves.

③ **Analysis and evaluation of experimental data**

As for the reflection rate and the transmission rate obtained via the experiments, the structure scales that contribute to the characteristics of reflected and transmitted waves or the wave-dissipating characteristics of structures such as the ratio of the length that represents the structure relative to the wavelength, the relative water depth, the relative breakwater body width, and parameters based on various values corresponding to wave periods such as wavelength, shall be used to study the relationship between the reflection rate and the transmission rate. Moreover, the characteristics of reflected and transmitted waves are evaluated.

2.10.8 Hydraulic model experiment on countermeasures against wave-overtopping of structures

(1) Performance to prevent wave-overtopping

Concerning revetments and breast walls, reducing damage by waves in the hinterland and securing safety are important functions that are required of structures. In particular, wave-overtopping hinders human activities and the passage of vehicles in the hinterland. Furthermore, an increase in the flow rate of wave-overtopping affects stability in the hinterland, and this may lead to the destruction of the structure. The wave-overtopping phenomenon of a structure is affected by the cross-sectional shape and the wave-dissipating characteristics of the structure. The characteristics of waves that act on a structure, including incident wave characteristics and the effects of seabed topography, also affect this phenomenon. Therefore, although Goda et al. have presented a calculation chart, it may be necessary to perform hydraulic model experiments depending on the cross-sectional shape of the structure and the characteristics of the incident waves. In the case that the intrusion of water into the site due to wave-overtopping is prevented or a certain amount of wave-overtopping is allowed, a wave-overtopping drainage canal may be constructed at the back of the structure. In that case, given that evaluation of the time variation characteristics of wave-overtopping discharge such as short-term wave-overtopping discharge is required to design a wave-overtopping drainage canal, it is often necessary to conduct hydraulic model experiments. As such, hydraulic model experiments on countermeasures against wave-overtopping of structures are conducted with the objective of assessing the flow rate of wave-overtopping of revetments, quay walls, and coast protection structures. In addition, they contribute to the determination of the crown height for securing safety in the hinterland of the structures, and in the design of wave-overtopping drainage canals.

(2) Specifications of waves and tide levels used in the experiments

In hydraulic model experiments on the prevention of wave-overtopping, waves as high as the design waves are assumed, and the experiments are conducted using wave heights set at approximately this value. As for the tide level, the high water level generally becomes critical because the parameters that control wave-overtopping are related not only to the water depth and the wave height, but also to the crown height of the structure (the height from the still water surface to the crown height). Therefore, it is often the case that experiments are conducted with the tide level as an experimental parameter that is changed around the high water level condition.

(3) Model scale and model production

In the case that the crown height of the structure is relatively high and the wave-overtopping discharge is relatively small, the effects of water viscosity and surface tension appear when wave-overtopping occurs. Therefore, the model scale should be as large as possible.

In the wave-overtopping experiments, the crown height of the target structure is the most important parameter, and the relative crown height to the wave height is also important. The factors that determine the crown height are the height of the structure and the tide level, and it is necessary to produce models while paying close attention to these parameters. Given that the reproduction of the wave height at the front side of the structures against wave-overtopping such as revetments is important, the seabed topography must also be appropriately modeled.

(4) Experimental method

The experiments shall be conducted using the same procedure as in **Reference (Part I), Chapter 1, 2.10.7 Hydraulic model experiment on the reflection and transmission performance of structures**, with the addition of equipment to measure wave-overtopping discharge.

For the evaluation of the extent of wave-overtopping, random waves are normally used. Wave-overtopping is extremely susceptible to nonlinearity of the incident wave height or the mean water level, including the tide level. Furthermore, wave-overtopping occurs selectively according to the wave height value. Therefore, it is not possible to achieve a complete understanding of wave-overtopping by evaluating this phenomenon using regular waves and applying linear superposition to the results.

One method that is used to measure the wave-overtopping discharge is to place a container (hereinafter referred to as wave-overtopping measure) at the back of the wall body to collect water from wave-overtopping and to measure the increased weight of the wave-overtopping measure at the end of the experiment. Another method is to measure the wave-overtopping amount from the water level in the wave-overtopping measure using equipment to determine the water level, such as a capacitance-type wave gauge. Netted fiber may be used to dissipate waves in the wave-overtopping measure so that water does not spill from this container due to the energy of wave-overtopping when water flows inward. The measured value for the wave-overtopping discharge shall be divided by the wave action time to calculate the wave-overtopping discharge.

For the evaluation of the short-term wave-overtopping discharge, the temporal changes in the wave-overtopping discharge must be obtained. If the time series of the water surface fluctuation in the wave-overtopping measure can be accurately determined, the temporal changes of the wave-overtopping discharge can be calculated. However, given that the water level in the wave-overtopping measure oscillates when wave-overtopping water flows inwards, it is necessary to suppress oscillations in the wave-overtopping measure. It is also possible to obtain the temporal changes associated with the wave-overtopping discharge by supporting the wave-overtopping measure with a load cell, and measuring the weight of the wave-overtopping measure over time. However, if the amount of wave-overtopping is large, the wave-overtopping measure must also be large, and suitable approaches for addressing the situation must be determined. Sekimoto et al. [101] installed an rectangular weir at the opposite side from the end where the wave-overtopping flows inward, and calculated the short-term wave-overtopping discharge by determining the weight of the wave-overtopping measure and the outflow rate from this container In this method, the accuracy of the estimate of the outflow rate from a sharp-crested weir is affected. Therefore, by introducing a headrace channel, with the width of approximately 1/3 of the experimental flume to the front side of the wave-overtopping measure, the inflow volume into the wave-overtopping measure by wave-overtopping can be suppressed and the required capacity of the wave-overtopping measure can be decreased. Consequently, a wave-overtopping dam is not necessary, and the measurement system becomes simple. As a result, it is possible to improve the accuracy of the measurement (**Fig. 2.10.13**).

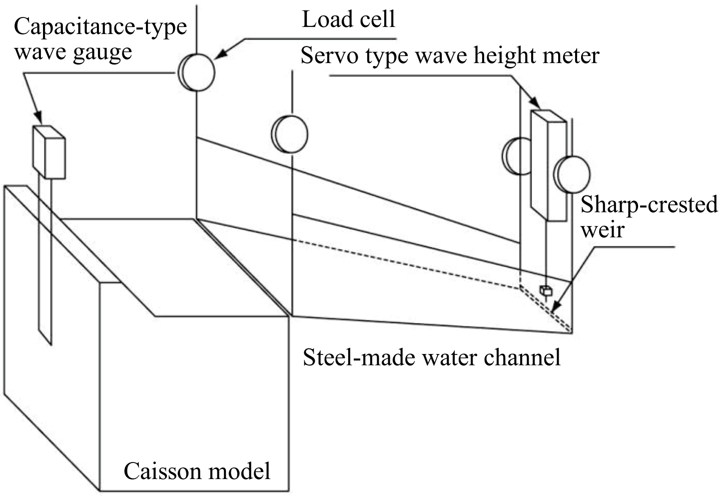


Fig. 2.10.13 Equipment to measure temporal changes of wave-overtopping [101]

(5) Organization of experimental data

The wave-overtopping discharge shall be evaluated as means of the amount of wave-overtopping by individual waves, and calculated as the rate of wave-overtopping average over the duration of the storm waves. The obtained wave-overtopping discharge shall be made to be dimensionless using the gravitational acceleration and the significant wave height at the front side of the wall body, and should be organized as the dimensionless wave-overtopping discharge. The explanatory parameters of the wave-overtopping discharge are the significant wave height at the front side of the structures such as revetments, the water depth and wavelength, the crown height defined as the height from the still water surface to the crown height of the superstructure, and the seabed slope. At present, it is typical for the data to be organized using these parameters. They were originally used in the equations to calculate the wave-overtopping discharge of sloping revetments shown in the wave-overtopping amount evaluation manual used in Europe [102]. Subsequently, Goda [95] reorganized a database called CLASH that was used to establish the wave-overtopping discharge evaluation manual and proposed equations to evaluate the wave-overtopping discharge of upright revetments using the aforementioned parameters.

In the case that the data is compared with the wave-overtopping calculation chart by Goda [103], the dimensionless wave-overtopping discharge shall be organized using the equivalent deep-water wave height. The equivalent deep water wave height shall be calculated by dividing the significant wave height in the area with uniform water depth obtained in the examination of experiment waves by the shoaling coefficient. The explanatory parameters are the water depth equivalent deep water wave height ratio at the position in the front side of the wall body, the relative crown height that is dimensionless due to the equivalent deep water wave, and the equivalent deep water wave steepness.

Finally, the allowable wave-overtopping discharge shall be set according to the performance required for the revetment, and it shall be confirmed by experiments whether or not the wave-overtopping discharge is controlled within the allowable value when the design wave attacks the revetment whose crown height set based on the preceding discussion.

2.10.9 Hydraulic model experiment on tsunami-resistant performance of structures

(1) Tsunami-resistant performance

The Great East Japan earthquake caused unexpectedly high tsunami waves, and numerous harbor structures were damaged. With regard to facilities such as tsunami protection breakwaters, the stability and destruction characteristics of structures against tsunamis have been clarified [104), 105)]. It is required that safety shall be secured against tsunamis that occur relatively frequently by tsunami-resistant structures. In addition, it is also required that safe evacuation shall be achieved by suppressing propagation of tsunami, to the back [106)] using tsunami-resistant structures with resiliency, with respect to tsunamis that occur extremely rarely. Hydraulic model experiments on tsunami-resistant performance of structures are conducted with the objective of evaluating the performance of structures for the development of tsunami protection countermeasures.

(2) Specifications of tsunamis and tide levels used for experiments

With respect to the tsunamis investigated in the experiments, the conditions at the incident position in the hydraulic model experiment shall be made clear using numerical simulation of the tsunami from the hypocenter corresponding to the assumed earthquake motion, and the height and duration of the tsunami shall be determined. Tsunamis that correspond to Level 1 earthquake motion shall be prepared for the stability limit of tsunami-resistant structures and tsunamis corresponding to Level 2 earthquake motion shall be prepared for the conditions that exceed the stability limit.

As for the tide levels, the HWL (mean monthly highest water level) or the HHWL (highest water level on record), which is assumed to be the most dangerous situation for the structure, is set.

(3) Model scale and model production

Although large-scale experiments are preferable for tsunami experiments, given that tsunamis themselves have a high wave height and long wavelength, the scale must be determined based on the conditions in which these waves can be generated in the available wave flume. Models can also be scaled down based on the Froude similarity rule in the tsunami experiments.

Given that tsunamis have long wavelengths, they are easily affected by changes in seabed topography. However, as they are hardly affected by microtopography, seabed slopes that are averaged to some extent, may be used for tsunami experiments.

(4) Experimental method

The basic flow of the tsunami experiments including the layout of the measurement equipment is the same as in **Reference (Part I), Chapter 1, 2.10.6 Hydraulic model experiment on wave-resistant stability of structures**. The wave generation of tsunamis can be reproduced by steady flows because they can be regarded as flows. To reproduce wave profile of bore type tsunamis, it is necessary to allow a large amount of water to flow within a short period of time from the start of the wave generation process. Therefore, it is necessary to store water in the head tank and to allow the prescribed amount of water to flow at once, similar to a dam break. By preparing a system in which water can be supplied to the head tank continuously, tsunamis with a long duration can be generated in the wave tank. It should be noted that not only the capacity to supply water into the wave tank is needed, but the capacity to drain water out of the tank is also required because all wave generation methods use a large amount of water to conduct experiments.

Surveys and research on the tsunami damage of the Great East Japan earthquake revealed the failure modes of tsunami protection breakwaters and related parameters. The major failure modes are considered to be the sliding of the structure body due to the increase in pressure caused by the difference between the water level in front of the structure body and the water level behind the structure body, the failure of the bearing capacity of the mound due to an increase in the toe pressure, and scouring of the mound by overtopping flow. With respect to tide protection facilities such as coastal banks, the major failure mode for a structure body covered with blocks and similar situations is the failure of the armor units behind the structure body due to the occurrence of negative pressure that accompany the strong flow behind the structure body at the time of overtopping flow. The major failure mode for an uncovered structure is the scoring at the back due to overtopping flow. It is important that individual failure modes should be reproducible for individual structures in the experiments.

Considering data measurement that is identical to that described in **Reference (Part I), Chapter 1, 2.10.6 Hydraulic model experiment on wave-resistant stability of structures**, it is important to understand the failure phenomenon by video recordings based on the experiments, as well as to measure physical quantities with wave gauges, current meters, wave pressure gauges, and similar instrumentation.

(5) Organization of experimental data

In the analysis of experimental data, physical quantities, such as water surface fluctuation, velocity change, wave pressure, and wave force, shall be evaluated on a time-series basis. As a result, the maximum values within the data shall be extracted, as well as the physical quantities at the time of structural failure. In addition, the failure can be analyzed in detail by organizing the temporal changes that event.

Using the analyzed experimental data, the relationship between the difference in the water levels of the front and back of the structure body and the stability of the structure body, in addition to the mound and the relationship between the overtopping discharge and the amount and the stability of the mound, shall be investigated for tsunami-resistant structures, such as tsunami protection breakwaters. As for tide protection facilities, the relationship between the pressure acting on the armor units or overtopping discharge, and the stability of the structure body shall be investigated. In addition, it is necessary to obtain the time required for the evacuation by examining the period of time from the initial interaction between the tsunami and the structure, and the time until failure.

2.10.10 Hydraulic model experiment on plane wave field

(1) Plane-wave field

Depending on the effects of the topography around a structure or the behavior of waves that accompany the plane arrangement form of a structure, the stability of the structure may be reduced compared to the case of a two-dimensional cross-section. The problem includes, for example, the wave concentration or dispersion that accompany wave refraction, wave concentration or dispersion due to wave diffraction, increase in local wave height due to diffraction. This also includes the dispersion of waves including reflected waves from structures and the propagation of a wave height to shielded areas due to directional spreading of multi-directional wave field. It is already possible to numerically analyze the propagation of wave height to shielded areas due to directional spreading of multi-directional wave or wave height concentration due to refraction. However, in the case of a complicated topography or when wave-breaking occurs due to wave concentration, the examination by numerical analysis is insufficient. In these cases, plane experiments are necessary. The target phenomena are as follows: stability of breakwater heads, stability, and wave-overtopping at inward corners, stability of detached breakwaters, stability of structures on topography where wave concentration due to refraction occurs significantly, and etc.

With respect to the stability of breakwater heads, a sloping breakwater or a breakwater covered with wave-dissipating blocks may not have support at the back for covering stones or the wave-dissipating blocks of the sloping breakwater, depending on the incident direction of the waves. Therefore, the stability could be significantly reduced compared to normal covering stones or wave-dissipating blocks.

An increase in wave height due to wave concentration is observed at the inward corners of a breakwater. In some cases, the wave-breaking phenomenon occurs accompanied by an increase in the wave height. As a result, a decrease in the stability of wave-dissipating blocks or an increase in the wave-overtopping discharge is observed.

Regarding the stability of detached breakwaters, the wave height may increase locally, affected by diffraction and scattering of the detached breakwater. In some cases, the stability of the breakwater body or wave-dissipating blocks decreases, and the wave-overtopping increases.

Plane experiments can deal with multi-directional irregularity of waves. In wave-overtopping at revetments, it is known that the wave-overtopping discharge decreases due to the multi-directionality of the waves [107]. The multi-directionality of waves may need to be considered in order to conduct precise experiments.

In some cases, the multi-directionality of waves may allow the design conditions to be more relaxed compared to the case of uni-directional waves, and it may become possible to design in a rational manner. It is sometimes come across difficulties to determine if the design conditions should be relaxed or not, considering the multi-directionality of the waves when designing structures. Given that it can be confirmed that the design conditions can be relaxed reliably by hydraulic model experiments etc., it is considered worthy of consideration.

(2) Specifications of waves and tide levels used for the experiments

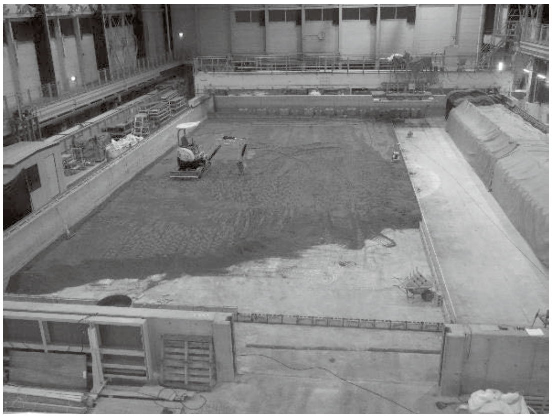
The wave height and the tide level for experiments may be appropriately set by considering the field site conditions in the same way as in various cross-section experiments. In the case when multi-directional random waves are used, it is necessary to conduct the wave transformation analysis based on calculations using multi-directional waves from the offshore area. The input conditions at the wave generation positions, especially information on the directional wave spectrum, must be determined.

(3) Model scale and model production

In plane experiments, the model scale shall be determined based on the target structures and holding facilities. It should be noted that if the scale is small, the actual phenomena may not be reproduced in the wave tank due to the scale effect. It is necessary to conduct experiments using a model scale that is as large as possible for wave generation equipment.

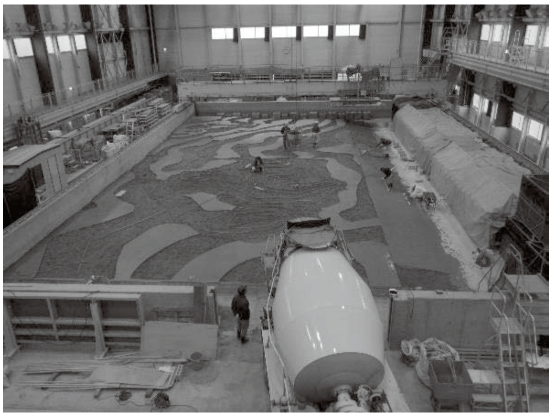
Models are produced by scaling down based on the Froude similarity rule. In producing and reproducing the field site topography, the reproduction area of the structure and the installation position shall be determined by considering the effective wave generation domain and the wave direction corresponding to the generated waves.

To reproduce the field site topography in the wave tank, the ground base shall first be produced by piling and compacting sand with reference to the nautical chart and the like. Then, based on information such as the nautical chart, the positions of the seabed contour lines shall be determined, and small wooden stakes shall be driven along the contour lines at appropriate intervals. The head portion of the wooden stakes shall be adjusted to be at the specific height of the contour lines. These intervals between the wooden stakes shall be connected using wooden plates that serve as forms for the mortar so that the top matches the contour line elevation. After the contour lines are produced in the wave tank in this way, the gaps between plates that describe the contour lines with sand up to approximately 3 cm under the contour lines are filled and the sand is compacted. Mortar shall be poured between the remaining contour lines, and the surface shall be smoothed using a trowel to avoid unevenness. Attention must be paid to drying shrinkage of the mortar when poured. When the mortar dries, the contour lines shall be marked, and the production of topography is completed (**Fig. 2.10.14**).

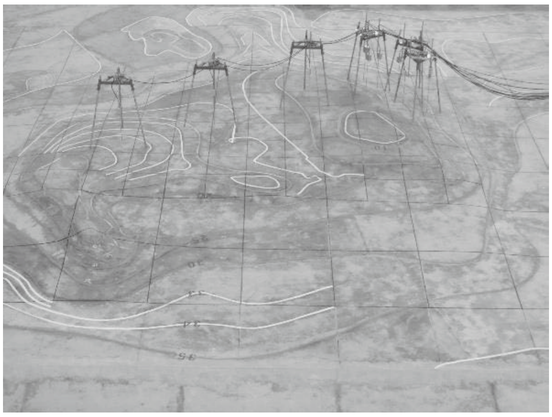


(a) Production of ground base by sand piling and compacting

(b) Placement of forms with the height matching contour lines



(c) Mortar pouring



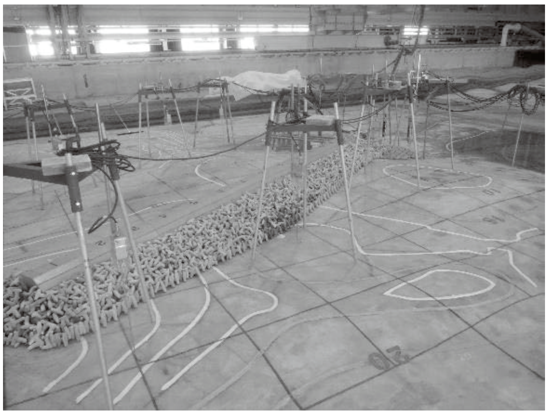
(d) Completed seabed topography

Fig. 2.10.14 Production of the model floor in a plane experiment

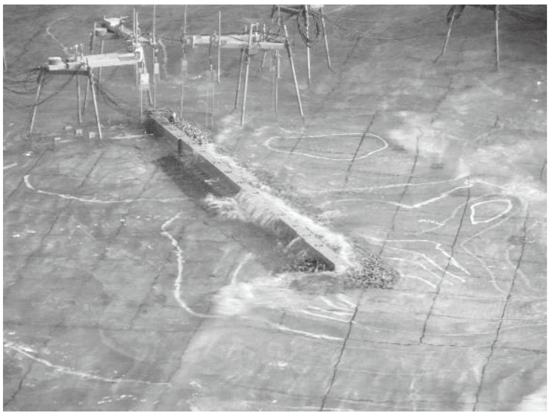
(4) Experimental method

The experiment processes in the plane experiments are basically the same as those in the two-dimensional cross-section experiments. However, in experiments with multi-directional random waves, it is necessary to conduct an examination of the experiment waves, including whether or not the target directional wave spectrum is reproduced in the wave tank. First, the amount of wave motion, such as the water surface elevation or the velocity fluctuation shall be measured using an array of wave gauges (star-shaped array, straight-line array, etc.) or an array of wave gauges in combination with current meters in the uniform water depth area in the front side of the wave generation paddle. Then, the directional wave spectrum shall be estimated using the maximum likelihood method [108], the maximum entropy method [109], or Bayes' type model [110]. The estimated directional wave spectrum shall be compared to the target directional wave spectrum, and the wave generation signal shall be revised. These processes shall be repeated to reproduce the directional wave spectrum at the prescribed accuracy. After matching the spectrum form to the target value, the wave height level shall be adjusted in the position of model installation.

In addition, in plane experiments, it is possible to control the absorption of reflected waves in multi-directional random wave generation studies. This function may be utilized in random waves and similar experiments. With respect to waves propagation outside of the effective wave generation area, wave-dissipating material such as netted fiber, shall be appropriately placed to minimize the effects of reflected. Netted fiber with different mesh sizes is available on the market, and it is possible to effectively dissipate waves by placing fiber with a coarse mesh-size at the entrance of the waves, and using fibers with finer a mesh size as the waves propagate. Given that the reflection characteristics in a wave tank vary according to the installed model form, trial and error is required to develop effective countermeasures against reflected waves. An example of the setup of a plane experiment is presented in **Fig. 2.10.15**.



(a) Installation of seabed topography, model, and wave height meter



(b) Experiment situation

Fig. 2.10.15 Example of setup for a plane experiment

(5) Organization of experimental data

The processes involved in analyzing the experimental data are also basically the same as in the case of cross-section two-dimension experiments. However, in the case that the reflection rate of the structure is evaluated, it is necessary to conduct the directional wave spectrum analysis by considering the phase relationship between the incident waves and the reflected waves using the extended maximum likelihood method [111], the extended maximum entropy method [112], and the extended Bayes' type model [113]. In this case, it is necessary to place at least one wave gauge out of the array of gauges within the position of 0.2 wavelength from the structure [111].

In addition, with respect to the aim of plane model experiments, it is necessary to organize data with respect to the spatial distribution of various physical quantities, such as the wave height around structures.

(6) Points to be noted in plane experiments

One-directional random waves or multi-directional random waves are used in plane experiments. Given that the wave-generating paddle is finite, the wave energy spreads around due to diffraction. The attenuation of energy is relatively small in the domain of approximately 5º inside, from both ends of the wave-generating paddle. This domain is called the effective wave-generating domain. The domain changes according to the directional spreading of the component waves that constitute multi-directional random waves. The smaller degree of the directional energy concentration, the wider the wave direction angles of component waves and the narrower the range that wave energy can reach as a multi-directional random waves (the vertex of a triangle with the wave-generating plane as the base approaches this plane). As a result, the effective wave-generating domain decreases. Given that the model of a structure in which wave effects must be considered, construction must be within the effective wave-generating domain, and it is necessary to determine the model scale considering these effects. With respect to the wave generator of multi-directional random waves obtained by placing two wave-generating paddles in the shape of an "L", it has been reported that it is possible to conduct plane experiments considering almost the entire area of a rectangular wave tank with these two plates as two sides, as an effective wave-generating domain depending on the main wave direction [114)].

The finiteness of wave-generating paddles leads to scattered waves from the ends of the wave-generating paddles. As a result, a change in wave height occurs along the wave crest line, and it becomes impossible to generate waves with a spatially uniform wave height.

To avoid this spatial fluctuation of wave height, it is possible to adjust the wave generation efficiency so that the movement of several wave-generating paddles from the ends gradually reaches 100 %. Although the phenomenon can be relaxed in this measure, the effective wave-generating domain would be narrower. In the case that unidirectional random waves propagate in the uniform water depth along the coast, it is possible to suppress the diffraction of the energy and scattering from the end points by introducing the waveguide plates. However, in the case of experiments using structures, it is necessary to note that multiple reflections occur in the domain inside the waveguide plates.

2.10.11 Movable bed experiment

Plane movable bed experiments on beach deformation will be described because the coast is positioned as a part of the harbor facilities, and their required performance for the coast is specified [115), 116)].

As previously indicated, it is extremely difficult to set the similarity rule appropriately in movable bed experiments and to perform quantitative evaluation. However, given that sufficient understanding of the actual conditions and characteristics of the topographic changes in the target coasts, and analyzing and evaluating the obtained results, it is possible to ① qualitatively predict changes in topography regarding the effects of harbor structures on the coast, ② compare and study the effects of shore protection facilities such as jetties and detached breakwaters, and ③ qualitatively predict the shoreline changes on the coasts behind large-scale offshore facilities. Therefore, it is often the case that a comprehensive investigation on beach deformation is conducted based on various information that utilize field observations, numerical simulations, and movable bed experiments.

(1) Organization of field site conditions

① Organization of topography condition

Concerning topography of target site, the result of the bathymetric survey shall be obtained and analyzed. In general, when the shoreline sets back after storm waves attack, the seabed slope becomes steeper, and the sand bar moves offshore direction. However, during a calm period, as the sand that has moved to the offshore side moves to the onshore direction, the shoreline advances, and the seabed slope become mild. On a stable coast, the sediment transport in the on-offshore direction is often negligible if averaged over a year. Although it is desirable to obtain bathymetric survey data for as long a period as possible, if it is difficult to obtain long term data, the data surveyed in the most stable conditions at the target locations shall be selected from the existing information.

② Organization of wave conditions [117)]

Wave conditions shall be organized appropriately based on the measured record of waves at the observation stations. In the case that the wave observation stations are far from the target locations of the investigation, wave refraction and wave shoaling calculation shall be performed if necessary, and the incident waves for the target locations will be calculated. Considering waves related to littoral drift, in the case that long-time topography changes shall be investigated, energy equivalent waves are used. In the case that short-term topography changes shall be investigated, the waves that arrive several times per year shall often be used. The period of energy equivalent wave T_m, wave height H_m, and wave direction α_m shall be calculated using **equations (2.10.3)** to **(2.10.5)**.

$$T_m = \frac{1}{N}\Sigma T_i \tag{2.10.3}$$

$$H_m = \left[\frac{1}{NT_m}\Sigma\left(H_i^2 T_i\right)\right]^{1/2} \tag{2.10.4}$$

$$\alpha_m = \frac{1}{2}\sin^{-1}\left\{\frac{2\Sigma\left(H_i^2 T_i \cos\alpha_i \sin\alpha_i\right)}{NH_m^2 T_m}\right\} \tag{2.10.5}$$

where

H_i : Observed significant wave height (m)

T_i : Significant wave period (s)

α_i : Wave direction

N : Number of waves

The waves that arrive several times per year shall be commonly calculated using the following methods.

- Mean wave height and mean period of 5 waves with 5 highest wave height in a year
- Wave height corresponding to the 99 % accumulated appearance rate in incursion frequency by wave height class and the most frequent occurrence wave period corresponding to the wave height
- 1-year probabilistic wave height and the most frequent occurrence wave period corresponding to the wave height

Wave periods can be obtained from the correlation between wave height and wave period if it is available.

As for the tide level, the mean tide level shall be generally used when long-time topography changes shall be investigated, and the mean monthly highest water level shall be used when short-term topography changes shall be investigated. Thus, the tide level measurement data at a tide level observation station in the neighborhood shall be referred to and organized.

③ **Survey of the sediment**

In movable bed experiments, the selection of the sediment is important. For this purpose, it is necessary to understand the distribution of the grain size and the unit volume mass of sediment, in the alongshore and in the on-offshore direction.

(2) Setting of experimental area

The experimental area can be determined based on the capes or structures that border the littoral drift. However, in the case that it is inevitable to cut out part of the landform in relation to the experimental scale, it is necessary to conduct experiments by carefully considering the differences between the actual field and the experimental model.

(3) Selection of model scale

The model scale shall be determined by the Froude similarity rule based on the target topography, and the experimental equipment scale. Distorted models are not normally used. For the experimental scale determined in this way, the similarity rule cannot be perfectly satisfied with respect to the bottom sediment and the sediment movement.

(4) Specifications of external force and bottom sediment particle size used for experiments

The external forces used during the experiments shall be determined using the Froude similarity rule for field site wave conditions that are collected and organized. As already mentioned, in the case that long-time topography changes shall be investigated, the mean water level shall be used as the tide level, and the energy equivalent waves shall be used as the waves. In the case that short-term topography changes shall be investigated, the mean monthly highest water level shall be used as the tide level, and the waves that arrive several times per year shall be used as the waves. However, for the particle size of the bottom sediment that constitutes a coast, if the Froude similarity rule is followed, materials such as silt and cohesive soil shall be adopted, and the conditions may become different from the characteristics of sandy soil. Even in sandy soil, soil with extremely small particle sizes has a lower settling velocity that is affected by the viscosity of water. Therefore, it is necessary that the minimum particle size of the sediment used for experiments shall be approximately 0.1 mm or above. In movable bed experiments, given that the experiment scale is smaller, the littoral drift phenomenon may not be adequately evaluated with the available bottom sediment. In this case, the external force conditions and the particle size of the sediment shall be selected considering the similarity with the Shields number or the settling velocity of the sediment as the external force. In addition, cross-section movable bed experiments on the relationship between the external force and the sediment motion shall be conducted as preliminary experiments if necessary, and the external force and the particle size of the sediment shall be selected.

(5) Production of structure models and model floors

Structure models shall be scaled down based on the Froude similarity rule and produced. The model sea bed shall determine by the approximating field site topography based on the result of the bathymetric survey. In movable bed experiments, given that significant effort is required to form the initial topography, a sea bed with a uniform slope based on the representative seabed slope is frequently used as the initial topography. After forming the model sea bed, structure models shall be placed, and the experiments shall be conducted.

(6) Experimental case

An experiment to confirm the current situation shall be conducted to initially confirm the validity of the movable bed experiment and to provide the control data for comparison to the later experiments to predict the future. Then, the experiments to predict the future by reproducing the future form of structures shall be planned and conducted. To improve the accuracy of prediction experiments and to achieve more useful results, it is important to reproduce the current situation. Therefore, trial and error investigations, including the setting of the external force conditions, may be needed.

(7) Experimental method

First, the topographic survey in wave tank shall be conducted with respect to the initial topography. In general, the survey lines are set in the longitudinal direction, and the topography along each survey line shall be measured with an ultrasonic sand surface meter.

In the experiments, the measurement equipment shall be appropriately placed to monitor the plane wave field and nearshore current field. Given that the time scale of littoral drift depends on the magnitude of the drift, it is necessary to prolong the final wave duration if the amount of littoral drift is large. Finally, wave actions shall be applied until the topography is stable to some extent, and the experiment is completed. To examine the changes in topography that occur during the experiments, wave generation shall be temporally stopped at appropriately set time intervals, and the topography shall be measured using the same procedures as the initial topography survey.

(8) Organization and evaluation of experimental results

The data on wave and velocity fields shall be organized, and wave transformation and flow patterns shall be examined. If necessary, the data shall be compared to the numerical simulation result for the wave transformation and nearshore current. Based on the calculation result for the nearshore current, the spatial distributions of indexes related to littoral drift, such as Shields number, shall also be calculated.

The extent of the changes in the seabed topography obtained using the topography survey shall be calculated, including the data obtained during the experiments. The changes are examined in addition to the relationship with the indexes of the external forces and the littoral drift is evaluated as indicated in the preceding section.

The quantitative evaluation of movable bed experiments is difficult, and the experimental result shall be analyzed based on a comparison to the field survey and numerical simulation of beach deformation. It is necessary to conduct a final evaluation, considering the accuracy of the field surveys and the numerical simulations, in addition to the accuracy of the movable bed experiments.

2.11 Numerical Analysis Concerning Performance Verification of Structures

2.11.1 General

(1) Purpose

Although the performance verification of structures has been mainly performed by a hydraulic model experiment until now, the recent advancements in computer technology have enabled the use of numerical analysis for evaluating phenomena such as the interaction between waves and structures, which could be evaluated before only with a hydraulic model experiment. Nevertheless, given that a certain modeling approach accompanies the discretization for digital analysis, a modeling method that uses the result of a hydraulic model experiment or other experiments needs to be validated.

(2) Analysis flowchart

The flowchart of the general numerical analysis, including the preparation, is shown below:

① Setting of questions (if possible, include the exploration of the target port or shore)

② Collection and arrangement of the weather, oceanographical phenomena, seabed topography, and relevant data

③ Setting of the field condition (e.g., tide level, waves, and seabed topography)

④ Selection of the calculation zone and target structures or structure arrangement plans

⑤ Selection of a calculation method

⑥ Setting of a calculation case

⑦ Setting of a calculation parameter or boundary conditions

⑧ Setting of a computational grid and a computation time step

⑨ Modeling and discretization of structures

⑩ Execution of numerical calculation

⑪ Analysis of numerical calculation data

⑫ Consideration of the calculation result and implementation of additional calculations

⑬ Creation and presentation of a report

(3) Systematic arrangement of numerical analysis

Fig. 2.11.1 shows the result of a systematic arrangement of wave propagation and transformation calculation methods.

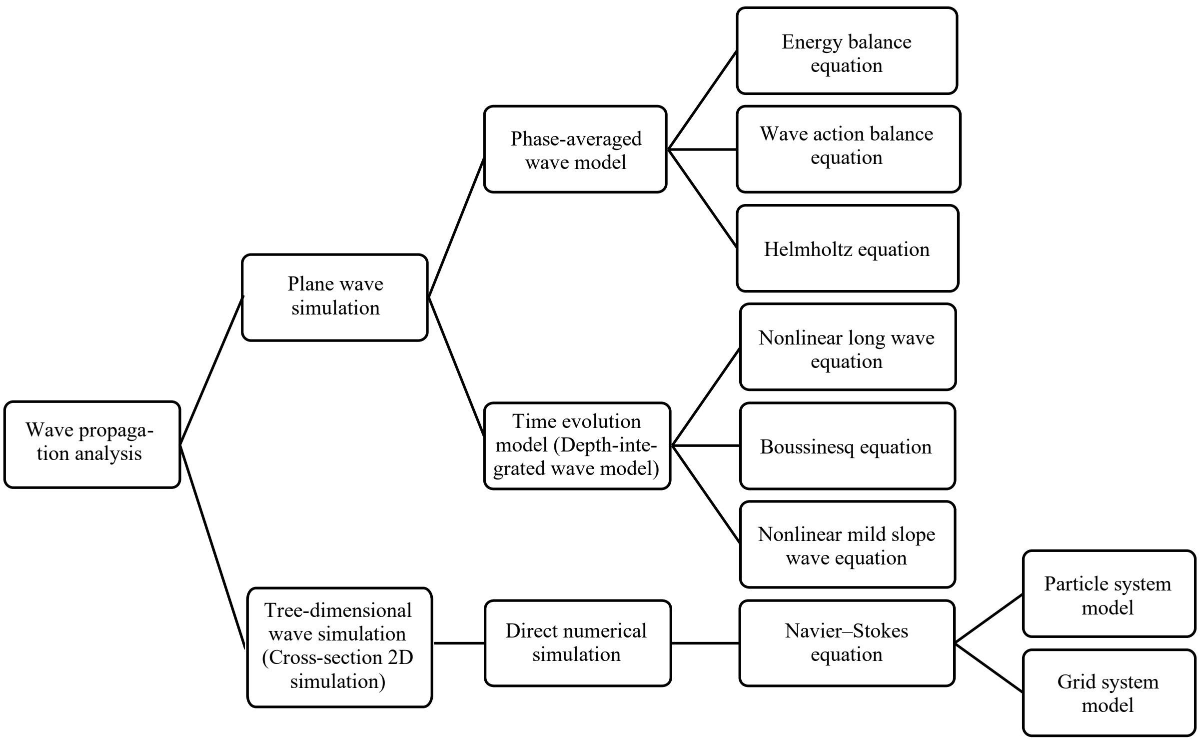


Fig. 2.11.1 Systematic Arrangement of Numerical Analysis Technology for a Wave Field

The wave propagation and transformation models are roughly classified into two models namely; a model used for plane wave field analysis and the wave field analysis model used for 3D analysis. Some models used for 3D analysis are currently applied only to two dimensions by considering their calculation load.

The plane wave field analysis is divided into the phase-averaged wave model, which handles the wave height value or wave energy as a function, and the time evolution model (or depth-integrated wave mode). The phase-averaged wave model is further divided into the energy balance equation or the wave action balance equation, which targets refraction; the Helmholtz equation, which targets diffraction; and the mild slope equation, which targets both. The phase-averaged wave model is an analysis method that targets linear waves. The time evolution model is classified into the nonlinear long wave equation, which is used for tsunami analysis; the Boussinesq equation; and the nonlinear mild slope wave equation.

The 3D wave field analysis is a method used for directly solving the Navier–Stokes equation, and can be classified into the particle system model and the continuum system model.

① **Phase-averaged wave model** [118) 119)]

The energy balance equation and wave action balance equation are the models that can handle the wave refraction and wave shoaling with the method and the space change in wave energy and wave action amount, respectively. The wave action balance equation is used when considering the influence of the flow on waves.

The wave action amount N is defined as the wave energy E divided by the angular frequency σ. When S is the energy source showing the inflow and outflow of energy, the wave action balance equation is expressed as follows:

$$\frac{\partial N}{\partial t}+C'_{gx}\frac{\partial N}{\partial x}+C'_{gy}\frac{\partial N}{\partial y}+C'_{g\sigma}\frac{\partial N}{\partial \sigma}+C'_{g\theta}\frac{\partial N}{\partial \theta}= S/\sigma \tag{2.11.1}$$

where C'_{gx} and C'_{gy} express the group velocity in the x- and y-directions, respectively; $C'_{g\sigma}$ is the rate of change in the angular frequency accompanying the propagation of the wave energy resulting from the wave refraction or flow; and $C'_{g\theta}$ is the rate of change in the wave direction angle accompanying the propagation of the wave energy resulting from wave refraction. This group velocity considers the Doppler shift by flow. When there is no flow, the term concerning the angular frequency change can be ignored and the equation can be modified as follows:

$$\frac{\partial E}{\partial t}+C_{gx}\frac{\partial E}{\partial x}+C_{gy}\frac{\partial E}{\partial y}+C_{g\theta}\frac{\partial E}{\partial \theta}=S \tag{2.11.2}$$

This is the energy balance equation, where the group velocities (energy transport velocity), C_{gx}, C_{gy}, and $C_{g\theta}$, do not include the influence of the flow.

These equations enable to stably analyze the wave propagation even if the computational grid is enlarged in the numerical calculation. Therefore, these equations are used for the numerical analysis of a wide area. Moreover, by adding the wave generation mechanism, nonlinear interaction term, and mechanism caused by wave breaking as a source term, these equations can be used for wave hindcasting by inputting the wind velocity field. In the handling of nonlinear interaction, the second generation, which is expressed using the parameter for specifying a spectrum, is changed to the current third-generation model, which calculates the nonlinear interaction as correctly as possible. The wave hindcasting model of the third generation is already used in the wave forecast services. Moreover, given that the energy balance equation without a source term can be used for wide-range refraction calculation from the offshore, it is practically used to calculate propagation to the vicinity of structures from the given deep water wave conditions and to calculate the incident wave condition of the structures. The Helmholtz equation is derived from a 3D wave equation with no changes in water depth, and is expressed as follows:

$$\frac{\partial^2 \Phi}{\partial x^2}+\frac{\partial^2 \Phi}{\partial y^2}+k^2\Phi=0 \tag{2.11.3}$$

where Φ is a function indicating the amplitude of the water surface and k is the wave number. A Green function method is used as the wave field analysis using the Helmholtz equation. Moreover, the Takayama method is used to approximately handle the evaluation of the wave field inside the harbor by superimposing solutions considering the reflective surface and by using the basic solution of the Green function.

Moreover, the mile slope equation is expressed as follows:

$$\nabla \cdot \left(C C_g \nabla \Phi \right) + \sigma^2 \frac{C_g}{C} \Phi = 0 \tag{2.11.4}$$

This is a method that can handle both refraction and diffraction simultaneously. However, considering that the equation to be solved is an elliptic differential equation, it is necessary to provide all boundary conditions and determine the distribution of water surface elevation that satisfies them; this requirement significantly increases the calculation load. Thus, a parabolic differential equation and equations that reduce the calculation load are proposed. There are examples that are applied to the wave breaking calculation with varying individual wave breaking limits for the height level of each wave [120].

② **Time evolution model**

The time evolution model can analyze changes in water surface, velocity, and other parameters in a time series. This model integrates motion in the vertical direction and evaluates variables such as horizontal velocity as a mean value to suppress the calculation load while enabling a time-series analysis. A nonlinear long wave equation is evaluated by assuming uniform motion in the vertical direction, and the equation of continuity and 1D equation of motion are expressed by **equations (2.11.5)** and **(2.11.6)**, respectively.

$$\frac{\partial \eta}{\partial t} + \frac{\partial Q}{\partial x} = 0 \tag{2.11.5}$$

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\frac{Q^2}{D} \right) + gD \frac{\partial \eta}{\partial x} = 0 \tag{2.11.6}$$

where η is the change in water surface, Q is the depth-integrated velocity, D is the total water depth, and g is the gravitational acceleration. The nonlinear long wave equation is also called a shallow water equation and is used for analyzing tsunamis. Although this equation can analyze waves with very long wavelengths, such as tsunamis, very accurately, its calculation accuracy becomes problematic at the deep water because it cannot express the dispersion of waves.

The Boussinesq equation corrects the distribution form in the vertical direction of motion by using a secondary or higher-order function to provide dispersion. Among these, a 1D equation of continuity and the equation of motion of the modified Boussinesq equation of Madsen and Sørensen[122], which is a primitive equation of NOWT-PARI [121], are often used when handling wave transformation in a port with complicated seabed topography. These equations are expressed as follows:

$$\frac{\partial \eta}{\partial t} + \frac{\partial Q}{\partial x} = 0 \tag{2.11.7}$$

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\frac{Q^2}{D} \right) + gD \frac{\partial \eta}{\partial x} = \left(B + \frac{1}{3} \right) h^2 \frac{\partial^3 Q}{\partial x^2 \partial t} + Bgh^3 \frac{\partial^3 \eta}{\partial x^3} + \frac{\mathrm{h}}{3} \frac{\partial h}{\partial x} \frac{\partial^2 Q}{\partial t \partial x} + 2Bgh^2 \frac{\partial h}{\partial x} \frac{\partial^2 \eta}{\partial x^2} \tag{2.11.8}$$

where h is the water depth and B is the correction parameter of 1/15. These equations are used for a numerical analysis of weak dispersible waves by approximately taking the vertical distribution of motion . An accurate analysis is possible to a certain extent if the water depth is shallow, but the calculation accuracy decreases fundamentally as the water depth increases. The nonlinear mild slope wave equation evaluates the vertical distribution of motion by using a rational or polynomial expression. This equation has higher approximation accuracy than an evaluation using a higher-order function and is more accurate than the Boussinesq equation but increases the calculation load. However, enhancements to the Boussinesq equation (support of strong nonlinearity and strong dispersion) are currently being developed.

③ **Three-dimensional wave field analysis** [123)]

In contrast to the plane wave field, the method of 3D wave field analysis is available for handling the vertical directional motion as correctly as possible. So far, this method has been used only for 2D cross-section analysis with decreased functions, because the calculation load becomes very large. However, the improvement in performance of computers or the spread of parallel computation technology is gradually making 3D analyses practical. A 3D analysis is also called a direct numerical simulation, and the primitive equation is the Navier–Stokes equation. A particle system model follows a phenomenon in the Lagrangian description, while a grid system model pursues a phenomenon in an Eulerian description in a solution using the Navier–Stokes equation.

The moving particle semi-implicit (MPS) method and smoothed particle hydrodynamics (SPH) method are the main streams in the particle system model. MPS explains the behavior between particles in an incompressible fluid with a semi-implicit scheme, whereas SPH expresses fluidic behavior by interpolating variables from particles. Considering that the particle system model grasps the behavior of fluid particles, the free surface can be easily expressed. Given that structures and others cannot be fully expressed only using the particle system model, linking to the discrete element method (DEM) makes it possible to evaluate the interaction with structures, including destruction and large deformation [123)]. However, the calculation load becomes quite large.

On the contrary, the grid system model discretizes the physical quantity on a computational grid by using a finite differential method or a finite volume method to convert and solve algebraic equations, including boundary conditions. Given that this method allows the setting of fixed boundaries per computational grid, the interaction between waves and structures can be considered by modeling the structure surface with a fixed boundary. There are some methods for evaluating the free water surface. The method of expressing the fluid by F function which is the fluid-filling factor of each computational grid cell and expressing the motion of the water surface by the advection calculation of F is called the volume of fluid (VOF) method. Other methods for expressing the free water surface include a level set method in which the water surface is defined by the zero contour surface of the auxiliary function, and a density function method in which the water surface is defined by the fluid density in the cell. The Super Roller Flume for Computer Aided Design of Maritime Structure (CADMAS-SURF), which targets a 2D cross section, is often used for handling the interaction between harbor structures and waves, and the CADMAS-SURF/3D targets three dimensions to define the water surface, by using the VOF method [124), 125)]. Considering that this assumes the structure surface to be a fixed boundary, this cannot support the large transformation problem of structures by itself. However, it becomes possible to handle a large transformation or destruction of structures by linking with the DEM, similar to the case of a particle system model [126), 127)]. CADMAS-SURF adopts a porous model that can take a porous material into consideration. The primitive equations are expressed as follows [123)]:

$$\frac{\partial \gamma_x u}{\partial x}+\frac{\partial \gamma_z w}{\partial z}=S_\rho \tag{2.11.9}$$

$$\lambda_v\frac{\partial u}{\partial t}+\frac{\partial \lambda_x u^2}{\partial x}+\frac{\partial \lambda_z wu}{\partial z}=-\frac{\gamma_v}{\rho}\frac{\partial p}{\partial x}+\frac{\partial}{\partial x}\left\{\gamma_x \nu_e\left(2\frac{\partial u}{\partial x}\right)\right\}+\frac{\partial}{\partial z}\left\{\gamma_z \nu_e\left(\frac{\partial u}{\partial z}+\frac{\partial w}{\partial x}\right)\right\}-D_x u+S_u-R_x \tag{2.11.10}$$

$$\lambda_v\frac{\partial w}{\partial t}+\frac{\partial \lambda_x uw}{\partial x}+\frac{\partial \lambda_z w^2}{\partial z}=-\frac{\gamma_v}{\rho}\frac{\partial p}{\partial z}+\frac{\partial}{\partial x}\left\{\gamma_x \nu_e\left(\frac{\partial w}{\partial x}+\frac{\partial u}{\partial z}\right)\right\}+\frac{\partial}{\partial z}\left\{\gamma_z \nu_e\left(2\frac{\partial w}{\partial z}\right)\right\}-D_z w+S_w-R_z-\gamma_v g \tag{2.11.11}$$

where u and w are the velocity in the horizontal and vertical directions, respectively; ρ is the density of water; p is the pressure; ν_e is the sum of the molecule kinetic viscosity coefficient and turbulent eddy viscosity coefficient; γ_v is the percentage of the void; γ_x and γ_z are the area permeabilities in the horizontal and vertical directions, respectively; γ_v, γ_x, and γ_z are the coefficients regarding inertia force; D_x and D_z are the coefficients for an energy dissipation zone; S_p, S_u, and S_w are the source terms for wave generation source; and R_x and R_z are the drag force from a porous material.

2.11.2 Classification of Numerical Analysis Targeting at Structures

The following are assumed as numerical analyses concerning the following:

① The wave-resistance stability of structures
② The reflection/transfer performance of structures
③ The wave overtopping prevention of structures
④ The tsunami-resistance performance of structures

2.11.3 Computers and Analysis Software

(1) Computer

In the numerical analysis, the processing performance of the computer used for the calculation is important. Specifically, a processor with good performance is preferable to shorten the computation time. Recently, multicore CPUs have become the standard for CPUs used for calculation processing. This development enables a multiplex processing of calculation and plural numerical analyses. Moreover, some computers that can perform parallel processing of numerical analysis codes have recently been introduced in the market to reduce computation time. However, considering that considerable numerical calculation writes a large quantity of analysis results and intermediate files, mass storage devices are becoming a necessity.

(2) Analysis software

Although there are various general-purpose numerical analysis software that can be used for performance verification of structure, analyses using CADMAS-SURF and CADMAS-SURF/3D are described in this study.

2.11.4 Examination of Natural Conditions for Numerical Analyses

The following are assumed as examinations of natural conditions for numerical analysis concerning the performance verification of structures. They are fundamentally the same as those presented in **Reference [Part II], Chapter 1, 2.10**, and thus, are not described here.

① Tide level
② Waves
③ Seabed topography

2.11.5 Points to Note in the Numerical Analyses [123]

(1) Waves in the numerical wave tank

Given that CADMAS-SURF handles strong nonlinear and dispersion of wave, most of the problematic phenomena in hydraulic model experiments, except for the influence of surface tension, can be generated in numerical analysis (for details, refer to **Reference [Part II], Chapter 1, 2.10**). In this section, the problems of waves occurring only in numerical analysis are described.

When waves are evaluated by numerical analysis, the wave height is attenuated by the generation of numerical viscosity (or numerical diffusion) accompanying the differential analysis of an advective term. Although it is preferable to make a computational grid as small as possible to avoid this phenomenon, attention should be paid to not apply an excessively large calculation load.

(2) Setting of the wave generation method

CADMAS-SURF uses a wave generation source that creates waves by generating the flow rate on the basis of nonlinear wave theory from a certain vertical cross section in a tank. In the case of regular waves, given that the flow-out velocity is controlled by considering the velocity characteristics of a nonlinear wave motion and considering that the wave generation source is a fixed point, waves called the secondary wave crest do not occur fundamentally within a numerical wave tank. Although attention should be paid to the necessity of correction for the mass transport (of wave) velocity in the wave generation signal, a simple correction method has been proposed [128].

(3) Settings of computational area and computational grid

The computational area should be determined by considering the calculation load and including waves propagating from the wave generation position, energy absorption area for the reflected or transmitted waves from structures, and distance required for the transmitted wave overtopping to stabilize when considering wave overtopping. The size of the computational grid greatly influences the calculation load. To improve the calculation accuracy, it is preferable to reduce size of the computational grid to the greatest extent possible. As a rule of thumb, it is the grid size required for reproducing the shape of the structures.

(4) Setting of the length of computation time and computational time interval

The concept of the length of computation time is important, such as in the hydraulic model experiment; further, it can be determined by referring to **Reference [Part II], Chapter 1, 2.10.5**. Moreover, although the calculation accuracy improves as the time interval in the calculation reduces, the calculation load increases. Therefore, the time interval should be set by multiplying a proper safety factor with the value that satisfies the Courant–Friedrichs–Lewy (CFL) conditions (conditions concerning the stability in advection calculation) and the conditions for the stability in viscous term calculation. The CFL conditions and the condition for the viscous term to stabilize are expressed by **equations (2.11.12)** and **(2.11.13)** for the time interval Δt.

$$\Delta t \le \min\left(\frac{\Delta x}{u}, \frac{\Delta z}{w}\right) \tag{2.11.12}$$

$$\Delta t \le \frac{1}{2}\frac{1}{\nu\left[\left(\frac{1}{\Delta x}\right)^2 + \left(\frac{1}{\Delta z}\right)^2\right]} \tag{2.11.13}$$

where ν is the turbulent eddy viscosity coefficient and Δx and Δz are the grid width in horizontal and vertical directions, respectively. The use of an automatically set condition by providing a suitable safety factor according to the calculation situation rarely makes the calculation unstable.

(5) Setting of the boundary condition

For the stable calculation in a numerical wave tank, a boundary condition should be set accurately. In a nonreflective boundary, it is necessary to set an energy dissipation zone appropriately while providing Sommerfeld's radiation boundary condition. Given that reflection occurs inside the energy dissipation zone if the parameter indicating the dissipation factor is extremely large, the energy dissipation zone needs to be set appropriately.

(6) Setting of the analysis parameters

There are a few parameters that should be set for the numerical wave tank. First, a turbulence model that can consider a turbulence influence that is smaller than the grid size can be introduced. Although the effect of a turbulence model is not remarkable in the propagation and transformation of usual waves, including wave breaking, the turbulence model becomes predominant when the water is shallow and turbulence is strong.

Although a sine wave, the 5th Stokes wave, 3rd cnoidal wave, and wave generation phenomenon by nonlinear waves using Dean's stream function method can be selected as a wave generation model, it is necessary to select a wave generation phenomenon by using the nonlinear wave to reduce the influence of secondary wave crest.

The advection calculation of the F value indicating fluid in the water surface calculation adopts the donor acceptor method, which is a hybrid approach of upwind difference in the primary accuracy and central differences in the secondary accuracy. Here, the primary and secondary accuracies are the degrees of approximation accuracy in the difference method. The donor scheme configured by the donor acceptor method introduces the donor parameter D, and is expressed as

$$\text{Donor scheme} = D \times \text{Primary accuracy windward difference} + (1 - D) \times \text{Secondary accuracy center difference} \tag{2.11.14}$$

If the upwind difference of the primary accuracy is selected, the calculation stabilizes; however, the water surface is excessively smoothed. On the contrary, although the water surface can be sharply expressed in the center differences of the secondary accuracy, there is a problem in the calculation stability. An intermediate value of these two schemes can generally be selected; however, setting the value of the donor parameter D to approximately 0.2 enables the reproduction of a relatively stabilized sharp water surface profile.

For porous materials, such as wave-dissipating blocks, two types of resistance rules are available. One is the drag force proportional to the square of velocity using the CD, and the other is the drag forces comprise proportional to velocity and to acceleration, called Dupuit–Forchheimer rules. Therefore, it is necessary to select a suitable resistance coefficient value for the target structures.

The drag forces obtained using the C_D value are expressed as

$$R_x = \frac{1}{2}\frac{C_D}{\Delta x}\left(1-\gamma_x\right)u\sqrt{u^2+w^2} \quad \textbf{(2.11.15)}$$

$$R_z = \frac{1}{2}\frac{C_D}{\Delta z}\left(1-\gamma_z\right)w\sqrt{u^2+w^2} \quad \textbf{(2.11.16)}$$

The drag forces obtained using the Dupuit–Forchheimer rule are expressed as

$$R_x = \gamma_v\left(\gamma_x u\right)\left(\alpha+\beta\sqrt{\left(\gamma_x u\right)^2+\left(\gamma_z w\right)^2}\right) \quad \textbf{(2.11.17)}$$

$$R_z = \gamma_v\left(\gamma_z w\right)\left(\alpha+\beta\sqrt{\left(\gamma_x u\right)^2+\left(\gamma_z w\right)^2}\right) \quad \textbf{(2.11.18)}$$

$$\alpha = \alpha_0\frac{\left(1-\gamma_v\right)^3}{\gamma_v^2}\frac{\nu}{d}\ ,\ \beta = \beta_0\frac{\left(1-\gamma_v\right)}{\gamma_v^3}\frac{1}{d} \quad \textbf{(2.11.19)}$$

where ν is the kinetic viscosity, d is the typical diameter of a stone, and α_0 and β_0 are the coefficients according to the material [122].

2.11.6 Numerical Analysis Concerning the Performance Verification of Structures

(1) Performance verification of structures

CADMAS-SURF, which is a numerical analysis code with a proven track record, has been conducted by several researchers and engineers in a numerical analysis concerning the performance verification of structures. It is possible to support the performance verification of wave-resistance stability, reflection/transmission performance, wave overtopping prevention performance, tsunami-resistance performance, and others, except for the phenomena accompanying the destruction of structures.

(2) Specifications of waves and tide level used for analysis

The wave conditions and tide level specifications used for the analysis, such as the setting of incident waves, are the same as those used for the hydraulic model experiment. More information is available in **Reference [Part II], Chapter 1, 2.10.6** to **2.10.9**.

(3) Modeling of structures

Structures must be modeled appropriately in CADMAS-SURF because the resolution when presenting them depends on the size of the computational grid. What is necessary for impermeable structures is to approximate a structure form in the resolution of grid size and to set the conditions for impermeability. When targeting structures with complicated shapes, such as slit structures, set the resistance appropriately to the slit part and reproduce the predetermined reflection/transmission characteristics because the 2D cross-section calculation cannot reproduce the slit form completely. In this case, care must be taken in the verification of data, such as the data of the hydraulic model experiment.

(4) Method of numerical analysis

Fig. 2.11.2 shows the procedure for numerical calculation, which involves the following steps. 1) Set the calculation area and determine the locations of the structures and energy dissipation zone. 2) Model the topography and arrange it in a computational grid. 3) Set various calculation parameters. 4) Examine the incident wave. 5) Arrange the modeled structures on the computational grid. 6) Set the tide level conditions. 7) Set the amount monitoring position of the water surface elevation, velocity, wave pressure, etc. 8) Set the wave conditions. 9) Perform a numerical analysis and obtain the result.

The wave overtopping discharge can be calculated by setting a control cross section on an upper side of structure and with the time average of the volume of the water mass passing the control cross section. Another method for calculating the wave overtopping discharge is to count the increment of F value in the water area behind the wall body.

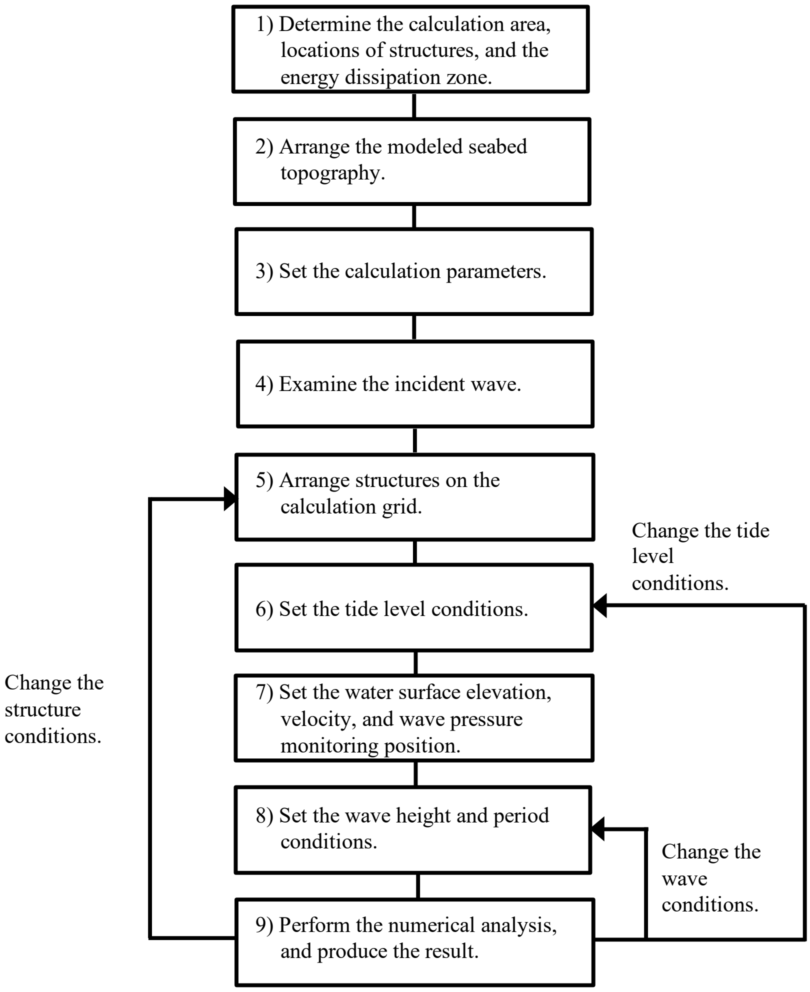


Fig. 2.11.2 Procedure for Numerical Analysis in Numerical Wave Tank

(5) Arrangement of numerical analysis data

The numerical analysis data of usual water surface fluctuation, velocity change, and pressure fluctuation are produced as an analysis result, similar to the output data obtained from the measuring instruments in the hydraulic model experiment. Considering that it is necessary to judge whether the calculation is appropriate, the calculation result should be validated by outputting the cross-sectional profile of velocity, pressure, and other factors using a visualization tool.

The arrangement of numerical analysis data is the same as that of the data in a hydraulic model experiment. Refer to **Reference [Part II], Chapter 1, 2.10.6 (5)** for further information.

2.11.7 Three-dimensional Numerical Analysis [123)]

(1) Outline and utilization of 3D numerical analysis

Although the 3D numerical wave tank for CADMAS-SURF/3D can handle transformations such as spatial wave transformation by using obliquely incident waves, the large calculation load currently requires a time-intensive and large-scale calculation. When reproducing an experiment concerning a wave flume, CADMAS-SURF/3D can be used to reproduce structures in detail; this could not be realized by the cross sectional simulation.

(2) Specifications of waves and tide level used for 3D analysis

Considering that the setting of waves and tide levels used for 3D analysis is the same as that of the hydraulic model experiment, refer to **Reference [Part II], Chapter 1, 2.10.10**.

(3) Modeling of structures

The 3D analysis can reproduce relatively detailed structures. Moreover, given that numerical analysis can be conducted by arranging a model and a wave flume in the hydraulic model experiment, in this case, it is also possible to perform analyses without modeling the structures. On the contrary, it is also required to appropriately model structures and perform a numerical analysis, because the calculation load becomes extremely large.

(4) Method of numerical analysis

The procedure of the numerical analysis is the same as that of the 2D cross-sectional analysis (**Fig. 2.11.2**).

(5) Arrangement of numerical analysis data

The arrangement of the 3D analysis data is the same as that in the experiment using the wave flume or wave tank, and a directional wave spectrum analysis can be performed using the result of numerical analysis. On the contrary, given that the amount of numerical analysis data exceeds that in the case of the hydraulic model experiment, the data should be handled with care.

(6) Points to note in 3D analysis

Considering that the 3D analysis can express structures in detail, it enables a more realistic numerical analysis as compared with the 2D cross-sectional analysis. Although the 3D analysis requires considerably more calculation amount than the 2D cross-section analysis, high-speed processing is possible by using parallel processing with multicore CPUs.

However, the setting of a nonreflective boundary requires devices that set only a radiation boundary condition, because it is necessary to set a large calculation area, and the surrounding analysis area increases the calculation load. Moreover, acceleration by parallelization or 2D analysis, by considering one mesh in the on-offshore direction, enables 2D calculation using CADMAS-SURF/3D. In this case, the effect of parallelization enables faster calculation than CADMAS-SURF.

(7) Example of calculation using CADMAS-SURF/3D analysis

Figs. 2.11.3 and **2.11.4** show examples of calculation (snapshot) conducted using CADMAS-SURF/3D.

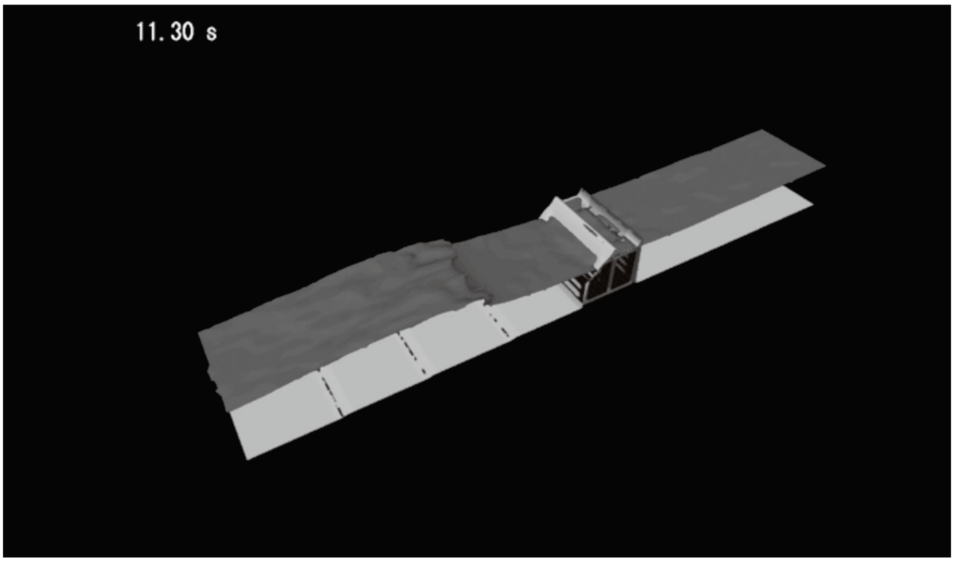


Fig. 2.11.3 Example of a Reproduction Calculation of Hydraulic Experiment Concerning the Wave-Absorbing Structure in Cross-Sectional Tank by CADMAS-SURF/3D (Snapshot)

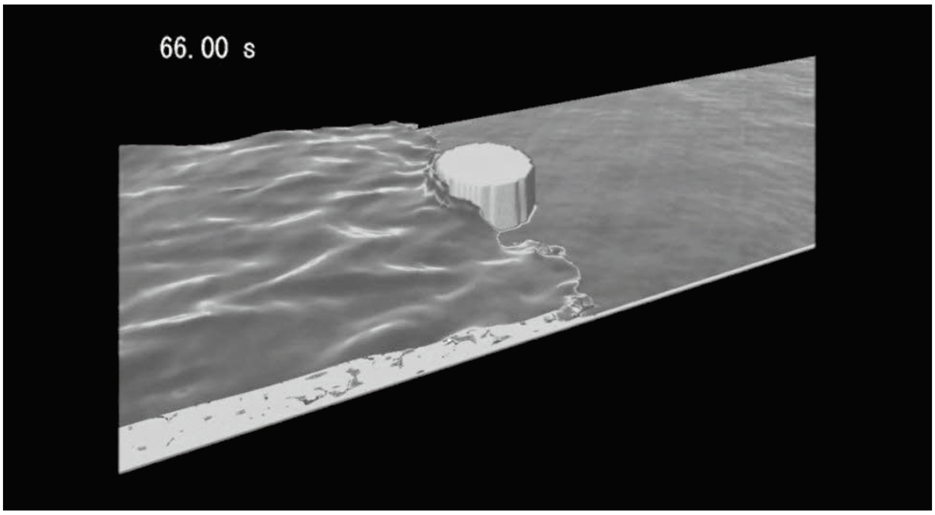


Fig. 2.11.4 Example of a Wave Calculation around a Cylinder Using the Cross-Sectional Tank by CADMAS-SURF/3D (Snapshot)

2.11.8 Calculation of Design Waves (Plane Wave Field Analysis)

(1) Numerical analysis in calculation of design waves

When performing design verification using a hydraulic model experiment or numerical analysis, it is necessary to evaluate, in advance, the design waves entering the structures. These waves are calculated by plane wave field analysis using the design deep water waves obtained by conducting statistics analysis of extreme waves using the wave observational data over a long period or wave hindcasting data at the time of high wave invasion as the incident wave conditions. Therefore, the plane wave field analysis performs numerical analyses, including refraction, diffraction, and wave breaking, by targeting a wide range from the offshore point, where the design deep water waves are provided to structures subjected to the design. Although the plane wave field analysis arranges calculation conditions by using the results of examination of natural conditions for the numerical analysis shown in **Reference [Part II], Chapter 1, 2.11.4**, the spatial topography conditions (i.e., spatial distribution of water depth) are indispensable in the calculation. Moreover, the conditions for surrounding structures should be considered in the calculation, if required.

The energy balance equation, which is a phase-averaged wave model, and the Boussinesq equation, which is a time evolution model, are often used for calculating the plane wave field. Although these calculation methods are briefly described in **Reference [Part II], Chapter 1, 2.11.1**, their features are shown below.

(2) Plane wave field analysis using the energy balance equation

① Outline of the energy balance equation

The energy balance equation is a method for calculating the space distribution of the wave energy. Energy is a scalar quantity and does not have information concerning direction. Therefore, energy flux is defined by multiplying wave energy with energy transport velocity, and the wave energy is considered to be transported in the direction of wave propagation. The energy balance equation is derived from the fact that this energy flux is preserved spatially. Given that the energy balance equation considers the preservation of energy in the wave ray direction, it can handle wave refraction or shoaling with accuracy but cannot handle the wave diffraction phenomenon, which is the flow of energy in the direction of the wave crest line. Mase et al.[129)] proposed a model that can approximately consider the effect of diffraction on the energy balance equation by adding the approximated value to the energy balance equation, by using the wave energy of the diffraction term in the parabolic equation, which isolates and approximates only the progressive wave from the mild slope equation that can evaluate the diffraction of waves.

② Consideration of reflected waves

Given that the energy balance equation is also the wave transformation calculation method for the progressive wave in the wave direction, the incident and reflected waves heading offshore cannot be considered simultaneously. When considering reflected waves, new waves must be generated at the reflective boundary, the reflected waves should be separately calculated as waves progressing toward the offshore direction; further, the wave field should be evaluated by the principle of summation of incident and reflected energy components. However, because the energy balance equation cannot evaluate phases, the calculation accuracy deteriorates near the reflective surface. The design waves are generally evaluated as progressive waves when calculated. Therefore, the calculation result of progressive waves obtained by the energy balance equation can often be used as it is.

③ **Consideration of wave breaking**

The wave breaking in the energy balance equation can be considered by adding an energy dissipation term. Although various methods have been proposed for the energy dissipation term, Takayama et al [130] evaluated the energy dissipation rate through wave breaking by locally applying Goda's breaker index [131].

④ **Other features**

In the energy balance equation, the propagation deformation of the directional wave spectrum can be considered by superimposing the calculation results of various frequencies and wave directions. Moreover, the energy balance equation requires a short computation time because it follows the variations in the average amount of energy and direction of flux without calculating the time series of water surface fluctuation, such that the interval between computational grids with approximately one-tenth of the wavelength is sufficient, even when the change in water depth is relatively predominant. However, the energy balance equation is a phase-averaged wave model and handles an average amount called energy. Therefore, it cannot calculate quantities such as the maximum wave height directly. It is necessary to conduct evaluations of the maximum wave height by using the limiting breaking wave height within a surf zone approximately or by performing a separate evaluation by means of Goda's wave breaking model using the equivalent deep water wave height, or other methods.

(3) Plane wave field analysis using the Boussinesq equation [121]

① **Outline of the Boussinesq equation**

The waves exhibit significant features of dispersion and nonlinearity. The dispersion of waves is a phenomenon in which the wave propagation velocity differs by the period. On the contrary, the nonlinearity of waves represents a phenomenon in which the crest of a wave sharpens and its trough becomes flat when the wave is high or when the water is shallow. Furthermore, effects of heighten waves as nonlinearity becomes stronger, even if the wave energy remains constant. To calculate the wave transformation correctly, it is necessary to consider the dispersion and nonlinearity of waves as correctly as possible.

The Boussinesq equation is a calculation model that can consider nonlinearity to some extent, along with the dispersion of waves, when water is shallow. It is a model that belongs to the so-called wave equation and is an analysis method that can simultaneously handle the refraction, diffraction, and reflection of waves. Various models have been proposed by the approximation method of dispersion and the degree of nonlinearity that can be considered. With the Boussinesq equation, the moment-by-moment changes in water surface elevation and velocity can be spatially calculated by numerically resolving equation of continuity and the equation of motion as if performing a hydraulic model experiment. Therefore, the load of calculation is large, a long computation time is required, and the output of results obtained by the calculation is quite large. Moreover, given that it is an equation based on long waves, attention should be paid to the fact that the calculation accuracy decreases in deep water area.

② **Consideration of reflected waves**

In the plane wave field analysis using the Boussinesq equation, the condition for fully reflecting the structures can be calculated by assigning "0" to the velocity perpendicular to the face line of the structure at its boundary. On the other hand, in case of the partial reflection condition, it is necessary to arrange an energy [132], called the reflectance-adjusted sponge layer, same as that used by CADMAS-SURF, or a wave-absorber using resistance rules such as the Dupuit–Forchheimer rule, in the front surface of structures.

③ **Consideration of wave breaking**

Although the wave breaking is evaluated by adding a diffusion term to the equation of motion, some views of the coefficient of dissipation have been proposed. Hirayama and Hiraishi [134] proposed a method that combines a time evolution bore model and a one equation turbulence model to evaluate the dissipation coefficient of wave breaking and adopted a condition based on the pressure gradient in the vertical direction on the water surface on the basis of the Boussinesq equation, as a judgment condition for wave breaking.

④ **Other features**

Considering that the Boussinesq equation calculates the moment-by-moment changes in water surface elevation or velocity, it is possible to calculate the highest wave height or directional wave spectrum by statistically processing such time-series data. To perform such statistical processing, a computation time that ensures at least 200 or more waves is required.

The Boussinesq equation can consider a phenomenon resulting from the nonlinearity of waves in its calculation, such as the long-period wave containing the surf beat, which becomes remarkable in the shallow water area, rise of the mean water level after wave breaking (wave setup), and the nearshore current. Attention should be paid to the nearshore current because it may degrade the calculation accuracy for a reason that will be mentioned later.

Recently, methods that use a highly precise Boussinesq equation for calculating wave runup, waves overtopping structures, and other waves have been proposed, which can be applied to wave transformation on a coral reef with good accuracy. A practical study of the evaluation of a phenomenon where the sea bottom is exposed by back rush or of an inundation caused by waves has become possible [135)].

2.11.9 Analysis Procedure for Plane Wave Field

(1) Analysis procedure for plane wave field using the energy balance equation

Fig. 2.11.5 shows the procedure for the numerical calculation of the plane wave field by using the energy balance equation. The steps are as follows. 1) Set the calculation area considering the structure locations. 2) Set the seabed topography (water depth) according to the computational grid from nautical charts. 3) Arrange the modeled structures on the computational grid. 4) Set the tide level conditions. 5) Determine the directional wave spectrum as incident wave conditions and set the divided numbers of frequency component and that of wave direction component of a spectrum, according to a shape of the directional wave spectrum. 6) Perform a numerical analysis. 7) Produce the analysis result after the calculation is completed. Change the structure conditions, tide level conditions, or incident wave conditions, if needed, and repeat the calculations.

(2) Analysis procedure for plane wave field using the Boussinesq equation

Fig. 2.11.6 shows the procedure for the numerical calculation of the plane wave field using the Boussinesq equation. The steps are as follows. 1) Set the calculation area by considering the locations of structures. 2) Set the seabed topography (water depth) according to the computational grid from nautical charts. 3) Set the computational grid interval and computation time interval by considering the analysis conditions, set the conditions for the sponge layer having the required reflectance by performing trial calculation by using the Boussinesq equation, with arranging the sponge layer under appropriate area or water depth conditions, or arrange the wave-absorber by using a resistance rule. 4) Create an incident wave time-series data comprising the water surface fluctuation and velocity changes suitable for the Boussinesq equation according to the incident wave conditions. 5) Arrange the modeled structures on the computational grid. 6) Set the tide level conditions. 7) Set the monitoring position of the water surface elevation and velocity. 8) Set the incident wave data created in 4) according to the conditions for wave height, period, and tide level as the input data. 9) Perform the numerical analysis and output the analysis result after the calculation is completed. Change the structure conditions, tide level conditions, or incident wave conditions, if needed, and repeat the calculations.

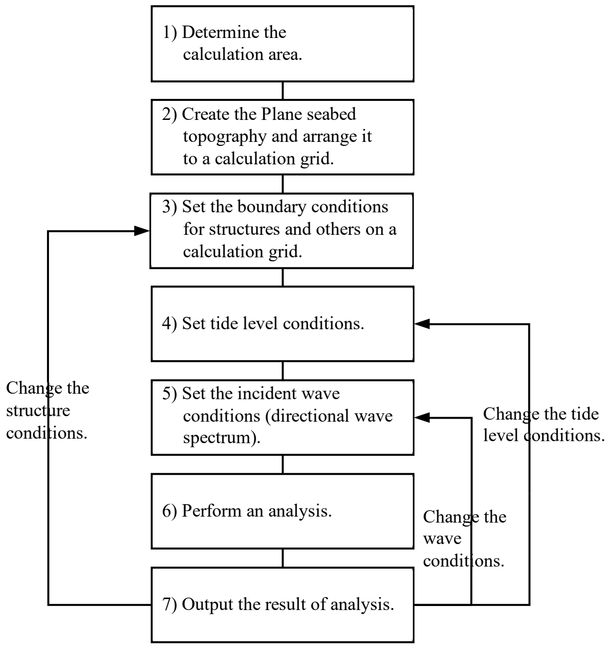


Fig. 2.11.5 Numerical Analysis Procedure for Plane Wave Field Using the Energy Balance Equation

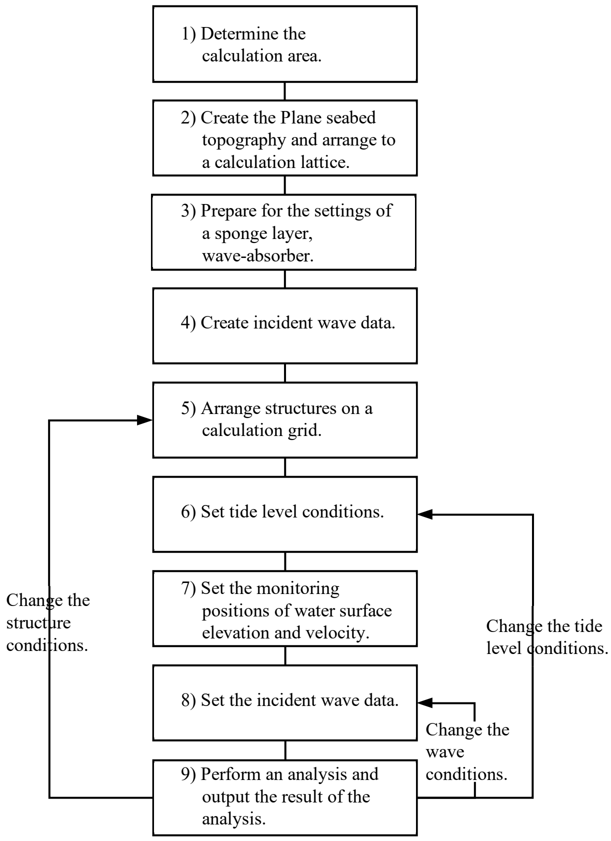


Fig. 2.11.6 Numerical Analysis Procedure for a Plane Wave Field Using the Boussinesq Equation

2.11.10 Points to Note in the Plane Wave Field Analysis

(1) Points to note in plane wave field analysis using the energy balance equation

① **Setting of calculation area**

Wave refraction depends on the degree of change in water depth. Generally, shallower water leads to more influence from the change in water depth, while deeper water corresponds to less influence. Furthermore, a longer wave period leads to more influence from the change in water depth, while a shorter wave period leads to less influence from the change in water depth. Therefore, the computational grid interval can be set wide if the water is deep and the period is relatively short, i.e., the wavelength is short in the numerical analysis using the energy balance equation. Conversely, if the water depth is shallow and the wave period is relatively long (wavelength is long), the computational grid needs to be appropriate.

Given that the energy balance equation transmits only a statistical value, called the directional wave spectrum, which changes gradually according to the topography change on adjacent computational grids, the influence of data degradation is small even if the directional wave spectrum located between adjacent grids is estimated by interpolation. Therefore, a calculation conducted by setting a large grid size in offshore side, making the grid size one-half or one-third in the onshore side from a certain position and connecting areas may not generate a harmful effect on the calculation accuracy. The process of dividing the calculation area into several steps, connecting the areas, and decreasing the grid size is called nesting. Usually, different types of analysis software using the energy balance equation comprise the nesting function.

The actual waves in the sea attack from various directions offshore. Therefore, it is necessity to study that the widest possible range is subjected to calculation by considering nesting.

② **Setting and division of the directional wave spectrum**

The directional wave spectrum should be added to the energy balance equation as the input condition. Generally, it contains the Mitsuyasu-type directional spreading function and uses the Bretschneider–Mitsuyasu-type spectrum as the frequency spectrum. When setting the incident wave conditions, it is necessary to appropriately select the degree of directional energy concentration of the directional spreading function according to the water depth and wave period conditions. Although the degree of directional concentration of the directional wave spectrum increases with refraction, while propagating from the offing toward the shore, and given that the frequency spectrum (or wave height) also changes, the incident wave conditions should be given at a point that is as deep as possible. However, when starting the calculation from a point where the water depth is relatively shallow, according to the calculation restrictions, this must be considered. Although the directional spreading function is divided into equal intervals in the wave direction, the frequency spectrum is generally divided such that each portion has equal energy (**Fig. 2.11.7**). The frequency when dividing the spectrum into equal energies should be that calculated from the secondary moment of the spectrum in the target frequency band. Usually, the direction division width is set to 5°–6° or less. Moreover, the number of frequency divisions is set to 5 to 15 in many cases. Although division even at this level provides a practically sufficient calculation accuracy, the frequency should be divided as finely as possible within a permissible range by considering the calculation load and computer performance.

Moreover, although the range in which the directional incident wave spectrum is given is directionally divided by the principal wave direction as its center, the range of the waves to be calculated should be the component that is on the shore side of the calculation area. Furthermore, the target range should be narrowed down, and fewer directional divisions may be set for waves with high degree of directional concentration, such as swell.

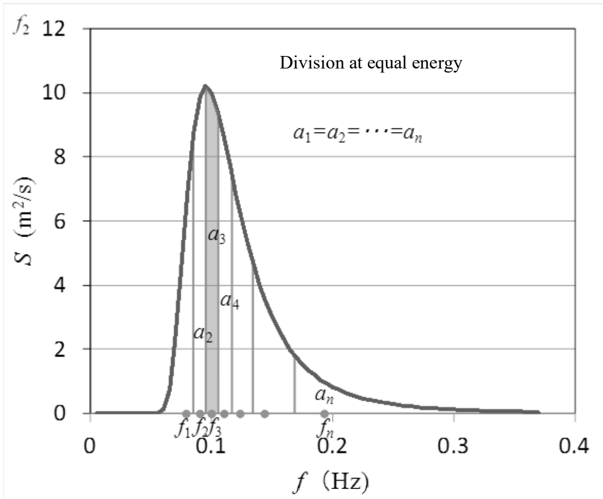


Fig. 2.11.7 Schematic of Frequency Spectrum Divided into Equal Energies

③ **Evaluation in wave shelter zone**

Considering that the energy balance equation is a numerical analysis method that fundamentally targets refraction only, caution is required for the calculation accuracy, which degrades in the wave shelter zone at the back of islands or structures. The energy balance equation considers waves toward various directions. Therefore, waves propagating toward a certain directional component may be sheltered but not by other wave components propagating toward different directions, even at the back of the islands or structures. As a result of the superimposition of these component, the energy of waves also gets transmitted the area where waves should have been sheltered in the principal wave directions. This phenomenon is called directional spreading. Furthermore, energy leaks from where more energy is stored to where less energy is stored in the numerical analysis; that is, the numerical dispersion generates a phenomenon that resembles diffraction. Given that the actual waves have relatively strong nature of propagating straight, the diffraction phenomenon can be reproduced, to some extent, even with the apparent diffraction, due to directional dispersion. Although the result of the numerical calculation, together with the numerical dispersion, may appear real diffraction, it must be notice that it is still the result obtained via approximation.

④ **Output of the analysis result**

The directional wave spectrum on the spatial grid is finally obtained from the result of numerical calculation by using the energy balance equation. Given that the storage capacity of these analysis results is not very large, it is common to save these pieces of information at all grid points. The energy of waves can be obtained by integrating the directional wave spectrum at each grid point, by using wave directions and frequencies. The root-mean-square value of wave height can be obtained by taking the square root of the energy obtained in this way and multiplying it with $8^{1/2}$. When waves are not broken, conversion to wave statistic values, such as a significant wave height, is possible by assuming Rayleigh distribution as the occurrence frequency distribution of wave height.

⑤ **Consideration of the influence of nonlinearity**

Although the nonlinearity of waves is significantly influenced under high-wave conditions, such as design waves, the energy balance equation is a type of linear analysis. Even under conditions with a significant influence of such nonlinearity, a linear analysis is enough when considering changes in energy caused by wave propagation. This is because the sharpened wave crests do not increase the potential energy itself, even if the wave height increases according to the nonlinear wave characteristics. The same is true for kinetic energy: the velocity profile is sharpened more but the kinetic energy itself does not increase. Therefore, as long as the wave transformation is evaluated by energy, the linear numerical analysis is sufficient. On the contrary, although an actual design requires the wave height, crest height, or velocity amplitude, it is necessary to consider the nonlinearity of waves by using nonlinear wave theories, such as the Stokes wave or cnoidal wave, when converting energy into these wave specifications.

(2) Points to note for the analysis of plane wave field by the Boussinesq equation

① **Setting of calculation area**

Given that the Boussinesq equation improved the wave dispersion from the long wave theory as the starting point, as stated in **Reference [Part II], Chapter 1, 2.11.8 (3)**, it cannot be denied that the calculation accuracy deteriorates as the water depth increases. According to the evaluation result of the approximation accuracy, approximately 0.3–0.5 of the relative water depth satisfies the linear dispersion relations. When considering this condition, waves with 10-s periods maintain sufficient calculation accuracy even at a depth of 50 m or more, but those with 5-s periods reduce the calculation accuracy at 20 m depth. In the calculation targeting at irregular waves, it is difficult to perform accurate calculations under the conditions for deeper water.

On the contrary, given that the wave velocity increases with the water depth, the stability of the calculation cannot be secured unless the computation time interval is shortened, provided that the computational grid is the same. This means that considerable computation time is required under the condition of deep water.

As mentioned before, the practical conditions for offing in the numerical calculation by the Boussinesq equation requires a water depth of approximately 20–30 m or shallower. If the calculation from a deep water area is required, improvement in the calculation accuracy is expected by calculating the energy balance equation in an extensive area, including a deepwater area, and by using a method to take over the directional wave spectrum obtained by the calculation using the energy balance equation in a narrow area where the water depth is 20–30 m or shallower to the Boussinesq equation as the incident wave conditions [136)]. Note that the connection water depth of an extensive area and a narrow area naturally depends on the wave or topographical conditions.

② **Creation of the incident wave data**

The Boussinesq equation requires the time series of water surface elevation and velocity changes at the incident wave boundary. When multidirectional random waves are considered, the incident wave data may be created by the single summation method, such as that in the hydraulic model experiment shown in **Reference [Part II], Chapter 1, 2.10.5**. What must be taken care of here is the relation between the computational grid and the physical quantities such as water surface elevation or velocity. The numerical calculation of the Boussinesq equation usually adopts an assignment system of the physical quantity for the computational grid, which is usually called a staggered grid. This technique improves the stability of the numerical analysis by using a method for obtaining the position at which the change in velocity is given on the grid point providing the water depth and the change in water surface elevation is given at the intermediate position of the grid point providing the velocity. Therefore, incident wave data must be created by considering that the velocity and water surface elevation are shifted by half of the grid interval.

③ **Setting of a sponge layer or wave-absorber**

In the numerical calculation using a wave equation, it is necessary to use either the transparent boundary or wave-absorbing boundary to reduce the reflection from an incident wave boundary, side boundary, boundary at the shore side edge, etc. As the transmission performance of the transparent boundary depends on the passing velocity (period and direction of waves) of boundary waves, its setting is difficult when handling the wave transformation of irregular waves. Therefore, an energy absorption zone, called a sponge layer, is generally installed as a wave-absorbing boundary. This is the same as a netlike fiber or other fiber types that are suitably arranged to absorb waves in a hydraulic model experiment.

In the energy absorption zone, the dissipation term, to be in proportion with the velocity, is corrected, so as to conform to the Boussinesq equation. Given that the rapid increase in the energy absorption rate of the energy absorption zone in the wave propagation direction generates reflected waves from this direction, the energy absorption rate should be devised to increase it spatially and gradually. The necessary width of the energy absorption zone set in this manner varies according to the wavelength. The width of the energy absorption zone also needs to be widened if the wavelength is long, but it may be relatively narrowed when the wavelength is short [137)].

The energy absorption zone set in this manner should be arranged on boundaries that require wave absorption, such as those behind an incident wave boundary and those on the front surface of a side boundary, to control the reflected waves that are unnecessary for calculation.

Moreover, the waves reflected by structures enable to reproduce wave-absorbing characteristics of wave-dissipating concrete blocks in the calculation area by installing the wave-absorber by using the Dupuit–Forchheimer rule, such as CADMAS-SURF [138)].

④ **Setting of the monitoring position of water surface elevation or velocity change**

The Boussinesq equation can obtain the change in water surface elevation on a grid point or that in the velocity in two horizontal directions by the computation time step (strictly speaking, the calculation positions of the water surface fluctuation and the velocity change shift by half grid). Given that these pieces of information for all grid points become immense as data, the time-series data for computation time length concerning the limited monitoring points are usually selected and stored. The data at each grid point are statistically processed during the calculation, and all representative values of waves, such as wave height or period, are often outputted and saved as the data on the grid points.

Therefore, it is necessary to preset the grid points on which detailed data should be outputted before the calculation. It is necessary to determine the positions at which useful information that are needed after the calculation is given at this time, for the purpose of calculation of reflection rate, directional wave spectrum, long-period wave, or others.

⑤ **Evaluation of the calculation result using the Boussinesq equation**

Although the change in water surface elevation, including nonlinearity, can be calculated with the Boussinesq equation, care should be taken regarding the trend that the calculation result changes more mildly than that in reality, at places wherein nonlinearity is quite strong, such as near the wave crest at the time of high waves, depending on the degree of approximation of dispersion or nonlinearity in the Boussinesq equation. This is the reason for the actual phenomenon being reproduced with good accuracy by the easing of the judgment conditions for wave breaking in the Boussinesq equation. However, given that these extremely strong nonlinear phenomena occur under limited conditions, the analysis result with the Boussinesq equation does not indicate practical problems.

As already stated, phenomena arising from the nonlinearity of waves can be considered in the calculation. It is peculiar that the second-order nonlinear phenomena, which are proportional to the square of the wave height, can be evaluated with high accuracy. As mentioned above, they are long-period waves that contain the surf beat, which becomes remarkable in shallow areas and during the rise of the mean water level after wave breaking (wave setup), nearshore current, etc. Although it is necessary to consider the diffusion of the momentum resulting from the disorder by wave breaking to evaluate the nearshore current accurately, the coefficient suitable for the long-time-scale phenomenon, such as the nearshore current and the coefficient suitable for the energy dissipation by wave breaking, cannot be set simultaneously because the Boussinesq equation utilizes the diffusion of the momentum for the wave breaking model. Tajima et al. [139)] indicated that the reproducibility of the strength or occurring position of a circulation system decreases in the calculation of nearshore currents using the Boussinesq equation, and that the calculation accuracy of the nearshore current improves to some extent by introducing the anisotropic diffusion coefficient, which uses the different diffusion coefficients for the longshore direction and the on–offshore direction.

⑥ **Calculation example of numerical analysis using the Boussinesq equation**

Fig. 2.11.8 shows a calculation example of wave height distribution by the Boussinesq equation, around the Hitachi Naka Port at the time that the protective facilities for harbors were under construction.

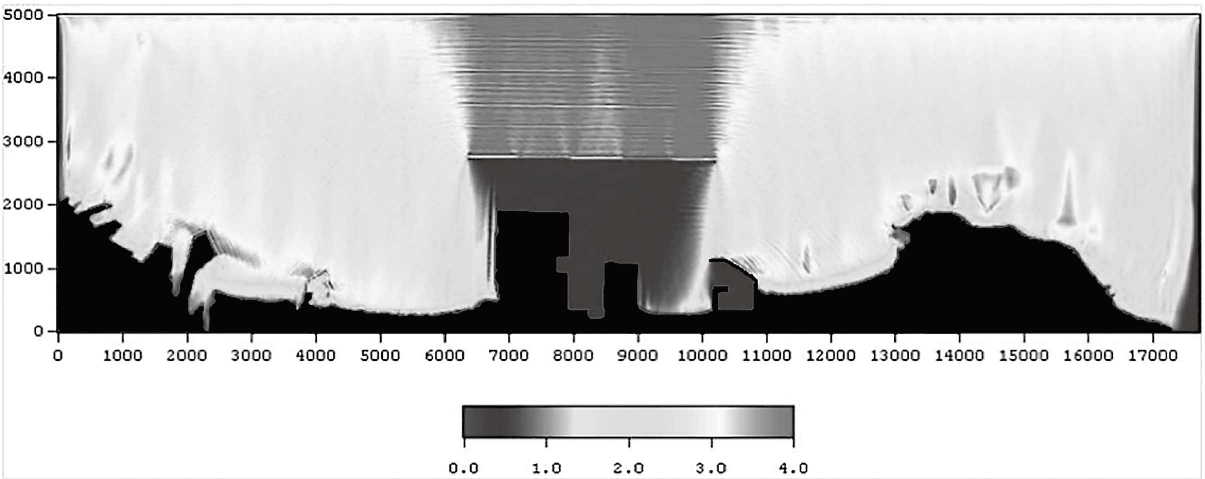


Fig. 2.11.8 Calculation Example Using the Boussinesq Equation of the Wave Height Distribution around the Hitachi Naka Port (Under Construction) (Primary Wave Direction: E, Wave Height: 3 m, Period: 10 s)

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3 Surveys and Tests on Ground

3.1 General

3.1.1 Purpose

The general purpose of surveys and tests on ground is to obtain accurate information on the properties of the ground to ensure the safest and most rational design and execution of the construction, improvement and maintenance of the respective structures, and to enable these structures to effectively fulfill their functions. The ground surveys which have been generally implemented mainly through boring comprise in-situ tests and physical logging using boreholes, sampling and laboratory soil tests, etc. This Chapter also deals with subgrade and base course surveys, pile loading tests and observations during and after construction.

3.2 Ground Survey Plans

3.2.1 Staged Implementation of Surveys

Detailed and accurate information on ground becomes available by appropriately planning staged surveys, from general surveys to detailed surveys, in a manner that determines the locations of the detailed surveys and tests including sampling based on the information on ground conditions obtained through general surveys. The staged implementation of ground surveys also allows project master plans and design policies to be flexibly modified revised, etc. For effective and efficient implementation of ground surveys, it is necessary to have them implemented in stages by survey engineers who have a full understanding of the roles and purposes of the survey work to be implemented by them.

It is important to implement preliminary surveys such as field surveys and collections of information on the surrounding ground as well as existing materials in preparation for the general surveys, as shown in **Fig. 3.2.1**. In addition, planning detailed surveys requires the selection of appropriate boring points based on the distribution of bearing layers and engineering foundation layers used in seismic resistant design, test items, etc. based on an understanding of the ground information necessary for design. An example of the sets of survey and test methods as well as items in the respective survey stages is shown in **Table 3.2.1**.

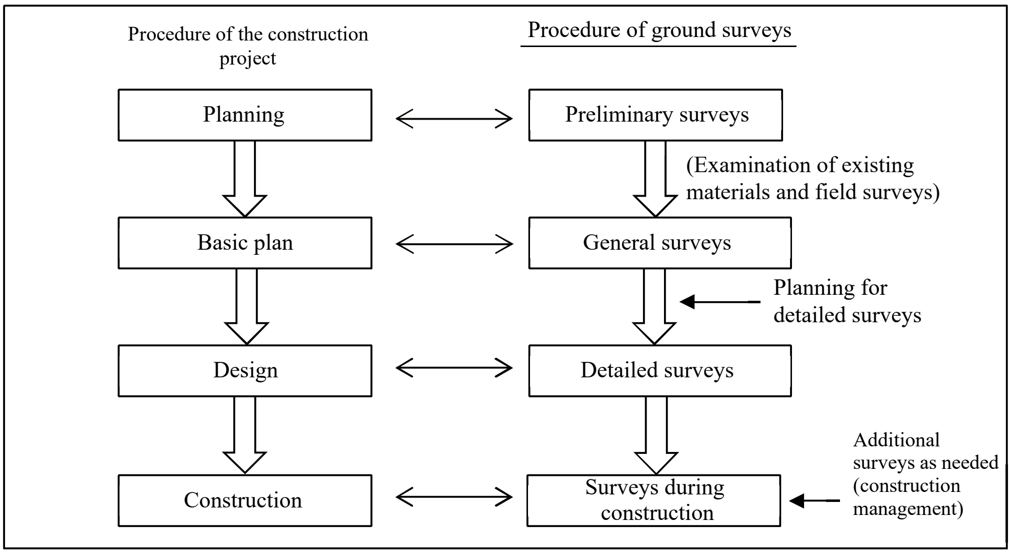


Fig. 3.2.1 Outline of Staged Ground Surveys in Relation to the Construction Project Procedure

Table 3.2.1 Methods and Items for Surveys and Tests in the Respective Survey Stages (Example)

Survey stage	Survey and test method	Item	Purpose
General survey	Boring In-situ test	Standard penetration test Velocity logging (PS logging)	Understanding of ground formation (stratigraphic succession) Understanding of engineering foundation and several elastic constants
Detailed survey	Boring In-site test Sampling Laboratory soil test	Standard penetration test Load test inside boreholes (pressure meter, etc.) Physical, shear and consolidation tests Liquefaction and dynamic deformation tests	Understanding of ground properties (deformation coefficient, strength parameter, consolidation constant, density, etc.) Liquefaction determination (earthquake response analysis) Dynamic deformation analysis

As shown in **Table 3.2.1**, staged surveys are implemented first by general surveys mainly comprised of a standard penetration test and PS logging for the purpose of understanding the ground formation and engineering foundation of an object area, and then are based on the general survey results, by detailed surveys comprising in-situ, sampling and laboratory soil tests for the purpose of obtaining ground information necessary for design.

For example, when examining liquefaction, it is important to implement general surveys in a manner that identifies layers subjected to liquefaction, determines the possibility of liquefaction based on the *SPT-N values* and grain sizes, evaluates the necessity of samplings, cyclic undrained triaxial compression tests, etc., and compiles a report showing the general survey results necessary for planning subsequent detailed surveys. Then, detailed surveys shall be implemented for the comprehensive compilation of data including the general survey results and evaluations as well as an analysis of the ground properties, etc.

3.2.2 Procedure for Establishing Survey Plans

Ground surveys for setting ground conditions are generally based on planning according to the procedure shown in **Fig. 3.2.2**, with due consideration of the structures, scales and importance of the facilities subjected to the technical standards and the properties of the ground around the areas where the facilities are constructed.

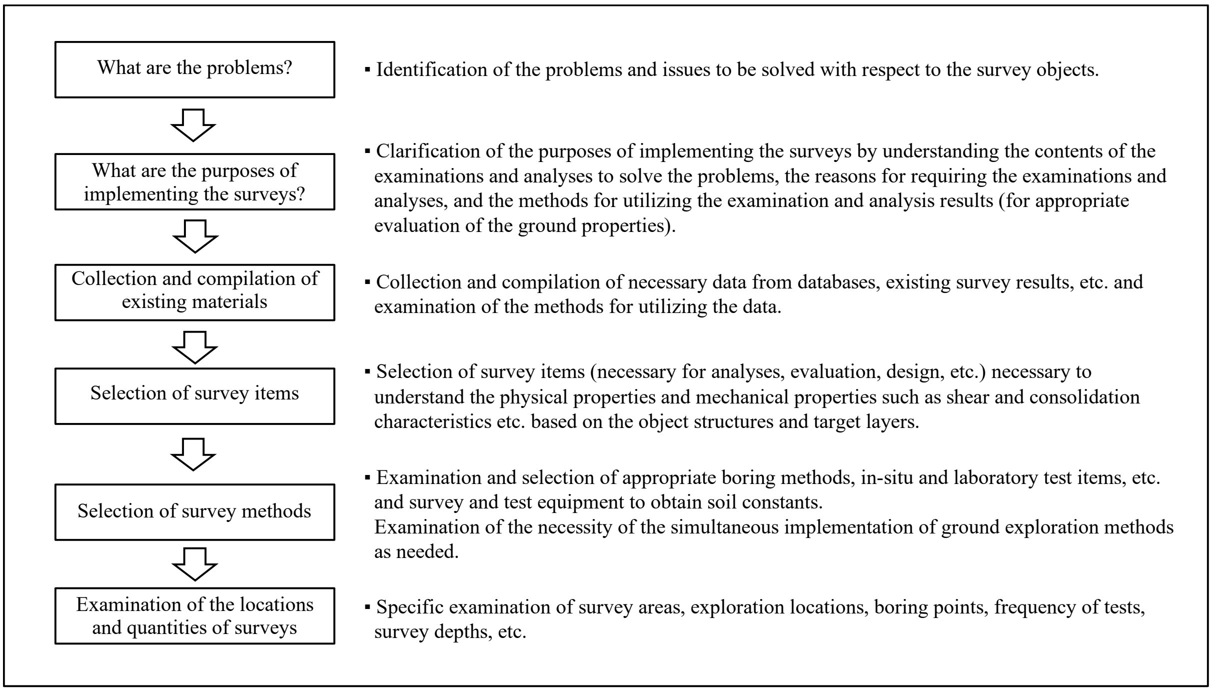


Fig. 3.2.2 Procedure for Establishing Survey Plans

The existing materials necessary for establishing detailed survey plans are mainly the information obtained through past surveys; however, it is also necessary to refer to the boring database, etc. published by the government, municipalities, etc.

3.2.3 Points of Caution When Establishing Survey Plans

The items to be examined when establishing survey plans include the understanding of problems and issues as well as the examination of the methods, items and frequency of surveys and tests to obtain the necessary information on ground. The points of caution when examining the above items are summarized in **Table 3.2.2**.

Table 3.2.2 List of Points of Caution When Establishing Survey Plans

Examination stage	Example point of caution	Remarks
Understanding of problems and issues	○ Circular slip failure ○ Consolidation settlement ○ Liquefaction and lateral flow ○ Stress and deformation due to earthquake ground motions ○ Stability (with respect to sliding, overturning and bearing)	Identification of problems and issues related to ground mainly through an approximate understanding of the ground formation in the object area with reference to existing materials
Identification of necessary ground information	○ Strength parameter (c, φ), unit weight γ_t ○ Consolidation characteristics (consolidation curve, p_c, c_v) ○ Grain size, density, moisture content, liquid limit, plastic limit, liquefaction strength, several elastic constants (V_p, V_s) ○ Liquefaction strength curve, dynamic deformation characteristics	Establishment of survey plans based on reliable data, excluding old, fuzzy or unclear data, etc.
Necessary survey and test methods	○ Boring ○ Soundings ○ Standard penetration tests ○ PS logging ○ Sampling ○ Laboratory soil tests (physical test, uniaxial compression test, direct shear test, simplified CU test, triaxial compression and tensile test, wet density test, consolidation test, cyclic undrained triaxial test)	Appropriate arrangement of test positions and setting of sampling and test intervals in the depth direction, etc. based on existing materials. Appropriate selection of test items in accordance with soil properties and analysis and test conditions.
Frequency of surveys and tests	○ Standard penetration test: general interval of 1 m in the depth direction excluding the sampling position ○ PS logging: general interval of 1 m for soil layers and 2 m for rock layers in the depth direction ○ Sampling: depending on the thickness of the layers	Necessity of setting appropriate sampling positions and frequency based on the layer thicknesses and the distribution tendency of *SPT-N values*

3.2.4 Organization of Systematic Survey Methods

(1) Survey and test items

When implementing ground surveys, it is necessary to select the test items most suitable for the survey purposes with due consideration to the types, scales and importance of the structures, economic performance, the properties of the surrounding ground, etc.. It is also necessary to determine the items and quantities of ground surveys in accordance with the survey stages. In addition, the items, quantities, positions and depths of the surveys and tests, the types and dimensions of the test equipment to be used, and the conditions and procedures of the tests and measurements shall be appropriately determined while taking into consideration the ground models to be set based on the existing materials, etc.

Tables 3.2.3 shows the survey methods suitable for general survey purposes and the information obtainable through the methods, and **Table 3.2.4** shows the standard survey and test items used for the design and construction of port facilities.

Table 3.2.3 General Survey and Test Methods for Different Survey Purposes

Classification	Survey purpose	Survey method	Obtainable information
Stratification condition	Confirmation of stratification condition	Geophysical exploration, boring, soundings	Stratigraphic succession, bearing layer depth, engineering foundation depth, soft layer thickness
Physical properties	Classification of soil	Physical tests with disturbed specimens (undisturbed test pieces for wet density tests)	Wet density ρ_t, moisture content w_n, liquid limit w_L, plastic limit w_P, uniformity coefficient U_c, grain size accumulation curve
Permeability	Evaluation of permeability	In-situ permeability tests, laboratory permeability tests with undisturbed specimens	Permeability coefficient k
Stress condition	Confirmation of stress condition	Pore water pressure measurement, groundwater level observation	Groundwater level in aquifers, pore water pressure p_w
Mechanical properties	Evaluation of bearing capacity, slope stability, earth pressure, subgrade reaction	Shear tests with undisturbed specimens, in-situ tests (soundings)	Wet density ρ_t, unconfined compressive strength q_u, cohesion c, angle of shear resistance ϕ, undrained shear strength c_u, strength increase rate m, indexes of penetration resistance value (*SPT-N value*, W_{SW}, N_{SW}), deformation coefficient E
	Evaluation of consolidation properties	Consolidation tests with undisturbed specimens	Consolidation yield stress p_c, compression coefficient C_c, consolidation coefficient c_v, volume compressibility coefficient m_v, compression curve (e to log p), (permeability coefficient k)
	Evaluation of compaction properties	Compaction tests (disturbed specimens are acceptable)	Compaction curve, maximum dry density ρ_{dmax}, optimum moisture content w_{opt}
	Evaluation of dynamic properties	Dynamic shear tests with undisturbed specimens	Rigidity modulus G, damping constant h, liquefaction characteristics

Table 3.2.4 Standard Ground Survey Items for Port Facilities (In-Situ Tests, Sampling, Laboratory Soil Tests)

Facility name	Examination item	Assumed soil type	In-situ test or sampling method	Physical test	Mechanical test, consolidation test	Remarks
Quaywall/ Breakwater	Sliding, overturning, bearing capacity of foundation ground, slip failure of ground	Sandy soil	Disturbed specimens through standard penetration tests	Degree of roughness, moisture content	Triaxial	Triaxial test on an as-needed basis
		Cohesive soil	Sampling	Degree of roughness, moisture content, wet density, consistency	Uniaxial, triaxial	
	Settlement	Sandy soil	Disturbed specimens through standard penetration tests	Degree of roughness, moisture content		Calculation of instantaneous settlement using a deformation coefficient estimated from *SPT-N values*
		Cohesive soil	Sampling	Degree of roughness, moisture content, wet density, consistency	Consolidation	Estimation of settlement from compression curves of the respective layers Calculation of the time for consolidation settlement using a consolidation coefficient

<table>
<tr><th>Facility name</th><th>Examination item</th><th>Assumed soil type</th><th>In-situ test or sampling method</th><th>Physical test</th><th>Mechanical test, consolidation test</th><th>Remarks</th></tr>
<tr><td rowspan="2"></td><td rowspan="2">Verification in respect to Level 1 and Level 2 earthquake ground motions</td><td>Sandy soil</td><td rowspan="2">Standard penetration tests, PS logging, sampling</td><td>Degree of roughness, moisture content, wet density (consistency in the case of fine fractions of 15% or more)</td><td>Liquefaction, dynamic deformation</td><td rowspan="2">Refer to Part III to determine the necessity for verification.
Only silty cohesive soil is subjected to liquefaction tests.</td></tr>
<tr><td>Cohesive soil</td><td>Degree of roughness, moisture content, wet density, consistency</td><td>Uniaxial, triaxial, liquefaction, dynamic deformation</td></tr>
</table>

(2) Relationship between simulation analyses of earthquake effects and survey items

There are several earthquake response analysis methods for ground. For the details of these methods, refer to **Part II, Chapter 6, 1 Earthquakes** and **Reference [Part III], Chapter 1, 2 Basic Items of Earthquake Response Analyses**. Among these methods, this section describes "SHAKE" and "FLIP," which have been frequently used in port-related design work with focuses on the parameters used in these two methods and the necessary surveys and tests for obtaining these parameters.

Table 3.2.5 Surveys and Tests for Obtaining the Parameters Used in the Equivalent Linear One-Dimensional Analysis "SHAKE"

Type of parameter	Parameter	Survey and test
Physical properties	Wet density	Wet density tests with less disturbed specimens or in-situ density logging
Engineering foundation	S-wave velocity, *SPT-N value*, broad-based layer distribution	Boring, standard penetration tests, PS logging
Dynamic deformation properties	Initial shear rigidity: G_0	In-situ PS logging, laboratory dynamic deformation tests
	Non-linear properties of rigidity: G/G_0 to γ	Dynamic deformation tests with less disturbed specimens
	Non-linear properties of damping constants: h to γ	Dynamic deformation tests with less disturbed specimens

Table 3.2.6 Surveys and Tests for Obtaining the Main Parameters Used in Dynamic Analyses by "FLIP"[1)]

Type of parameter	Parameter*	Survey and test
Physical properties	Wet density: ρ_t	Wet density tests with less disturbed specimens or in-situ density logging
	Porosity: n	Physical tests with less disturbed specimens
Dynamic deformation properties	Initial shear rigidity: G_{ma}	In-situ PS logging, laboratory dynamic deformation tests
	Volume elastic modulus: K_{ma}	Calculations from initial shear rigidity and Poisson ratio
	Angle of shear resistance: ϕ'	Triaxial CD tests or CU-bar tests
	Cohesion: c'	Calculations from triaxial compression test results
	Maximum value of damping constant: h_{max}	Dynamic deformation tests
	Shear stress in steady state: S_{us}	Non-drained shear tests to large strain regions
Liquefaction properties	Phase transformation angle: ϕ_p	Standard values are normally used or can be calculated through the stress paths in liquefaction tests
	Liquefaction parameters: w_1, p_1, p_2, c_1, S_1	Can be calculated through the fitting treatment of the liquefaction test results

* Refer to "**Reference [Part III)], Chapter 1, 2 Basic Items of Earthquake Response Analyses**"

3.2.5 General Frequency of Surveys and Tests

(1) Target intervals of survey points

The intervals of boring, sounding points, etc. shall be determined by taking into consideration the sizes of object facilities, stress distribution in the ground due to the weight of the structures and the homogeneity of the stratification conditions of the ground. It is also necessary to consider the importance, etc. of the object facilities. Thus, there are no general rules to determine the intervals, and in many cases, they have been determined using the values in **Table 3.2.7** as targets. An example of an arrangement of boring points is shown in **Fig. 3.2.3**.

Table 3.2.7 Target Intervals of Boring and Sounding Points

① **For uniform stratification conditions in both horizontal and vertical directions** (Unit: m)

		Face line direction		Normal to face line direction			
		Interval		Interval		Distance from face line (maximum)	
		Boring	Sounding	Boring	Sounding	Boring	Sounding
General survey	Wide survey range	300-500	100-300	50	25	50-100	
	Small survey range	50-100	20-50				
Detailed survey		50-100	20-50	20-30	10-15		

② **For complex stratification conditions** (Unit: m)

	Face line direction		Normal to face line direction			
	Interval		Interval		Distance from face line (maximum)	
	Boring	Sounding	Boring	Sounding	Boring	Sounding
General survey	50 or less	15-20	20-30	10-15	50-100	
Detailed survey	10-30	5-10	10-20	5-10		

Note: There are two types of sounding: one that requires boreholes and the other which does not. The intervals of sounding in the table above are for the type of sounding that does not require boreholes. The values for boring in the above table can be used as the intervals of boring for soundings that require boreholes.

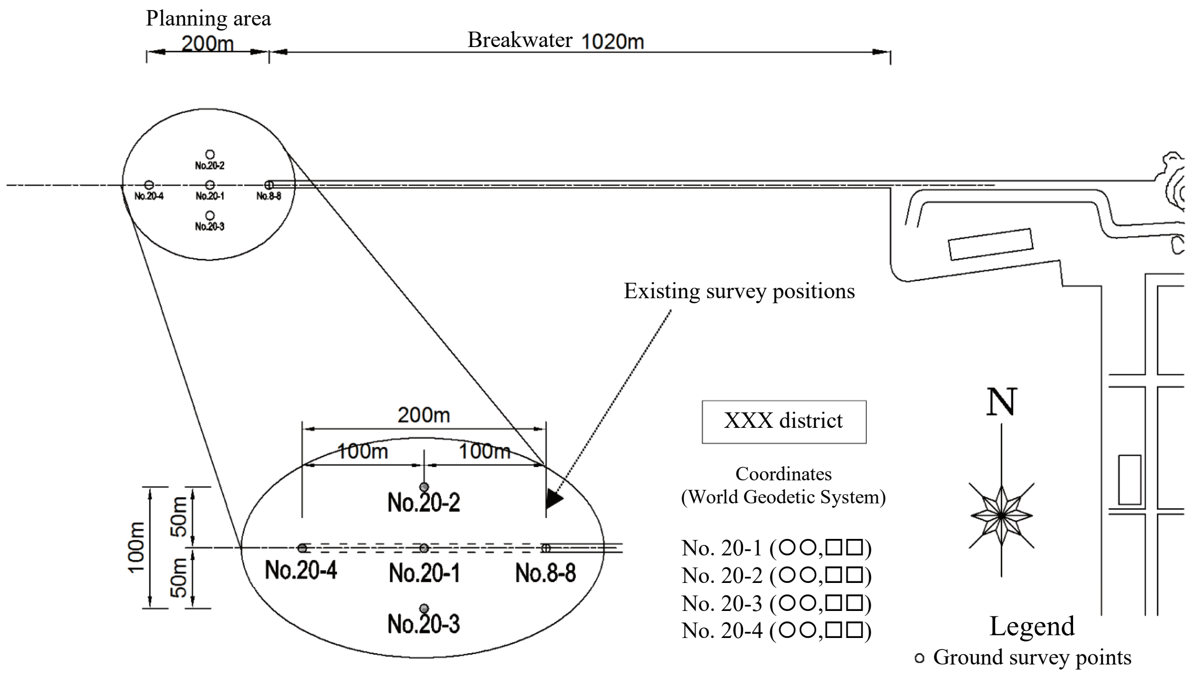


Fig. 3.2.3 Example of an Arrangement of Boring Points (For a Breakwater)

(2) Target frequency of in-situ tests

The target frequency of major in-situ tests is as follows.

1) Standard penetration test

The standard penetration test is implemented by dynamically inserting an SPT sampler into the ground for collecting soil specimens to determine the hardness or softness and compaction degree of the ground, and to identify the stratification compositions of the ground. In many cases, the standard penetration test is implemented at 1 m intervals in the depth direction for sandy soil or other types of ground. In the case of dredging work in which surveys are required only for obtaining the approximate strength indexes and layer thicknesses, the standard penetration test is normally implemented instead of sampling even for cohesive ground.

2) PS logging and density logging

PS logging is implemented to obtain S-wave velocities, among the parameters mainly used in earthquake response calculations, necessary for calculating the shear modulus of rigidity with small strains. PS logging can be applied to all types of ground with standard measurement intervals of 1.0 and 2.0 m in the depth direction for soil ground and rock ground, respectively, in many cases. Density logging is implemented at the same survey points as PS logging to obtain wet density to be used together with the PS logging results for calculating the elastic moduli of the ground.

Table 3.2.8 shows examples of the measurement intervals in the depth direction of other typical soundings and other surveys as well as the points of caution when applying them to actual surveys.

Table 3.2.8 The Intervals of Typical Soundings and Other Surveys and Points of Caution When Applying Them to Actual Surveys

Item	Measurement interval	Outline and points of caution for applying the intervals to actual surveys
Swedish weight sounding test	25 cm	The Swedish weight sounding test is applicable to soft ground up to 15 m deep and not applicable to dense sand layers, ground mixed with gravel or cobble stones, or consolidated ground. Because the test uses a single pipe rod, the test results are inevitably subjected to the influence of skin friction of the peripheral surface of the rod. Therefore, the soil strength estimated from the measurements of W_{sw} and N_{sw} shall be regarded as a basic indication of its approximate tendencies.
Portable cone penetration test	10 cm (less than 10 cm is available in the case of automatic measurements)	There is a tendency for the skin friction of the peripheral surface of the rod to be large in high-plasticity cohesive soil layers and small in silty or fibrous peaty layers. In the case of a single pipe rod, a depth of about 3 m is considered to be the limit for the test to be able to ensure that the influence of the skin friction of the peripheral surface of the rod on the measurements is negligibly small.
Mechanical cone penetration test, electric cone penetration test	As appropriate	Because the mechanical cone penetration test statically presses a cone at the tip of the test equipment into the ground using the reaction force of an anchor installed in the ground, it is difficult to apply it to extremely dense sand, gravel or cobble stone layers, etc. Furthermore, in the case of very soft ground, measurement may become impossible with inner and outer pipes which settle down under their own weight. For other types of ground, the test can measure penetration resistance with uniform accuracy. The mechanical and portable cone penetration tests measure only the cone penetration resistance q_c without considering the influence of pore water pressure. In contrast, the electric cone penetration test measures pore water pressure (and skin friction, in many cases) concurrently with cone penetration resistance. Thus, the electric cone penetration test can obtain a wider range of ground information than the other cone penetration tests and makes available reliable information on the stratification compositions and mechanical properties.

Item	Measurement interval	Outline and points of caution for applying the intervals to actual surveys
In-situ vane shear test	As appropriate	The in-situ vane shear test is applicable to soft cohesive ground, which is mostly cohesive, silty and well degraded organic ground with *SPT-N values* of 2 or less, down to about 15 m deep. However, in many cases, the test is difficult to be applied to cohesive or sandy soil with *SPT-N values* of 4 or more. The test has good performance records in the evaluation of the physical properties of peaty soil as a special case to which the dimensions and rotation speeds different from those specified in the Standards of the Japanese Geotechnical Society introduced in 3.5 are applied. Thus, when applying the test to peaty soft ground, it is necessary to fully examine how to use the vane shear strength obtained through the test. For example, it is necessary to implement another test concurrently with the vane shear test so as to compare and examine the shear strength obtained through the two tests.
Load tests inside boreholes (pressure meter, etc.)	As appropriate	It is very difficult to evaluate the load test results, even when the load test can be implemented, when applied to ground types such as sand gravel ground, which cause borehole walls to have coarse surfaces, or to ground types such as soft cohesive ground, which cause borehole walls to be easily disturbed.
Simplified dynamic cone penetration test	10 cm	Having impact energy smaller than other dynamic penetration tests such as the standard penetration test, the simplified dynamic cone penetration test cannot be applied to hard cohesive or gravel ground, etc. having large penetration resistance. Furthermore, even in ground with low penetration resistance, the skin friction between the peripheral surface of a rod and the ground becomes larger with the increase in penetration depth, thereby making accurate measurements of the penetration resistance difficult. Thus, the simplified dynamic cone penetration test is applicable to superficial layers of about 4 to 5 m from the ground surface.

(3) Target sampling intervals in depth direction

The lengths, numbers and intervals of specimens in sampling shall be determined based on the *SPT-N values* and approximate soil information obtained through general surveys to be implemented before sampling. The intervals of samplings are preferably set at 1.5 to 2.0 m so as to cope with variations in data. When sampling the soil with estimated sand content of 80% or less, the standard sampling intervals are 1.5 m in the major survey spots to understand physical and mechanical characteristics and 1.0 m in the survey spots of particular importance. A sampling interval of 2.0 m is still acceptable when sampling is conducted in supplementary survey spots. Mechanical tests shall be planned in a manner that divides the ground into several layers according to the distribution of ground properties, etc. and collects typical test specimens from each layer. In the case of uniform ground, alternate specimens are collected at intervals of 1.5 m; in other words, specimens collected at intervals of 3 m are the minimum requirement for laboratory tests and it is preferable to collect specimens at shorter intervals so as to cope with variations in the test results. There are also cases of using multiple specimens collected from identical depths as needed to cope with any variations. The sampling intervals can be shortened as needed in the case of complex ground or ground for which consolidation settlement is of particular importance. Furthermore, there may be cases where specimens are collected from thin cohesive layers through additional boreholes prepared next to the main layers.

For sampling intervals in planar directions, it is preferable to conduct sampling at every survey spot; however, the number of sampling spots can be less than that of the survey spots, as appropriate, depending on the importance of the survey work, the uniformity of the object ground, or the types of soil constants, etc. necessary for design.

3.3 Examination of Existing Materials and Field Surveys

3.3.1 Examination of Existing Materials

Ground surveys are preferably implemented in stages from general to detailed surveys so as to enhance their effectiveness and efficiency. For that purpose, first it is important to appropriately collect and organize geographical, ground, environmental ground information, etc. by examining existing materials. That is, the examination of existing materials is to establish the bases of subsequent ground surveys and it is important to collect a wide range of updated information.

The materials to be preliminarily collected include existing boring and sounding data, seabed topography information through hydrographic charts and bathymetry, and geological maps, as well as aerial photographs of the survey object areas and their surroundings. Existing research materials compiling the items required in previous design and construction are the guides from which possible problems and principles of subsequent surveys can be known. Therefore, it is important to establish research and examination plan on the basis of these existing materials.

Construction and disaster records, etc. are also very useful materials. Previous construction work left reports (construction records) which showed the progress and as-built forms of the construction work, photographs of sites and records of accidents, problems, etc., and were stored by project owners or administrators for certain periods after completion (normally about 5 to 10 years). These construction records are very important when implementing regular and soundness inspections for the maintenance of built structures, etc. Furthermore, when defects occur, construction records, together with existing materials, can be used as effective means to find the causes of the defects. In addition, disaster records are effective materials for understanding the topographical and geological vulnerability of the survey object areas, and are important for determining reinforcement or security countermeasures to be taken when designing structures. Thus, it is preferable to proactively utilize existing materials, when available, in order to reasonably and safely develop, maintain and manage infrastructures.

It is important to use existing materials and records after understanding their original purposes and accuracy of the data they contain. Furthermore, attention shall be given to the management of information contained in the existing materials because many of the individual materials, construction and disaster records, etc. have not been made available to public inspection.

3.3.2 Field Surveys

Field surveys shall be conducted in preparation for ground surveys. Generally, the engineering properties of ground are the result of a series of actions over a long period of time, and, therefore, it is difficult to evaluate the engineering properties of ground without understanding their history, including the origin and subsequent actions that the ground has undergone such as stresses, weathering and transformation. Thus, field surveys are conducted for the purpose of comprehensively understanding the basic properties of the object ground based on its origin and history, thereby obtaining information useful to clarify the influence of geological actions on the engineering properties of the object ground.

Onshore field surveys (surficial geology surveys) are generally conducted for clarifying the spatial distribution of the engineering properties of geology and ground by comprehensively determining the results observing the geology exposed to ground surfaces and measuring strikes, dips, etc. of bedding planes and cracks. Onshore field activitiess are the most fundamental surveys among all the types of geological and ground surveys.

Field surveys for offshore object ground cannot be conducted as is generally the case with onshore objective ground, but even sea bottom geology is closely connected with onshore geology nearby. Therefore, even in the case of offshore object ground, field surveys of land close to the object offshore ground are still important. For offshore field surveys, because direct visual observation of the conditions of the bedrock and sea bottoms, etc. is difficult, unlike in the case of onshore field surveys, indirect observation methods such as echo sounding and acoustic exploration have been used in many cases. These observation methods usesurvey boats which enable continuous data to be acquired as they travel through the water. However, the efficiency of field surveys and the quality of data obtained through the field surveys are affected by the operation of survey boats susceptible to hydrographic and meteorological conditions. Therefore, it is necessary to: preliminarily understand the meteorological, tidal and current conditions of areas; examine survey lines and spots with reference to existing materials; and appropriately set them.

Another important item to be examined during offshore field surveys is the confirmation of the local conditions for planning temporary equipment to be used in subsequent ground surveys. These local conditions include the use purposes of object water areas (during normal operation) and the availability of docks for survey boats.

3.4 Boring

Ground surveys for design and construction are generally implemented in stages from preliminary to detailed surveys, etc. Boring which bores holes according to necessities in the ground is often implemented in the general survey stage or at a later stage in combination with samplings and soundings, and is an important survey method element for understanding the compositions of ground, confirming bedrock depths, and obtaining strength and consolidation properties. For effectively and efficiently implementing necessary and sufficient quantities of boring, detailed implementation plans shall be established while taking into consideration not only the compositions of the ground estimated through preliminary surveys but also the types, scale, importance, etc. of the construction works. Considering that there have been many cases where the foundations of structures or structural types were forced to be revised due to structural designs based on insufficient survey results, it is necessary to fully examine the implementation plans so as to avoid redoing the design or construction in the future. It is also necessary to pay attention to the diameters of boreholes which vary depending on the types of samplings and soundings to be implemented. **Table 3.4.1** lists the survey methods which use boreholes and the required diameters of the boreholes.

Table 3.4.1 Survey Methods Using Boreholes and the Required Diameters of the Boreholes

Standard No.	Name of in-situ test and sampling method	Borehole diameter (mm)				Survey point in borehole
		66	86	116	146 or more	
JIS A 1219	Method for standard penetration test	○	○	○	△	Bottom
JGS 1121	Method for electrical logging	○	○	○	○	Side wall
JGS 1122	Method for seismic velocity logging	○	○	○		Side wall
JGS 1221	Method for obtaining soil samples using thin-walled tube sampler with fixed piston		○	○		Bottom
JGS 1222	Method for obtaining soil samples using rotary double-tube sampler			○		Bottom
JGS 1223	Method for obtaining soil samples using rotary triple-tube sampler			○		Bottom
JGS 1224	Method for obtaining samples using rotary double-tube sampler with sleeve	○	○	△		Bottom
JGS 3211	Method for obtaining soft rock samples by rotary tube sampling	○	○	△		Bottom
JGS 1311	Method for measuring groundwater level in borehole	○	○	○	○	Bottom and side wall
JGS 1312	Method for measuring groundwater level in well			○		Bottom and side wall
JGS 1313	Method for measuring pore water pressure using electric transducer in borehole		○	○		Bottom and side wall
JGS 1314	Method for determination of hydraulic properties of aquifer in single borehole		○	○		Bottom and side wall
JGS 1315	Method for pumping test			○	○	Bottom and side wall
JGS 1321	Method for determination of hydraulic properties of rock mass using instantaneous head recovery technique in single borehole	○	○			Bottom and side wall
JGS 1322	Method for determination of hydraulic conductivity of rock mass using injection technique in single borehole	○	○	○		Bottom and side wall
JGS 1323	Method for lugeon test	○				Side wall
JGS 1411	Method for field vane shear test		○	○		Bottom
JGS 1531	Pressuremeter test for index evaluation of the ground	○	○			Side wall
JGS 3531	Pressuremeter test to evaluate mechanical properties of the ground	○				Side wall
JGS 1731	Method for measuring ground movement using strain gauge	○				Side wall

JGS: The standards of the Japanese Geotechnical Society, ○: Typical, △: Rare

3.4.1 Types and Methods of Boring

Rotary, auger, percussion, etc. are the types of boring used for ground surveys, with rotary type boring being the most commonly used. Rotary type boring is a method for drilling holes in a manner that rotates a bit while pressing it against the ground with a moderate amount of force through the rotary and feed drive of the boring machine. The rotary type can drill holes in all types of ground by selecting the appropriate bit. For rotary type boring, there are two types of feeding mechanisms: a hand (or manual) feeding mechanism and a hydraulic feeding mechanism. Rotary type boring using a hand feeding mechanism is currently rarely used, except for some cases of relatively shallow soil surveys, and can be said to be a method that requires artisanal skills in that operators can feel the difference in soil types through their hands when manually feeding the bit. In contrast, rotary type boring using a hydraulic feeding mechanism can reduce the burden on the operators compared to hand feeding mechanism and has been used for drilling boreholes at relatively large depths or in rock ground because of its good operational performance.

The items requiring attention when selecting the types and methods of boring and implementing the selected boring at sites are as follows.

① **Drilling capacity**: borehole diameters, depths, soil conditions and bit types

② **Logging, in-situ tests and field measurements**: the purposes and types of surveys

③ **Sampling**: the purposes of sampling and the types of samplers

④ **Mobilization and transportation of equipment**: opographic conditions of above sea level, etc. and the use of mobilization routes (landing fields, cargo handling yards, navigation channels, etc.)

⑤ **Work space**: presence or absence of neighboring structures

⑥ **Temporary scaffolds**: topographic and work conditions as well as withstand loads

⑦ **Boring water**: boring pumps, discharge rates and cooling bits

⑧ **Protection and cleaning of borehole walls**: casing tubes, treatment of drilling fluid and removal of slime

⑨ **Noise and vibration**: effect on neighboring residential areas

For surveys to confirm the bearing layers, it is necessary to have enough spare rods in case the bearing layers cannot be confirmed at the anticipated depths.

Important items requiring attention during drilling work are the protection and cleaning of borehole walls and the prevention of issues with drilling. The boreholes need to be protected from wall collapses by using the appropriate drilling fluid, etc. and shall be cleaned by discharging slime accumulated at the bottoms of boreholes, thereby allowing sampling and in-situ tests to be appropriately implemented. The issues affecting drilling work include core clogging, jamming, collapsing of borehole walls, bending of boreholes, spring water, and tools falling into boreholes.

In ground surveys, the appropriate boring methods shall be selected in accordance with the survey purposes and methods. In addition, boring machines and peripheral equipment shall be selected and prepared by comprehensively determining the types, borehole diameters, depths and quantities of samplings and soundings, the topography of the survey points, the anticipated ground conditions, etc.

(1) Drilling methods

Drilling methods are classified into core boring, where drilled soil and rock is taken out of the boreholes in the form of cores stored in core barrels, and non-core boring, where all drilled soil is turned into slime in the boreholes and discharged.

Table 3.4.2 summarizes the drilling methods for rotary type boring, which has been widely used. **Table 3.4.3** shows the applicability of the boring methods for sampling, logging, in-situ tests and field measuring.

Table 3.4.2 Drilling Methods for Rotary Type Boring

Name of drilling method	Drilling mechanism	Method for identifying layer boundary	Applicable ground
Core boring	A core is sampled from a borehole drilled with a bit at the tip of a core barrel and rotated. A continuous core can be sampled by alternately drilling a core and cleaning the inside of the borehole.	Observation of the sampled core.	Applicable to soil and rock by selecting an appropriate core barrel but not applicable to gravel and cobble ground. Optimal for rock ground.
Core boring (wire line boring)	A borehole is drilled with a bit at the tip of a wire line rod and rotated. A specimen is stored in an inner tube and collected using a wire rope.	Determined through the drilling rate and slime in the drilling fluid. Observation of the sampled core.	Applicable to soil and rock by selecting an appropriate core barrel but not applicable to gravel and cobble ground.
Non-core boring	A borehole is drilled in a manner that crushes the ground with a bit at the tip of a rod which is rotated. The borehole wall is stabilized with a drilling fluid (mud water etc.). Slime is discharged while circulating the drilling fluid.	Determined through the drilling rate and slime in the drilling fluid. Also, felt through a lever used to rotate the bit in the case of boring with a hand feeding mechanism.	Applicable to all types of soil and rock but not to boulders and cobble ground.

Table 3.4.3 Applicability of the Boring Methods to Sampling, Logging, In-Situ Tests and Field Measuring

Purpose of survey \ Drilling method		Core boring	Non-core boring	Wire line boring
Sampling	Acquisition of less disturbed specimens at relatively shallow survey points	○	◎	
	Acquisition of less disturbed specimens at survey points of general depths	○	◎	
	Acquisition of less disturbed specimens at particularly deep survey points	○	○	◎
	Acquisition of rock cores, etc.	◎		◎
Logging, in-situ test, field measuring	Standard penetration test	◎	◎	
	Electric logging, PS logging, density logging	◎	◎	
	In-situ permeability test, ground water level observation	◎	◎	
	Pumping test	◎	◎	
	Load tests inside boreholes (pressure meter, etc.)	◎	◎	
	Measurement of pore water pressure	◎	◎	
	Measurement using strain gauges	◎	◎	

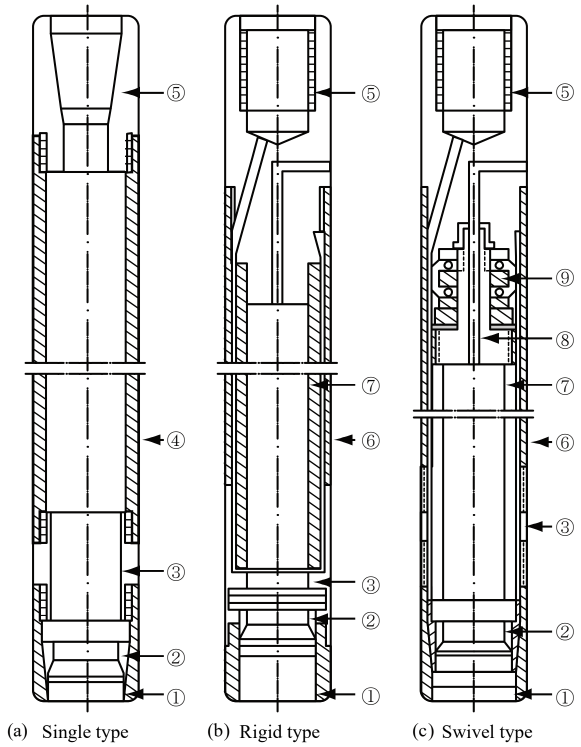
◎: Optimal, ○: Applicable

① **Core boring**

Core boring is a method for drilling a borehole while storing the drilled soil and rock as a core specimen in a core barrel (also called a "core tube"), which is attached to the lower section of the boring rod. There are two types of core barrels: single tube core barrels and double tube core barrels.

In most cases, the double tube core barrel has an inner tube with a swivel structure to keep the core specimen from being affected by the rotation motion of the core bit and so it is not to rotate with the outer tube. The swivel structure has also been applied to double tube and triple tube samplers which are for drilling boreholes using a rotation motion while collecting soil specimens. The types of core barrels (**Boring core tube: JIS M 1407**) are shown in **Fig. 3.4.1**. The double tube core barrel is classified into a rigid type with an inner tube directly linked to the core barrel head, and a swivel type, mentioned above, which has ball bearings, etc. at the head section of the inner tube to prevent it from being rotated with the outer tube.

Wire line boring is a type of core sampling using a double tube core barrel, enabling only the inner tube to be pulled out of the core barrel through a wire from the ground surface during drilling work. Wire line boring has been used for collecting core specimens at large depths because it does not require operation to move the boring rod up and down when collecting core specimens. For example, the ground survey conducted for the development of the Kansai International Airport used wire line boring, making use of its high applicability to deep boring.



① Core bit
② Core lifter
③ Reaming shell
④ Core tube
⑤ Core barrel head
⑥ Outer tube
⑦ Inner tube
⑧ Inner tube head
⑨ Ball bearing

Fig. 3.4.1 Types of Core Barrels[2)]

② **Non-core boring**

Non-core boring is a method for drilling a borehole with all the drilled soil and rock discharged out of the borehole as slime together with mud water circulated through the borehole. Non-core boring can effectively drill boreholes at a high drilling rate without frequently moving the boring rod up and down when applied to soft or loose ground.

(2) Borehole wall protection methods

The borehole wall protection methods are classified into a core tube method, which uses viscous mud water produced by mixing bentonite, etc. with drilling fluid, and a steel casing pipe method, which uses a casing pipe inserted into the ground to protect the borehole wall. The advantages and disadvantages of these methods are summarized in **Table 3.4.4**.

① **Core tube method using mud water**

The core tube method is to drill a borehole by rotating a core tube, which has a rod with a bit attached to its lower tip, while supplying drilling water to protect the borehole wall. This method requires a core tube to be entirely taken out of the borehole every time a specimen is collected or an in-situ test is conducted.

② **Casing pipe method**

The casing pipe method is for installing a casing to an upper limit of the target depth where a sampling or an in-situ test is to be conducted in a manner that drills a borehole or drives the casing by force while joining casing pipes having an inner diameter that allows a sampler or sonde measurement, etc. for in-situ tests to be lowered (with a blade shoe at the lower tip of the casing pipe). Because the casing pipes protect the borehole wall, the method is capable of collecting less disturbed specimens or conducting in-situ tests with less disturbed soil through the casing pipes, thereby enabling survey work to be effectively executed, reliably removing slime from the borehole, and preventing accidents due to drilling tools left in the borehole.

Table 3.4.4 Advantages and Disadvantages of Borehole Wall Protection Methods

Method	Advantages	Disadvantages
Core tube method using mud water	① Use of lightweight equipment (a) Relatively easy to determine soil properties through touch (b) Availability of hand feeding ② Use of a pump with a small capacity (about 40 L/min)	① Risk of abnormally high water pressure on soil in the walls and bottoms of boreholes due to the use of mud water for borehole wall protection ② Risk of inaccurate determination of soil properties due to the use of mud water ③ Risk of wall failure or collapse due to incomplete borehole wall protection
Casing pipe method	① Complete protection of borehole walls ② Reliable execution of pressing the sampler vertically into the ground ③ Relatively accurate determination of soil properties because mud water is not used ④ Effective survey work that does not use drilling tools that move up and down	① Heavy equipment ② Necessity of a pump with a large capacity (about 50 to 60 L/min) ③ Installation for casings in gravel ground, etc. by drilling is difficult and it may be necessary to drive them into the ground and spend significant time removing gravel from inside the casing through a core tube

3.4.2 Types of Boring Materials and Equipment

Rotary type boring using a hydraulic feeding mechanism is the most common method and has been frequently used for ground surveys, in contrast to the type using a hand feed mechanism, which has been rarely used. In addition, the vibration type boring machine, configured to press a sampler into the ground using high frequency vibrations, has been used for sampling in many environmental surveys. The types of boring machines and boring tools are shown in **Figs. 3.4.2** and **3.4.3**, respectively.

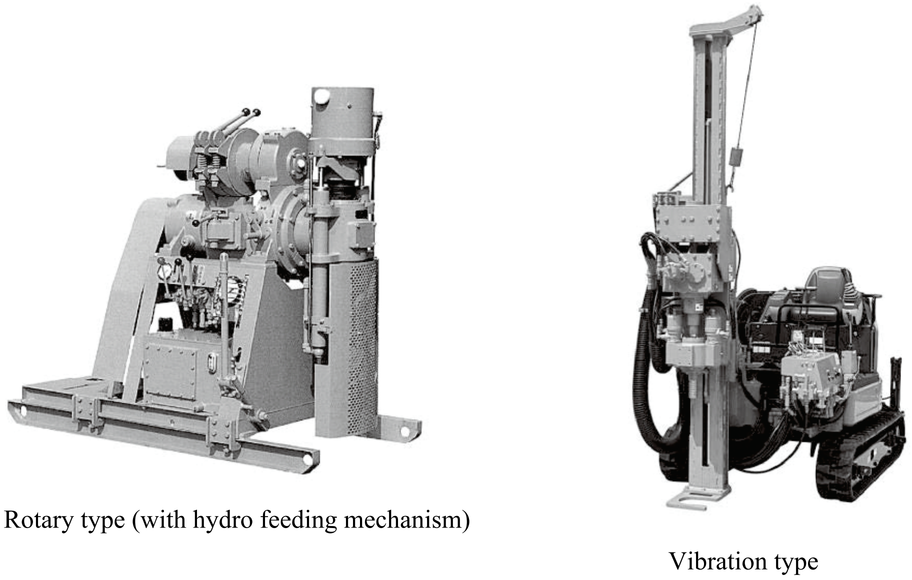


Fig. 3.4.2 Types of Boring Machines

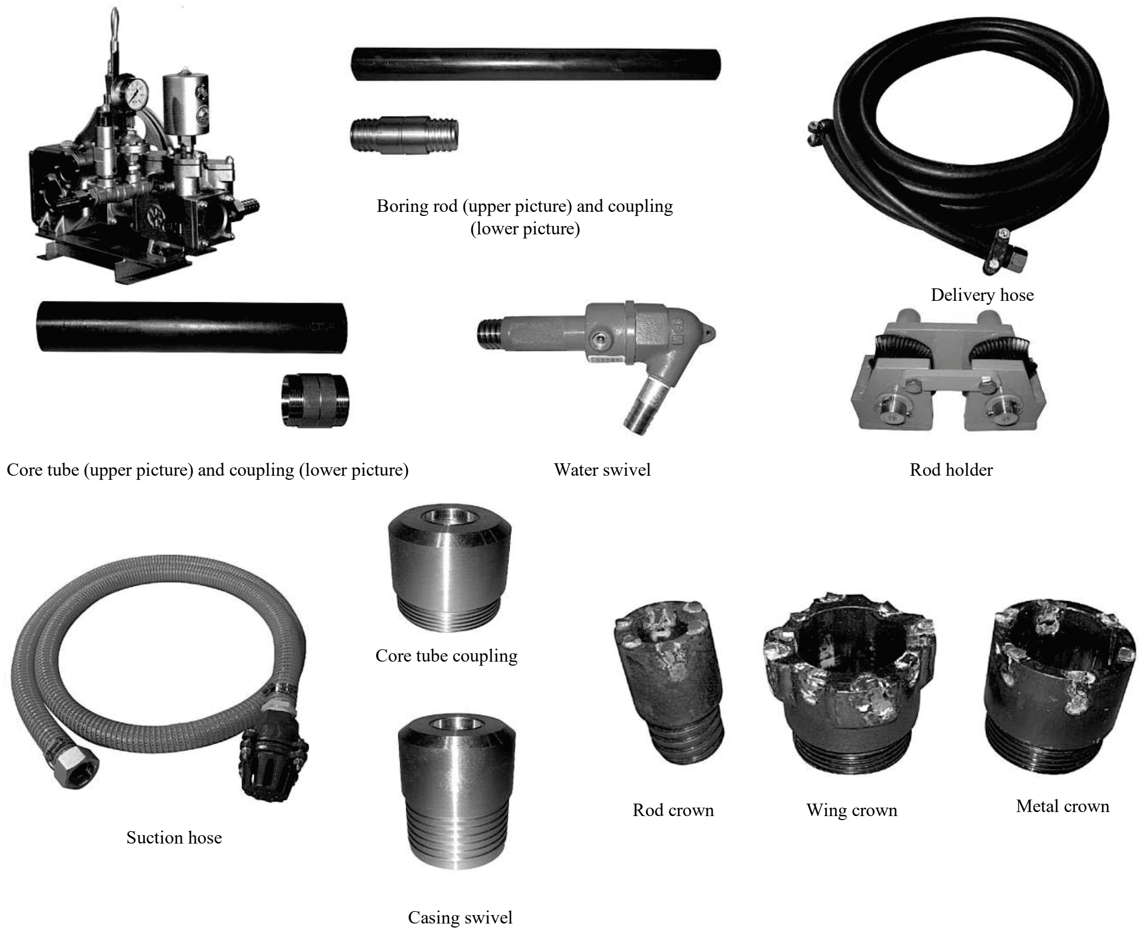


Fig. 3.4.3 Boring Tools

3.5 In-Situ Tests

This section deals with several types of in-situ tests, including those conducted inside boreholes such as load tests inside boreholes, physical logging and pore water measurements, in addition to soundings using a cone, etc. directly inserted into the ground. Those in-situ tests which have already been standardized are listed in **Table 3.5.1**.

Table 3.5.1 Major Standardized In-Situ Tests

Standard	Test method
Japanese Industrial Standard (JIS)	JIS A 1214: Test method for soil density by the sand replacement method
	JIS A 1215: Method for plate load test on soils for road
	JIS A 1219: Method for standard penetration test (*)
	JIS A 1220: Method for mechanical cone penetration test (○)
	JIS A 1221: Method for Swedish weight sounding test (○)
	JIS A 1222: Test method for the California Bearing Ratio (CBR) of in-situ soil
Standards of the Japanese Geotechnical Society (JGS)	JGS 1411: Method for field vane shear test (○)
	JGS 1521: Method for plate load test
	JGS 1531: Pressuremeter test for index evaluation of the ground (*)
	JGS 3531: Pressuremeter test to evaluate mechanical properties of the ground (*)
	JGS 3532: Method for borehole jack test (*)
	JGS 1431: Method for portable cone penetration test (○)
	JGS 1433: Method for portable dynamic cone penetration test (○)
	JGS 1435: Method for electric cone penetration test (○)
	JGS 1311: Method for measuring groundwater level in borehole (*)
	JGS 1313: Method for measuring pore water pressure using electric transducer in borehole(*)
	JGS 1314: Method for determination of hydraulic properties of aquifer in single borehole (*)
	JGS 1122: Method for seismic velocity logging (*)
	JGS 1121: Method for electrical logging (*)

The test methods with (○) and (*) are those that do not use boreholes and that do use boreholes, respectively. Tests with no symbol are those conducted on surface layers.

3.5.1 Soundings

Soundings can be largely classified into static soundings and dynamic soundings from the viewpoint of measurement and operation. These classifications are shown in **Table 3.5.2**.

Static soundings are to measure the resistance of ground when resisters are basically penetrated into, expanded or rotated at constant rates, and include the Swedish weight sounding test, portable cone penetration test, mechanical cone penetration test (Dutch double-tube cone penetration test), electric cone penetration test and vane shear test. Although not classified as soundings in this section, load tests inside boreholes such as pressure meter tests can be included in static soundings.

In contrast, dynamic soundings, also called “impact type penetration tests,” measure the resistance of ground in terms of the number of impacts of drop hammers, etc. required for the resisters to be pressed into the ground by the predetermined depths. Typical dynamic soundings include the standard penetration and dynamic cone penetration tests.

Table 3.5.2 Classifications of Soundings

Method	Operation	Resister	Rod
Static	Pressing	Cone	Single tube, double tube and differential double tube
	Weight loading and rapid rotation	Screw point, pyramid	Single tube
	Very slow rotation	Vane	Single tube, double tube
Dynamic	Impact of drop hammer	Cone, SPT sampler	Single tube, double tube

For the type of soundings to measure the penetration resistance of ground with a cone, etc. pressed into the ground, it is not necessary to consider the effects of the mechanical disturbance on the specimens or stress release due to the sampling of test results, unlike in the case of laboratory tests. In the case of the standard penetration test and loading (pressure meter) test in boreholes which require boreholes to be drilled to the target test depths, the states of the bottoms and walls of the boreholes largely affect the test results.

Soundings cannot make specimens available for visual confirmation but are advantageous in that they enable specimens to be tested under in-situ confined pressure. Although the in-situ confined pressure in a vertical direction can be calculated from unit weight and groundwater levels, it is difficult to accurately know the in-situ confined pressure in horizontal directions (that is, earth pressure at rest). Thus, it is difficult to completely reproduce in-situ stress states in laboratory tests.

In soundings, the properties of the object ground are obtained based on whether the object ground is classified as typical sand or cohesive soil, etc. by using relational expressions between the characteristic values of ground obtained through other survey methods and the sampling results, and figures and tables showing the direct relationships of the design values based on previous experience and performance records with the sounding results. Typical examples of these soundings are the estimations of angles of shear resistance and the bearing capacity of piles, and the determination of liquefaction using the results of standard penetration tests. **Table 3.5.1** shows standardized soundings. There are cases of implementing other types of soundings such as dynamic penetration and in-situ shear friction tests which have not yet been standardized.

When applying soundings to ground surveys, it is necessary to select equipment in accordance with the use of the sounding results, applicability to the object ground, penetration capacity and economic performance, etc., while taking into consideration the fact that both the dynamic and static methods have limitations in their applicability. There may be cases where some soundings can be combined depending on the purposes of ground surveys and survey stages. Respective soundings have their own characteristics as shown in **Table 3.5.3**. In ground surveys, it is necessary to select soundings optimal for the object ground in consideration of economic performance. In **Table 3.5.3**, loading tests in boreholes, such as pressure meter tests, are also shown for reference.

Table 3.5.3 Methods, Characteristics and Applicable Ground for Soundings

Method	Name	Continuity	Measurement	Use of measurement results and estimation method	Applicable ground	Applicable depth (m)	Characteristics
Static	Swedish weight sounding test	Discontinuous	Settlement by respective loads (W_{sw}), the number of half-turns for a penetration depth of 1 m (N_{sw})	*SPT-N value* of standard penetration test, unconfined compression strength, bearing capacity, etc.	All ground except cobble and gravel ground	About 15 m	Simple test procedure compared to the standard penetration test
	Portable cone penetration test	Continuous	Load, cone penetration resistance q_c	Unconfined compressive strength of cohesive soil, undrained shear strength	Cohesive and humus ground	About 5 m	Simplified and extremely fast test procedure
	Electric cone penetration test	Continuous	Cone penetration resistance q_{ct}, skin friction f_s, pore water pressure u	Undrained shear strength of cohesive soil, determination of soil properties, consolidation properties, etc.	Cohesive and sandy ground	Depending on the capacity of penetration and fixing equipment	Availability of highly reliable data
	In-situ vane shear test	Discontinuous	Rotation angle measuring torque	Undrained shear strength of cohesive soil, shear strength of remolded soil, sensitivity ratio	Soft cohesive ground	About 15 m	Direct measurement of c_u value of soft cohesive soil

Method	Name	Continuity	Measurement	Use of measurement results and estimation method	Applicable ground	Applicable depth (m)	Characteristics
	Load test inside borehole (pressure meter, etc.)	Discontinuous	Pressure, borehole wall displacement, creep	Deformation coefficient, coefficient of horizontal subgrade reaction, undrained shear strength of cohesive soil, etc.	All ground, provided that boreholes can maintain smooth and stable walls, and rock ground	Basically no limit	Availability of estimated values with clear mechanical definitions
Dynamic	Standard penetration test	Discontinuous, minimum measurement interval of 50 cm	*SPT-N value* (the number of impacts of drop hammers for predetermined penetration)	Unconfined compressive strength of cohesive soil, relative density of sandy soil, angle of shear resistance, shear rigidity, bearing capacity, etc.	All ground, except ground mixed with cobbles and boulders	Increase in impact energy loss in proportion to depth	High versatile method employed in almost all ground surveys
	Simplified dynamic cone penetration test	Continuous	N_d (the number of impacts of drop hammers for predetermined penetration)	*SPT-N value* and unconfined compressive strength of cohesive soil	Same as above	About 4 to 5 m (a rod may get stuck in the ground due to skin friction becoming larger in proportion to the depth)	Simple test procedure compared to the standard penetration test

Soundings are generally implemented for the purpose of estimating the characteristic values of ground, supplementing discontinued boring data and collecting information on the ground conditions of wide areas, etc., and are advantageous compared to laboratory tests conducted after boring and sampling in that they can be implemented under in-situ confined pressure through economical and simplified procedures, etc. The points of caution when implementing soundings are fourfold, as shown below.

① Dynamic soundings are not suitable for extremely soft cohesive ground.

② Static soundings are not suitable for dense gravel and hard cohesive soil.

③ Cone penetration tests, etc. are desirable when the object ground is considered to have complex properties or when continuous ground information is required.

④ It is difficult to apply soundings to ground that contains many cobbles, boulders, gravel, etc. Even when soundings can be implemented for such ground, the sounding results have a risk of overestimation and require correction in many cases. For example, the *SPT-N value*s of rudaceous soil obtained through soundings need correction based on careful review of the relationship between the number of impacts and the penetration depths.

In addition to standardized soundings, there are other types of soundings including the radio isotope (RI) cone penetration test as a variation of the static cone penetration test and liquefaction potential sounding as a variation of the dynamic cone penetration test.

(1) Radio isotope (RI) cone penetration test[2)]

The RI cone penetration test is a variation of the electric cone penetration test and is capable of measuring wet density using the RI method when a cone is penetrated and moisture content, in addition to the base resistance when a cone is penetrated, skin friction and pore water pressure. Self-propelled special cone penetration vehicles are basically used when penetrating cone probe groups into the ground, but boring machines or contaminated soil sampling machines, etc. may also be used.

(2) Double sounding survey (DS)

The double sounding survey (DS) is a test method which combines static and dynamic methods in a manner that applies a static press-in test method, the electric cone penetration test (CPT), to soft soil layers (with *SPT-N values* of less than 40 in the case of sandy soil, or *SPT-N values* alues of less than 20 in the case of cohesive soil) and a dynamic penetration test method, the standard penetration test (SPT), to hard soil layers which are too hard for CPT to be implemented.

(3) Liquefaction potential sounding (using a piezo drive cone [PDC])[2)]

Liquefaction potential sounding is an in-situ test method capable of easily evaluating the liquefaction strength of ground by estimating the ground water levels and soil classifications at the respective measurement depths based on measurements of the dynamic penetration resistance of ground and excess pore water pressure in the ground around the tip of a cone dynamically penetrated into the ground.

(4) Automatic ram sounding[2)]

Automatic ram sounding, which is a variation of the dynamic cone penetration test developed in Sweden, measures the number of impacts applied to a cone with a ram having a weight of 63.5 kg dropped from a height of 500 mm for every penetration depth of 200 mm into the object ground. A lightweight version of automatic ram sounding (called "mini ram sounding") with an identical system configuration has been developed in Japan. Recently, there have been many cases of applying automatic ram sounding to the ground of residential areas (to confirm the embedded depths of foundation piles, etc.). There has also been development of automatic ram sounding equipment which can collect specimens.

3.5.2 Loading Tests inside Boreholes

(1) Pressure meter and borehole jack tests

These tests apply pressure to borehole walls and measure the pressure on walls and their displacement and have been frequently used for obtaining displacement properties required for the examination of subgrade reaction acting on piles, sheet piles, etc. and deformation analyses, etc.. **Table 3.5.4** summarizes the outlines of the uniformly distributed loading type pressure meter test and the uniform displacement loading type borehole jack test, which have been used as variations of the horizontal loading tests inside boreholes.

These tests shall be conducted at locations within ranges that have dominant effects on subgrade reaction. For piles, the range shall be from ground surfaces to the depths corresponding to the characteristic lengths ($1/\beta$) of the piles (refer to **Part III, Chapter 2, 3.4.7 Calculation of Pile Deflection by Chang's Method**). These tests shall also be conducted at each layer when multiple layers are distributed in the range of $1/\beta$ from ground surfaces or when *SPT-N values* show different distribution patterns even in identical layers, etc..

Table 3.5.4 Outlines of Load Tests inside Boreholes (Pressure Meters, etc.)

Test method	Standard No.	Main available information	Main equipment type
Pressuremeter test for index evaluation of the ground	JGS 1531	Deformation coefficient, yield pressure, ultimate pressure	LLT (single chamber type), elastometer (single chamber type), pressiometer (triple chamber type)
Pressuremeter test to evaluate mechanical properties of the ground	JGS 3531	Initial pressure, shear rigidity, etc.	
Method for borehole jack test	JGS 3532	Deformation coefficient, yield pressure, coefficient of subgrade reaction	KKT

The pressure meter test is conducted with a probe inserted in the borehole, as shown in **Fig. 3.5.1**. There are two types of probes: a single chamber type comprising a rubber measuring cell and a triple chamber type comprising a measuring main cell and upper and lower guard cells. As shown in **Fig. 3.5.2**, the equipment for the pressure meter test is also classified into a self-boring type using a probe with test hole drilling capability, and a pre-boring type that inserts a probe without test hole drilling capability into a preliminarily drilled test hole (borehole).

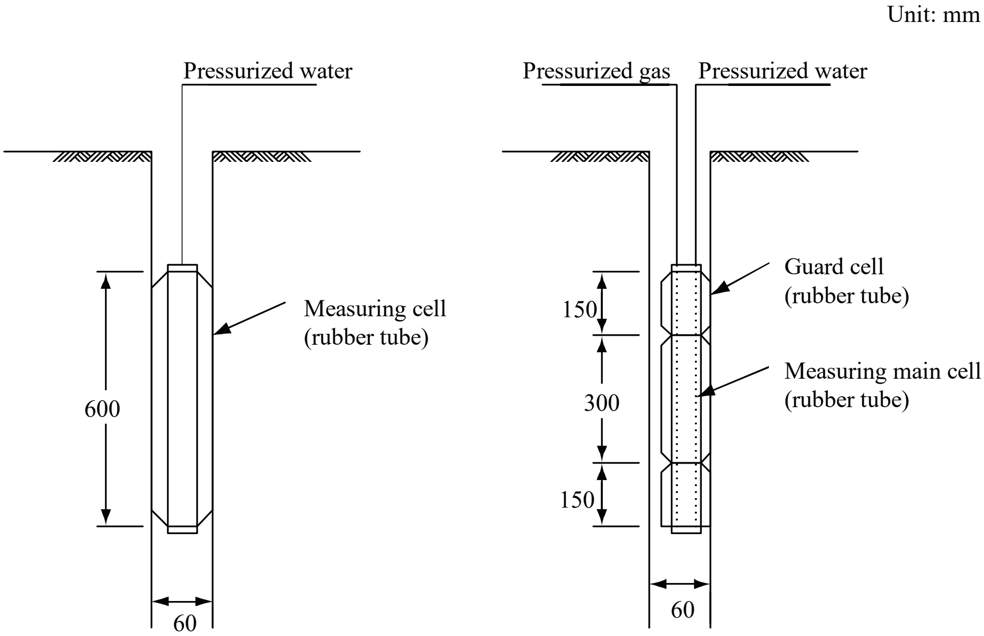


Fig. 3.5.1 Types of Probes[2)] (Left: Single Chamber Type, Right: Triple Chamber Type)

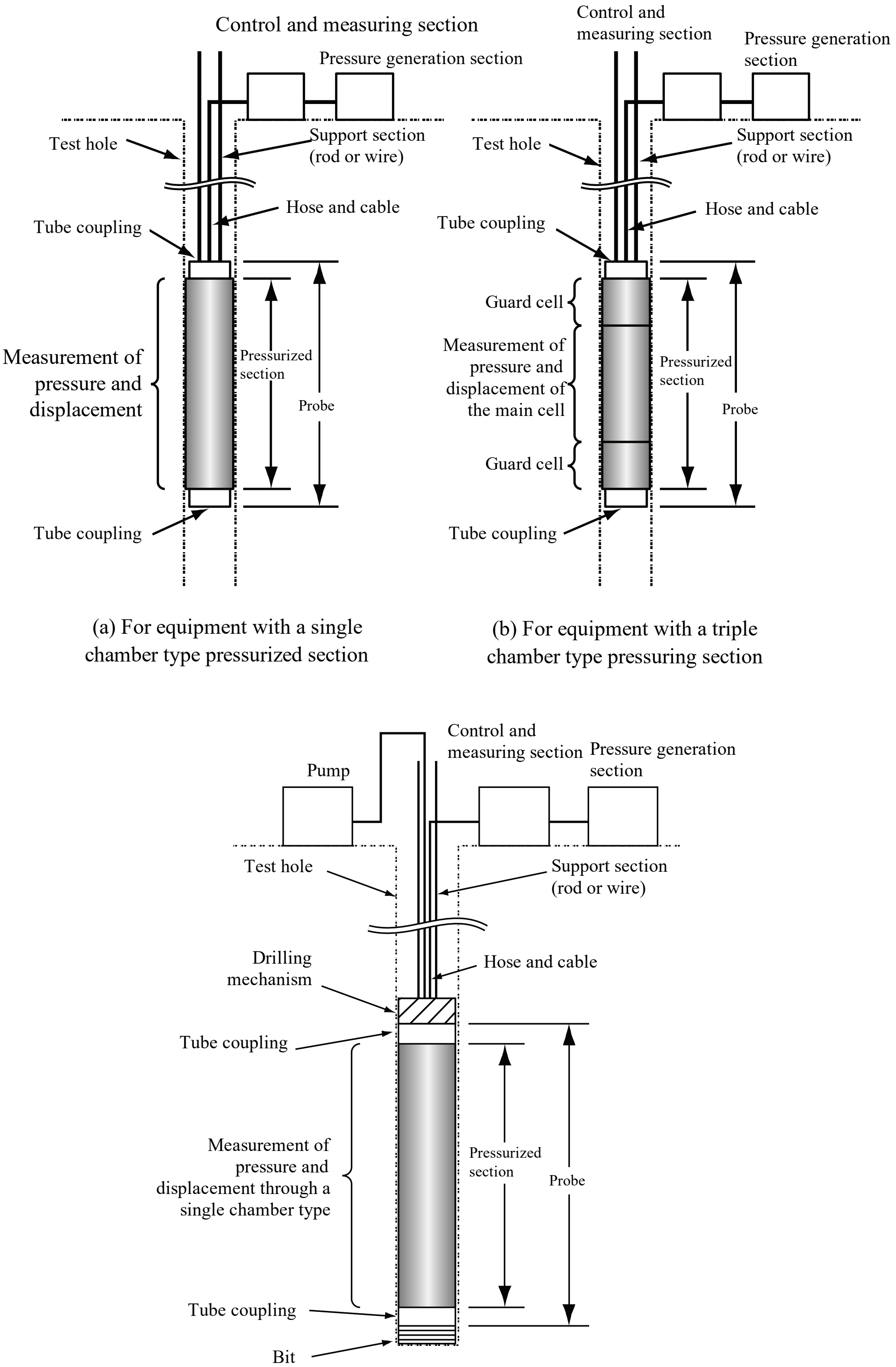
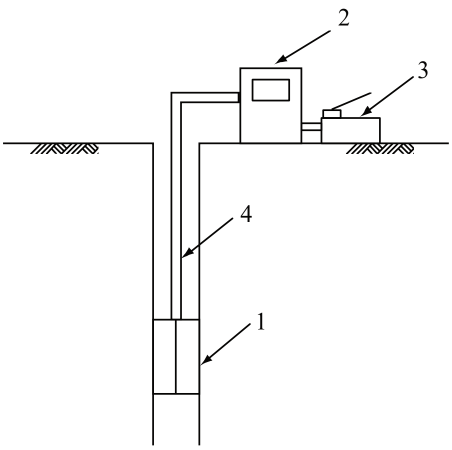


Fig. 3.5.2 Types of Pressure Meter Test Equipment[2)] (Upper: Pre-Boring Types, Lower: Self-Boring Type)

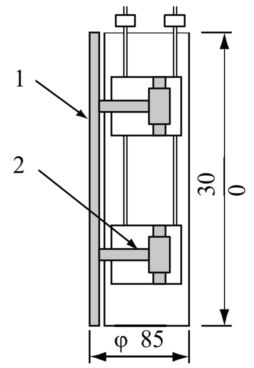
As shown in **Fig. 3.5.3**, the borehole jack test method uses uniform displacement loading in which a load is applied at a right angle to the borehole wall through a rigid loading plate of a measuring tube inserted into the test hole.



1: Measuring tube
2: Pressure and displacement control and measuring device
3: Pressure source
4: Hose

Example of a system configuration for the borehole jack test

Unit: mm



1: Loading plate (metal)
2: Piston jack

Example of a borehole jack

Fig. 3.5.3 Borehole Jack Test Equipment[2)]

The types of equipment for load tests inside boreholes mainly used in ground surveys are pressiometers, LLTs and KKTs. The characteristics of each type are compared in **Table 3.5.5**.

Table 3.5.5 Comparison of the Equipment for Load Tests inside Boreholes[2)]

Item	Pressiometer	LLT	KKT
Pressure generation mechanism	Pressure of nitrogen gas in a compressed cylinder	Same as on the left	Hydraulic pressure through a hand pump
Pressure control and measuring mechanism	Automatic regulator	Valve operation	Hand pump operation
Borehole wall displacement measuring mechanism	Calculation from injected water volume	Same as on the left	Calculation from oil discharge volume
Borehole wall pressurizing mechanism	Single or triple chamber type rubber tube	Single chamber type rubber tube	Semicircular metal plate equivalent to a single chamber
Dimensions of pressurizing section (standard)	ϕ56 mm, length 500 mm	ϕ80 mm, length 600 mm	ϕ85 mm, length 300 mm
Rigidity of loading section	Flexible rubber tube	Same as on the left	Steel loading plate
Pressurizing method	Uniformly distributed loading method	Same as on the left	Uniform displacement loading method

(2) In-situ shear friction test

The in-situ shear friction test is conducted inside the borehole to obtain the in-situ shear strength and the deformation coefficient of the ground. **Fig. 3.5.4** shows the outline of the in-situ shear friction test.

The test is conducted in the following order: ① drilling of the borehole to the measuring target depth through self-boring using a drilling bit attached to the tip of a pressurizing probe (measuring tube); ② uniform application of a stressat a right angle to the borehole wall with a pressurizing probe firmly placed against the borehole wall when the borehole depth reaches the target depth; and ③ application of shear force to the ground by pulling up the pressurizing probe at a constant rate with a jack installed on the ground surface while increasing the stress at a right angle to the borehole wall in stages and measuring the shear force at each stage.

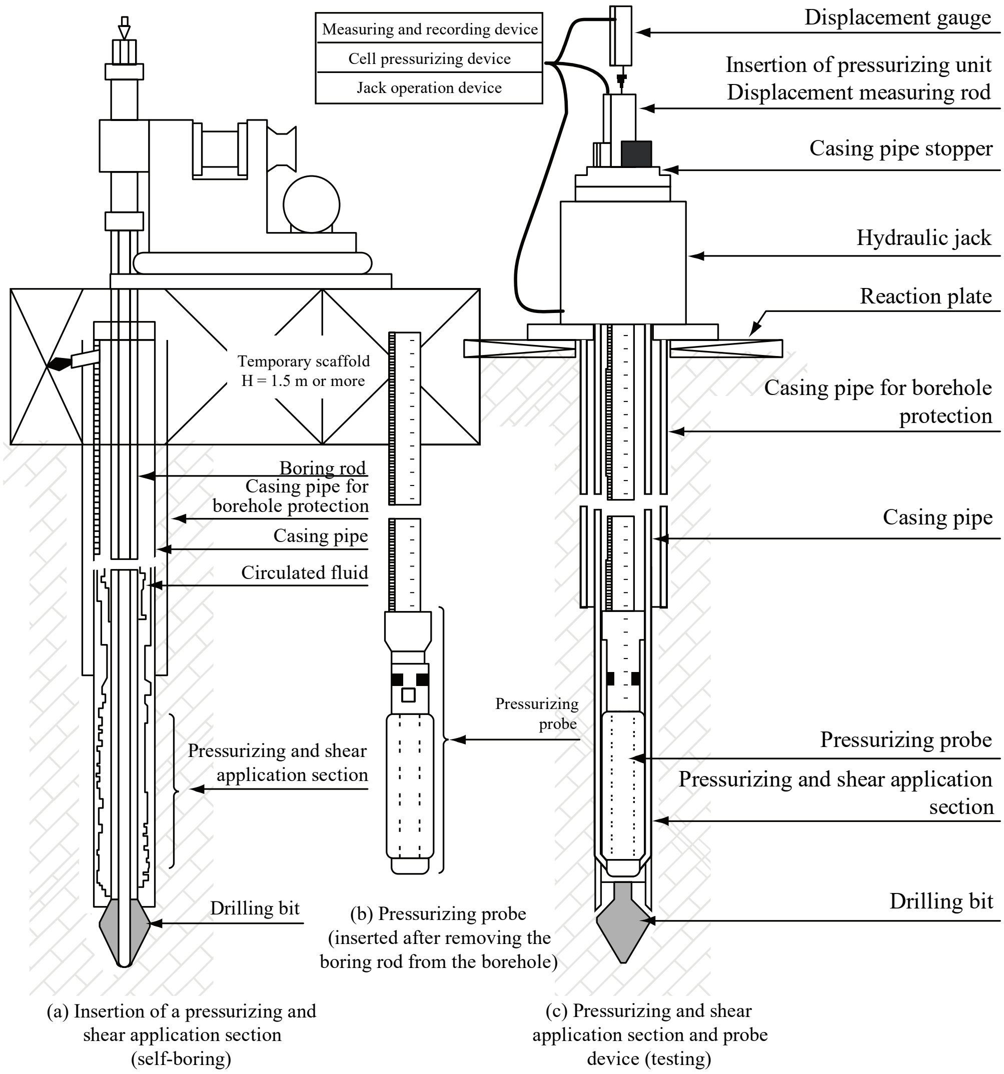


Fig 3.5.4 Schematic Diagram of the In-Situ Shear Friction Test (SBIFT)[2)]

3.5.3 Logging in boreholes (physical logging)

Logging in boreholes (physical logging) is a collective term of the geophysical technology which measures several types of physical quantities in boreholes drilled in the ground to clarify the distribution of the physical quantities in a depth direction, and, therefore, is classified as an in-situ test that uses boreholes. **Table 3.5.6** shows the general types of physical logging.

Table 3.5.6 Main Purposes of Various Types of Physical Logging

Type	Physical quantity	Main purpose	Comment
Velocity logging	Elastic wave velocity	Physical properties, vibration behavior and classification of ground, embedded length of piles and steel sheet piles	Wave source is on the ground surface or inside the borehole. There are cases where only P-waves are measured (when investigating embedded lengths, classifications of rock ground, etc.).
Electric logging (normal method)	Specific resistance	Stratified structure, detection of the positions of aquifers and springs, determination of weak layers, evaluation of ground improvement effects, verification of physical exploration results	Micro logging or spontaneous-potential well logging is also recommended. Measurement is made through a bare borehole or perforated PVC pipe in the borehole.

Type	Physical quantity	Main purpose	Comment
Density logging (γ-γ logging)	Density	Evaluation of compaction degree and porosity of ground	Field density measurement using RI Compliance to laws and regulations on radiation
Borehole temperature logging	Ground temperature Differential temperature	Determination of the positions of aquifers and springs	Measurement is available through a protection pipe. Logging with a focus on differential temperature.
Sonic logging	Acoustic wave velocity	Evaluation of rock ground, detection of the positions of cracks in rock ground	Transmission and reception of waves inside the borehole. Applicable to boring with depths of 100 m or more.
Borehole diameter logging (caliper logging)	Borehole diameter	Detection of cracks in rock ground, correction of the physical data of other logging	Applicable only to bare portions of boreholes. Measurement of diameters in two directions.

The measuring methods of physical logging are rarely affected by the types of object ground. The methods for velocity and electric logging have been standardized by the Japanese Geotechnical Society; i.e., **Method for Seismic Velocity Logging (JGS 1122)** and **Method for Electric Logging (JGS 1121)**.

In the planning stages for physical logging, it is necessary to deliberate measures against the possibility that the logging results may be largely affected by the presence or absence and the materials of protection pipes (casing pipes) as well as the water quality in the boreholes. For example, because electric logging cannot be conducted in steel protection pipes, it is necessary to use perforated PVC pipes. In addition, boreholes need to be cleaned with drilling fluid replaced with clean water, etc..

(1) Velocity logging (PS logging)

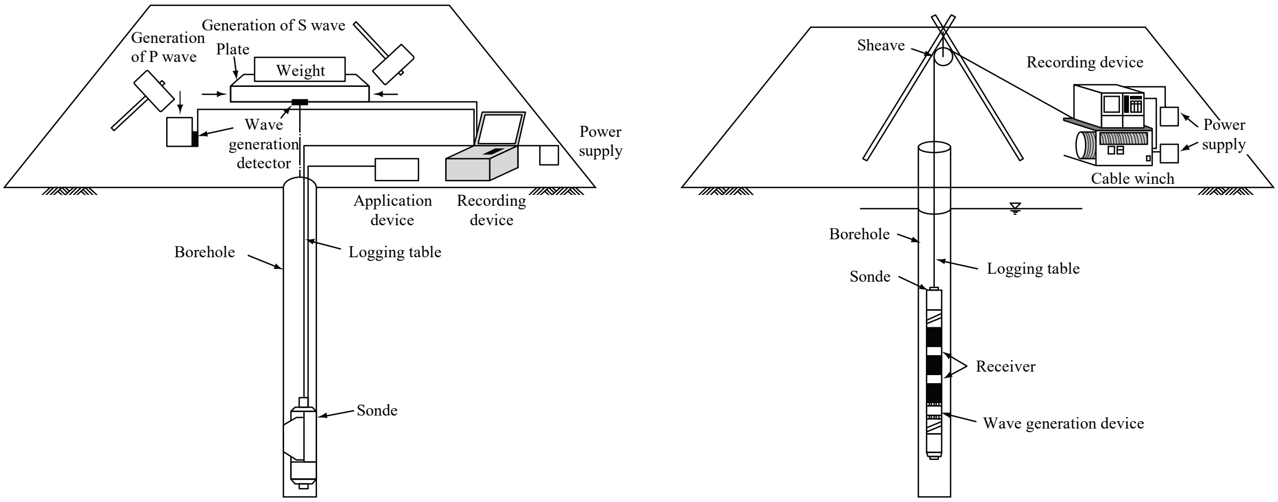
Velocity logging was standardized as **Method for Seismic Velocity Logging (JGS 1122)** in 1995 by the Japanese Geotechnical Society. The standard underwent two revisions: the 2004 revision was to incorporate the concept of allowable errors into the measurement intervals and to modify terms, etc., and the 2011 revision was to introduce the classification of velocity logging into a downhole system and an intra-borehole transmission and reception system, etc. Velocity logging has been used as part of ground surveys to examine earthquake disaster countermeasures such as earthquake resistant designs and predictions of ground motions, and for surveys in the civil engineering field to quantitatively evaluate the hardness of ground and cracks for designing dams, tunnels, etc.

The velocities of elastic waves propagating in ground have a close relationship with the strength and vibration characteristics of the ground, and, therefore, are important physical property values in ground surveys. Velocity logging is a type of physical logging used to measure the distribution of elastic wave velocities in a depth direction using boreholes. The elastic wave velocities comprise P waves (longitudinal and compressional waves) and S waves (lateral and shear waves), and the type of logging that measures both P and S waves is called “PS logging.” The P and S waves measured through PS logging make it possible to set engineering foundations, determine the types of ground, and obtain Poisson ratios, shear moduli of elasticity and young moduli at the level of small strains. Thus, PS logging is an effective survey method for modeling ground in earthquake response analyses, etc. and examinations of earthquake resistance.

In PS logging, the propagation velocities of P and S waves are measured in a manner that first generates P and S waves by applying compression and shear forces to the ground, respectively, then receives the P and S waves propagated directly through the ground with receivers arranged at equal intervals in the borehole, and obtains the velocities between the wave sources and receivers. Because elastic wave velocities vary depending on the degrees of hardness or softness, consolidation (geological ages), density, porosity, weathering and transmutation of ground, they are effective factors to quantitatively evaluate the mechanical properties of the ground. Two popular types of velocity logging which use one borehole and can be applied to any type of ground from soft to rock ground are classified into the downhole system and the intra-borehole transmission and reception system (suspension system). **Table 3.5.7** and **Fig. 3.5.5** summarize the characteristics and configurations of these systems, etc.

Table 3.5.7 Types and Main Wave Transmission and Reception Systems in Velocity Logging

Specification of measuring equipment \ System name		Downhole system	Intra-borehole transmission and reception system (suspension system)
Wave transmission position		Ground surface	Inside the borehole
Wave reception position		Inside the borehole	Inside the borehole
Type of wave source	P wave source	Impact by a hammer, weight, explosive, air gun, etc.	Electromagnetic hammer, sparker, piezoelectric wave generator, etc.
	S wave source	Manual or mechanical hitting of the plate, etc.	Electromagnetic hammer, piezoelectric wave generator, etc.
Measuring method		Waves are measured with wave sources arranged on a ground surface close to the borehole mouth and receivers arranged at arbitrary depths inside the borehole.	Waves are measured with a sonde with wave transmitting and receiving units integrated into it at a predetermined depth in the borehole.



Example of logging equipment for downhole system　　Example of logging device with intra-borehole transmission and reception system

Fig. 3.5.5 Configuration of Velocity Logging Equipment[2)]

The velocity logging systems shall be selected with due consideration of the following four points.

① States of the ground surface (availability of wave generation around the borehole mouth) and the inside of the borehole (groundwater level and borehole wall conditions)

② Availability of the intra-borehole transmission and reception system only when water is inside the borehole

③ Availability of a place to install wave generation devices and the effects of noise and vibration associated with wave generation in the case of the downhole system

④ Unsuitability of the downhole system for offshore or water boring

The standard measuring intervals are 1 ±0.1 m for soil ground and 2 ±0.2 m for rock ground. In the case of the downhole system, larger wave generation energy is required with an increase in measuring depths, and, depending on the conditions of the object ground and the surrounding environment, large wave generation devices enable waves to be measured at a depth of up to 100 m. In the case of the suspension system, which generates and receives waves inside the borehole, there is no depth limitation.

Velocity logging has been rapidly spread as a means to obtain important information when examining the problem with earthquake resistance of soil ground in particular, in the cases introduced below, leading to the further technological advancement of velocity logging. Furthermore, for use in the earthquake resistance design of important structures, velocity logging has become available for measurements at a depth of 100 m, even in rock ground.

① Because of the characteristics of P waves, which vary velocities depending on the degrees of hardness or softness, cracks, weathering, fracturing and moisture, the gas content of the ground, etc, velocity logging has been used for identifying classifications of rock ground at the planning sites of tunnel, earth cut or dam construction, etc, and for obtaining the positions and sizes of faults and fractured zones as well as the thicknesses of weathered zones.

② Because of the characteristics of S waves, which vary velocities depending on the shear rigidity of the ground, velocity logging has been used for evaluating the mechanical properties of rock and soil ground and for earthquake resistance design such as earthquake response analyses.

③ The important parameters (dynamic moduli of elasticity) of ground such as Poisson ratios, shear moduli of elasticity (rigidity) and young moduli at the level of small strains can be obtained from the velocities of P and S waves based on wave theory. Particularly, the shear moduli of elasticity (rigidity) are important input ground moduli in earthquake response analyses.

④ Because of the availability of detailed velocity distributions in the depth direction at boring points, velocity logging is an effective tool for modeling ground. The results of velocity logging have also been used in the prediction of behavior in response to earthquakes or artificially generated ground motions and the classification of ground, etc.

The points of caution when using velocity logging are as follows.

① The use of a borehole enables the velocities of each layer to be measured, even in ground with complex geological structures. However, in cases of the existence of high velocity zones parallel to the neighboring borehole or obliquely crossing the layers at gentle angles close to the borehole, the logging results may be affected by them and show velocity layers conflicting with the boring histograms.

② In the case of a downhole system with a casing pipe inserted into the borehole, the effect of the casing pipe on the logging results can be reduced by keeping a wave source on the ground surface away from the borehole mouth. Because the intra-borehole transmission and reception system (suspension system) cannot be applied to the distance of a borehole with a casing pipe inserted or the distance without water, the downhole system needs to be used for these distances.

③ There are two types of methods for measuring the shear moduli of rigidity and damping constants in the region of small shear strains: one is in-situ elastic wave exploration, such as velocity logging, and the other is laboratory dynamic deformation tests using less disturbed specimens. Velocity logging is applicable only to the measurement of the shear moduli of rigidity corresponding to a shear strain amplitude of about 10^{-6} and is not applicable to the measurement of the shear moduli of rigidity and damping constants in the region of large shear strain amplitude. However, velocity logging is advantageous in that it can measure values directly through the original ground. The measurement of the shear moduli of rigidity and damping constants in the range of medium to large strains is available through dynamic deformation tests using additionally collected specimens, or through estimations using the existing empirical equation[3)] based on the plasticity index, porosity, unconfined compressive strength and *SPT-N values*.

④ When reading waveforms of S waves, it is necessary to superpose S waves in different impact directions and to determine the rising points in superposed waveforms through the identification of points where the phases are inverted.

⑤ Although velocity logging is one of the in-situ tests that is advantageous for avoiding the effects of the disturbance of soil on the test results, it may require correction of measured S wave velocities when effective surcharge pressure is subjected to changes due to structures, landfill, etc.

⑥ In the case of velocity logging for soft seabeds, there are many caution items in measurement such as the methods for transmitting and receiving elastic waves (longitudinal and lateral waves), the accuracy of reading waveforms and borehole wall protection methods.

⑦ In velocity logging, the shear modulus of rigidity G_0 of ground can be determined based on the measured propagation velocities V_s of S waves. In general, the shear modulus of rigidity is a parameter dependent on confined pressure as is the case in the dynamic analysis program, FLIP, where the shear modulus of rigidity G_{ma} is a parameter under the standard effective confined pressure σ_{ma}' (effective confined pressure when the shear modulus of elasticity is measured). Furthermore, in FLIP, the volume elastic modulus of the structural frames of soil particles K_{ma} (under the standard effective confined pressure σ_{ma}') is also a parameter to be set based on the Poisson ratio (generally about 0.33, as obtained through laboratory tests). These extremely important physical parameters can be obtained through PS logging. Although there are several PS logging methods, the method using the equipment of the intra-borehole transmission and reception system (suspension system) is preferable from the viewpoint of securing measurement accuracy, etc. For the details for setting parameters, refer to **Reference [Part III], Chapter 1, 2 Basic Items for Earthquake Response Analyses**.

(2) Electric logging

The logging technologies to measure the electric resistance (specific resistance) and spontaneous potential of ground using boreholes are collectively called electric logging. There are many types of electric logging depending on the electrode arrangement patterns, measurement principles, etc.. Electric logging has been widely used for classifying and comparing layers as well as determining aquifers. Because the specific resistance of layers largely varies depending on the electric characteristics of groundwater, degrees of saturation, porosity, etc, electric logging has been used for groundwater surveys specifically for the estimation of aquifers and the examination of springs and water leaks, etc. In addition, because the specific resistance is high in sandy soil and low in cohesive soil and lower in highly weathered rock ground with an abundance of cracks than less weathered rock ground, electric logging has been used for the estimation of rock stratigraphy and lithofacies changes in ground surveys for tunnel, bridge construction, etc. The Japanese Geotechnical Society standardized the methods for normal logging, micro logging and spontaneous potential logging, which have been frequently used in ground surveys, collectively as the **Method for Electric Logging (JGS 1121)** in 1995, and then revised it in 2004 to correct the deffinitions of terms and incorporate the concept of allowable errors into the measurement intervals.

The scope of application of electric logging is in all types of ground, from soft to rock ground, below groundwater levels in consideration of the necessity of applying electric currents through water inside the boreholes to measure potential. The scope of application may include the ground above groundwater levels in the case of an availability of water inside the boreholes.

Electric logging is used to measure specific resistance, which is electric resistance per unit length and unit cross-sectional areas of the layers, measured inside the boreholes. The measured specific resistance is apparent, representing the average specific resistance of the ground surrounding the electrodes inboreholes subjected to the effects of water inside and their diameters.

Variations of electric logging include normal logging (normal method) and micro logging. An example of an arrangement of electrodes in electric logging is shown in **Fig. 3.5.6**. Spontaneous potential logging (SP logging) is another variation of electric logging and measures the potential electrochemically generated inside the boreholes. Examples of an arrangement of electrodes in spontaneous potential logging and electric logging equipment are shown in **Figs. 3.5.7** and **3.5.8**, respectively.

① **Normal logging**

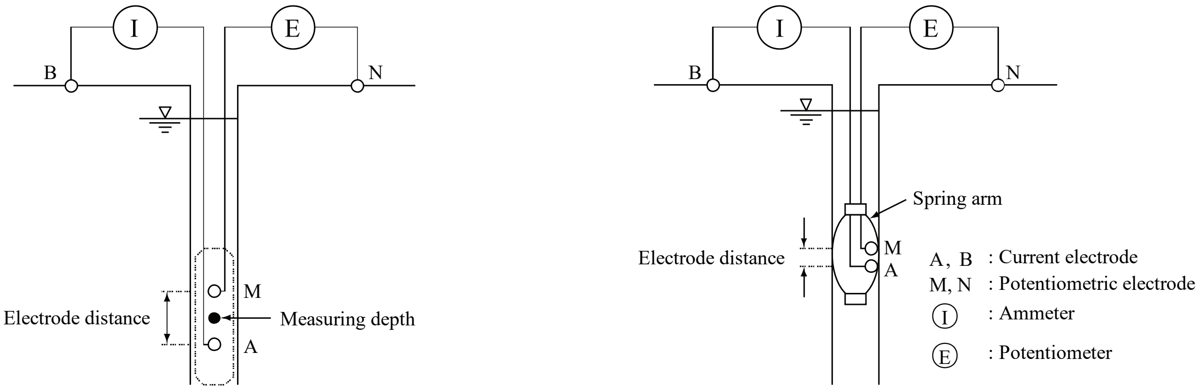
In normal logging, a set of an electric current electrode A and a potentiometric electrode M is arranged inside a borehole, and another set of an electric current electrode B and a potentiometric electrode N is fixed on the ground surface. The distances between electrodes A and M are called “electrode distances” and are set at 0.25 ±0.03 m, 0.5 ±0.05 m, and 1 ±0.05 m as standards.

② **Micro logging**

Micro logging has an electrode distance inside the borehole shorter than normal logging with the electrodes firmly attached to the borehole wall. The electrode distances are set at 25 ±5 mm and 50 ±5 mm as standards. Micro logging is used for detecting thin layers, accurately understanding layer boundaries and determining permeable layers, etc.

③ **Spontaneous potential logging**

Spontaneous potential logging measures only with potentiometric electrodes without current electrodes. One of the potentiometric electrodes, M, is arranged inside the borehole and another potentiometric electrode, N, is on the ground surface. Spontaneous potential logging is used for supplementing normal logging.



(a) Normal logging (normal method)

(b) Micro logging

Fig. 3.5.6 Method for the Arrangement of Electrodes in Electric Logging[2)]

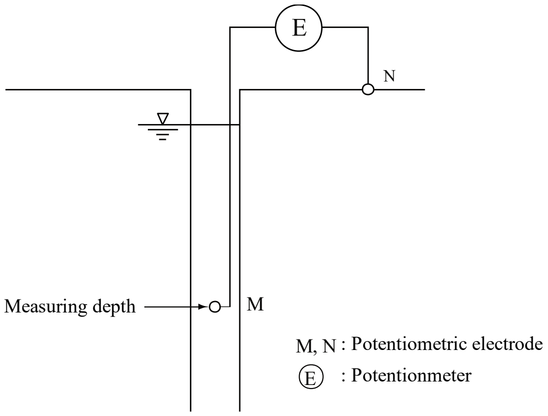


Fig. 3.5.7 Method for the Arrangement of Electrodes in Spontaneous Potential Logging[2)]

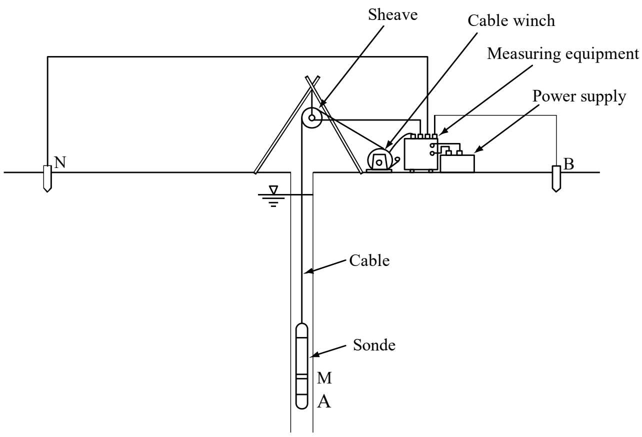


Fig. 3.5.8 Example of Electric Logging Equipment[2)]

The depth to which electric logging is applicable is subjected to the specifications of the test equipment, such as the cable length and the temperature and pressure resistance of the sonde.

The following items become available through electric logging.

① Apparent specific resistance curves (for three types with different electrode distances) and a spontaneous potential curve of the layers around the borehole wall.

② The estimated positions of aquifers, springs and water leaks, the confirmation and comparison of stratification, and the confirmation of the existence and positions of fractures and highly weathered zones through qualitative interpretation of apparent specific resistance curves in the depth direction and a spontaneous potential curve.

③ Comparative analyses with two-dimensional specific resistance exploration results and the specific resistance of ground to be used for grounding.

The specific resistance curves and the spontaneous potential curve obtained as a result of electric logging are continuous measurement records in a depth direction. The specific resistance curves corresponding to the respective electrode distances and the spontaneous potential curve are juxtaposed to the boring histograms or other test results on identical drawings so as to enable layers, soil and rock properties obtained through different surveys to be compared.

The specific resistance is the electric resistance per unit length and the unit cross section, expressed in units of $\Omega \cdot m$, and an inverse of electric conductivity σ (S/m) used as one of the physical quantities to evaluate groundwater. The specific resistance curves obtainable through electric logging show changes in apparent specific resistance and there may be cases where the values of specific resistance vary depending on the electrode distances even at identical measuring depths.

In general ground surveys, the evaluation of layers and the interpretation of survey results have been made by regarding the apparent specific resistance as the specific resistance of layers.

The electric logging results are used for the detection of the distribution of layer thicknesses, seams and aquifers; the determination of impermeable layers; the estimation of layers in distances lacking boring core data; the determination of weak layers such as crack zones and zones subjected to argillation; the comparison of layers and lithofacies among multiple boreholes; and the determination of the continuity of layers, and, as a special case, the ground improvement effects of chemical grouting, etc. Furthermore, there are cases where electric logging results are used for verifying the results of other physical explorations and setting the initial conditions or restrained conditions for analyses of electrical, specific resistance tomography and electromagnetic explorations.

It shall be noted that electric logging which uses electric currents cannot measure normal data when the measuring conditions inside the borehole correspond to any of the following.

① Portions of the borehole which are shallower than the groundwater level and are not filled with water

② Boreholes inserted with PVC pipes (excluding perforated PVC pipes) or steel casing pipes

③ Boreholes drilled with drilling fluid containing polymer mud agents

④ Boreholes in areas subjected to electric noise (stray currents) generated from substations, power stations, high voltage lines, factories, etc

⑤ Boreholes in coastal areas with groundwater mixed with seawater (salinity is likely to cause measured specific resistance values to be lower than the actual values)

(3) Density logging

Density logging is a method for in-situ measurement of the density distribution of ground continuously in a depth direction by using boreholes and uses gamma rays emitted from radioactive isotopes as a radiation source. Density logging continuously measures the density of ground with a sonde mounted with a radiation source and a detector while moving the sonde inside the borehole, thereby enabling the density distribution in the depth direction of ground including rock ground to be obtained. A schematic drawing of density logging equipment is shown in **Fig. 3.5.9**.

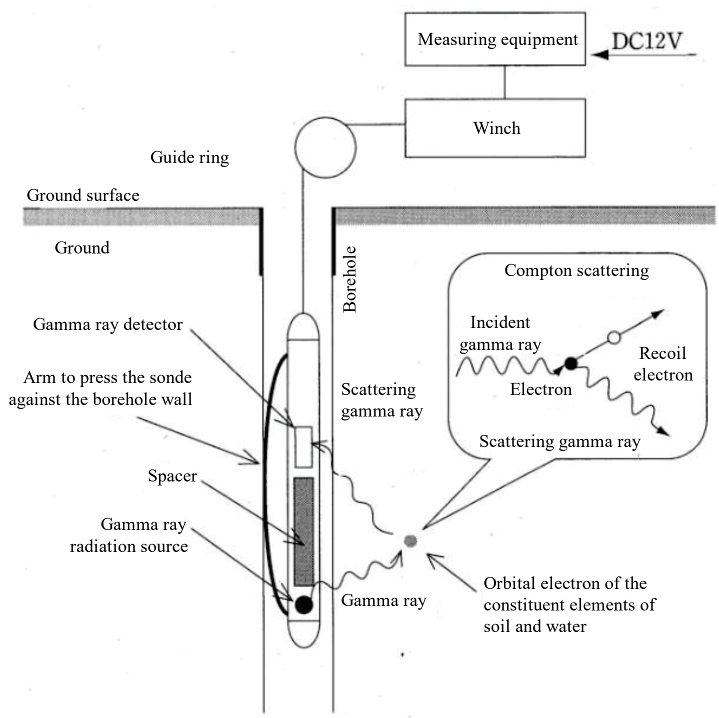


Fig. 3.5.9 Schematic Drawing of Density Logging[2)]

Density logging uses a sealed radiation source of cobalt 60 (60 Co) or cesium 137 (137 Cs) and a scintillation detector as a sensor in general. The radiation sources are mostly small (no more than 3.7 MBq) without notification obligations; for example, 60 Co has a half-life period of 5.24 years and energy of 1.17 MeV and 1.33 MeV, and 137 Cs has a half-life period of 26.6 years and energy of 0.662 MeV. The depth to which density logging is applicable is subjected to the specifications of the test equipment such as the cable length and the temperature and pressure resistance of the sonde. In the case of density logging, however, the applicable range in horizontal directions is more important than the range in a depth direction, and the applicable horizontal ranges are generally about 20 to 30 cm. Density logging shall be conducted after the confirmation of the conditions inside the boreholes so as to prevent accidents with radiation sources sticking to the borehole walls, and natural radioactivity logging with no radiation sources mounted on the sonde so as to obtain background equivalent activities, which differ ground by ground, as correction data. Then, density measurement is conducted in a manner that lowers the sonde with the mounted radiation source into a boerhole and measures density while pulling the sonde upward at a sufficiently slow and constant rate appropriate to the energy of the radiation source. In density logging, there are two types of measuring systems: a dose rate meter system and a counter system. In the case of the dose rate meter system, measurement is conducted continuously with a sonde slowly pulled upward from the bottom of the borehole with a winch (at a rate of about 2 to 3 m a minute) while recording count rate data on a pen recorder or a digital recorder. In the case of the counter system, measurement is conducted in a manner that lowers a sonde and keeps it stationary at a predetermined depth for about 30 to 60 seconds while measuring the count rate and repeats the cycle at 0.5 to 1.0 m intervals to the ground surface. The counter system requires a longer measuring time than the dose rate meter system but can enhance the accuracy of the count rates as the measuring time becomes longer.

Density logging can measure the distribution of the in-situ density of the ground around the borehole wall in a depth direction and obtain data on density in in-situ moisture states, etc. While the data obtainable through laboratory soil tests and field density tests using specimens are discrete, the data obtainable through density logging are continuous in the depth direction, and, therefore, capable of interpolating the results of laboratory soil tests or field density tests using specimens and measuring the density of gravel layers and thin layers from which it is difficult to collect undisturbed specimens.

The results of density logging have been used as an important index to classify the layers and density values to calculate elastic moduli of ground together with the results of the velocity logging. As is the case with density logging, the types of physical logging using radioactive isotopes include neutron logging which measures moisture content using neutron ray sources, and, more specifically, the distribution of moisture content of ground including rock ground in a depth direction. The density and moisture content obtained through density logging and neutron logging, respectively, enable physical properties of layers such as dry density, degree of saturation and porosity to be obtained.

The points of caution when using density logging are as follows.

① In density logging, the accuracy of measurements depends on the accuracy of the calibration curve which associates density with the number of counts (count rates) and is to be prepared before conducting measurements. The intensity of gamma rays to be measured in density logging is expressed by the number of counts per unit time and the calibration curve shall be prepared through the measurements in accordance with **the standards of the Japanese Geotechnical Society (JGS 1614)**.

② The accuracy of measurements is also affected largely by the conditions inside the borehole such as the borehole diameter. In many cases, favorable data cannot be obtained through density logging with borehole diameters of ϕ100 to 120 mm or more, large gaps behind the casings, or double casings.

③ There may be cases where the object ground or bedrock has a high natural radioactivity and the density logging needs to be conducted in a manner that eliminates the effects of the natural radioactivity on the measuring results when necessary.

④ It is preferable to conduct calibrations in preparation for density logging in order to confirm the attenuation conditions of the radiation source and deterioration of the detector, etc. It is also important to renew the radiation source approximately once every half-life period.

⑤ Radioactive isotopes shall be handled with extreme caution in compliance with the related laws and regulations including the Act on Prevention of Radiation Hazards due to Radioisotopes, etc.

3.5.4 Pore Water Pressure Measurement

Pore water pressure measurement is conducted for the purposes of confirming the consolidation degree of cohesive ground and obtaining data for stability analyses of slopes associated with earth cutting or filling, stability analyses (examinations of boiling and heaving) of bases associated with excavation work, and the calculation of effective stresses. Pore water pressure measurement is classified into short term measurement, which obtains the pore water pressure at the time of ground surveys to utilize the measurement results directly in design and construction, and long term measurement, which monitors the changes in pore water pressure associated with construction work, etc. on a long-term bais to utilize monitoring results in construction management, etc.

Pore water pressure measurement is conducted either in a condition where the borehole is kept open with a casing, etc. installed inside when testing sandy soil and gravel, or in a condition where the borehole is closed with an electric power water pressure meter installed inside, etc. when testing cohesive soil (as shown in **Fig. 3.5.10**).

There has been a proposal of a new method capable of measuring pore water pressure at multiple depths using a single borehole. The procedure for the new method is shown in **Fig. 3.5.11**. This method enables the pore water pressure from multiple layers to be measured with a single borehole.

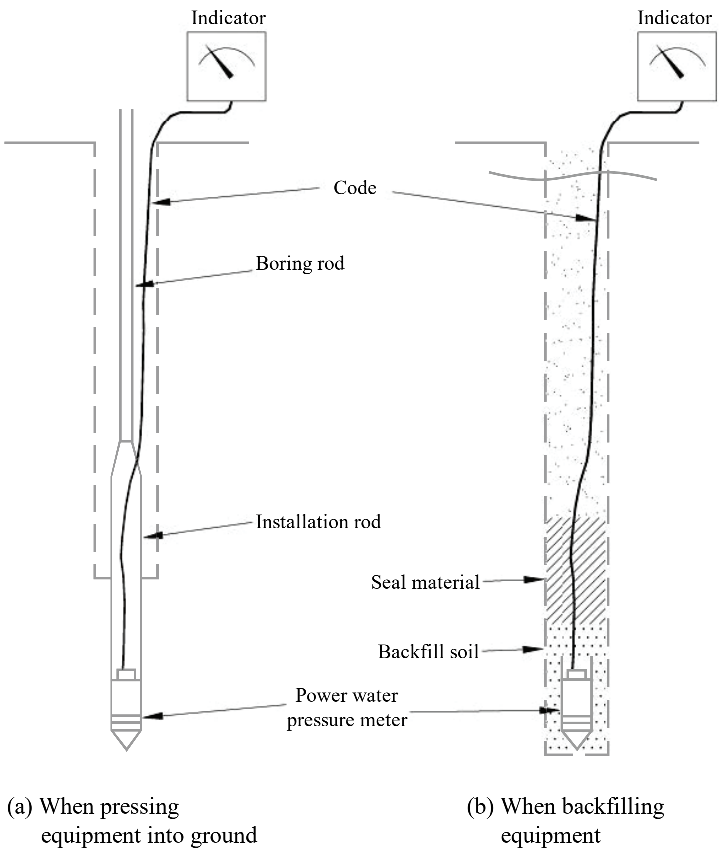


Fig. 3.5.10 Pore Water Pressure Measurement2)

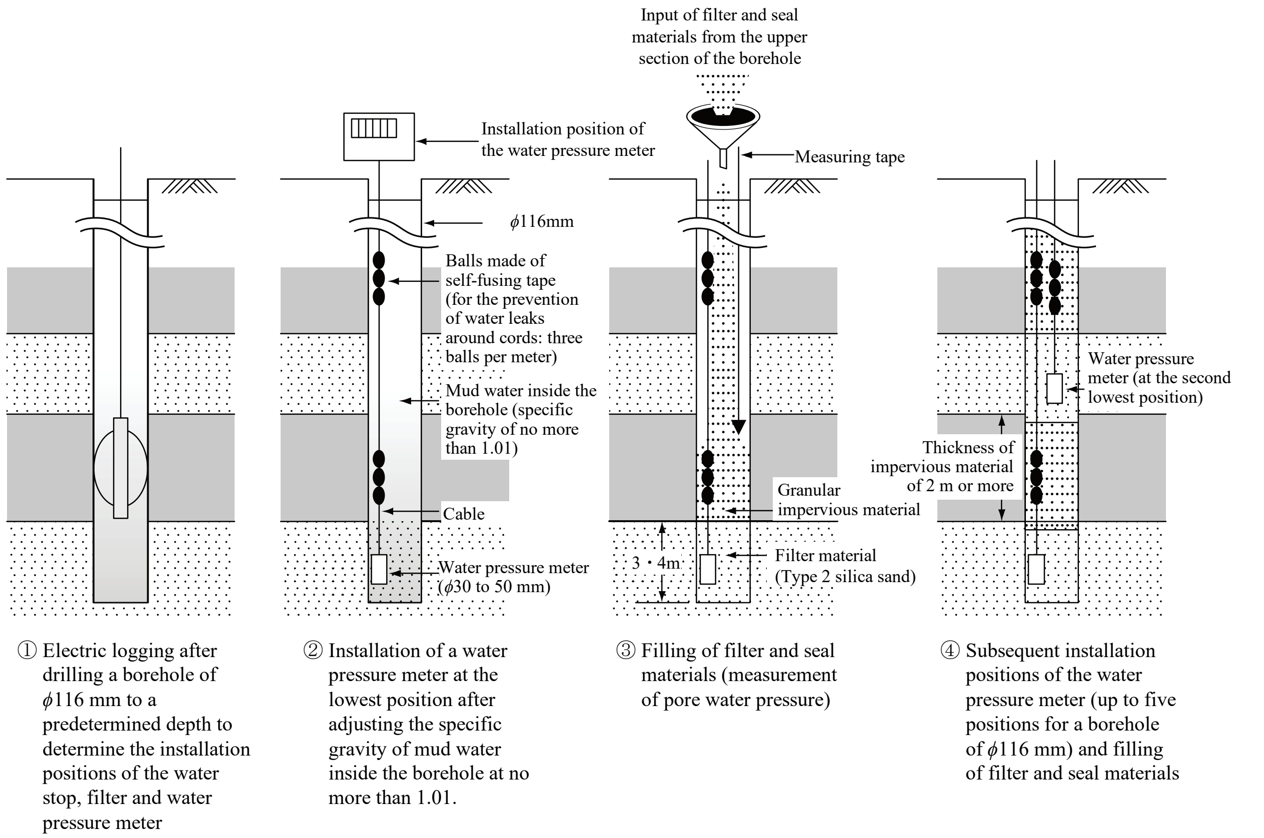


Fig. 3.5.11 Procedure of Constructing Hole for Measuring Pore Water Pressure at Multiple Depths[2)]

3.6 Sampling

Sampling in ground surveys is implemented for the purpose of collecting specimens for soil observation and laboratory tests to obtain information on ground necessary for designing and building structures. The specimens obtained through sampling used to be classified into “undisturbed specimens” and “disturbed specimens” depending on the test purposes and ground conditions. The specimens for laboratory mechanical tests collected through the “method for sampling undisturbed specimens” have been treated as if their quality was already assured. However, the technical advancement of laboratory test methods has improved the sensitivity of test results to the effects of the disturbance of specimens on strength and deformation properties, and has revealed the problems with this increased vulnerability. Thus, considering the necessity to evaluate the quality of the specimens, including “undisturbed specimens,” sampled in accordance with the intended laboratory test methods or test accuracy, the Japanese Geotechnical Society revised its sampling standards in 2003 to change the term “undisturbed” to “less disturbed” for the quality of soil to be sampled and eliminated the term “undisturbed” from the name of the standards.

The “less disturbed” specimens sampled with thin-walled tube samplers with fixed pistons, etc. are used as “undisturbed” specimens for mechanical tests, etc. in laboratory soil tests, and the “disturbed” specimens sampled through standard penetration tests, etc. are used in physical tests as “intentionally disturbed” specimens. In this way, it is necessary to pay attention to the differences in the terms used for specimens between field and laboratory tests.

The quality evaluation of sampled specimens is of extreme importance when examining the reliability of survey results and designs based on the results. Thus, appropriate samplers shall be selected when sampling less disturbed specimens. However, recently there have been an increasing number of cases of analyzing ground contamination using disturbed specimens.

3.6.1 Types and Characteristics of Sampling Methods

There are many sampling methods but they can be largely classified into two types: methods that use sampling tubes and those that do not. Typical sampling methods, the structures of standardized samplers and their applicable ground, and the typical types of unstandardized sampling methods and the specifications of samplers are shown in **Tables 3.6.1**, **3.6.2** and **3.6.3**, respectively.

In **Table 3.6.2**, five sampling methods, excluding the block sampling method (**JGS 1231**), are classified as sampling methods that use sampling tubes and require boreholes through boring. Thus, it is important to select samplers appropriate for the use purposes and the ground from which specimens are collected, and boring machines and methods appropriate for the required diameters and depths for the specimens. When collecting specimens, boring shall be implemented so as not to degrade their quality by disturbing the bottoms or walls of the boreholes with strong impacts or injection water as a result of giving priority to more efficient drilling.

Furthermore, the samplers shown in **Table 3.6.3** are used for the types of ground and special purposes (large diameters, large depths, continuous specimen sampling, etc.) to which standardized samplers cannot be applied. In addition to these samplers, there are SPT samplers used for the standard penetration test as specified in **JIS A 1219**, open tube samplers for environmental chemical analyses as specified in **JGS 1912**, closed piston samplers and double tube screening samplers, etc. The "disturbed" specimens sampled with SPT samplers are applicable to laboratory soil tests such as physical and minimum/maximum density tests, and are not applicable to mechanical tests such as wet density, uniaxial compression and consolidation tests.

Table 3.6.1 Typical Sampling Methods

Type	Sampling method using a sampling tube	Sampling method without using a sampling tube
Sampling method	• Sampling method collecting specimens from the bottoms of boreholes drilled by non-core boring • Sampling method collecting specimens concurrently with borehole drilling by rotary type machine boring (core boring) • Sampling method with a sampler directly pressed into very soft ground filled with cohesive soil, etc.	• Sampling method (block sampling) collecting specimens from blocks cut out of outcrops or pits • Sampling method collecting cylindrical specimens by pulling soil columns from frozen ground or coring frozen ground (frozen sampling) • Sampling method collecting specimens using an auger

Table 3.6.2 Structures of Standardized Samplers and Their Applicable Ground[2)]

Sampling and sampler		Structure	Boring diameter (mm)	Type of ground										
				Cohesive soil			Sandy soil			Gravel		Rock		
				Target *SPT-N value*			Target *SPT-N value*			Target *SPT-N value*		Soft	Medium-hard	Hard
				0 to 4	4 to 8	8 or more	10 or less	10 to 30	30 or more	30 or less	30 or more			
				Soft	Medium	Hard	Loose	Medium	Dense	Loose	Dense			
Thin-walled tube sampler with fixed piston (JGS 1221)	Hydraulic type	Single tube	86	◎	◎	○,◎	○,◎	◎	◎					
	Extension rod type	Same as above	86	◎	○		○							
Rotary double tube sampler (JGS 1222)		Double tube	116		◎	○								
Rotary triple tube sampler (JGS 1223)		Triple tube	116		◎	◎	○	◎	◎		○			
Rotary double–tube sampler with sleeve (JGS 1224)		Double tube	66 to 116	○	○	○	○	○	○	○	○	◎	◎	◎
Block sampling (JGS 1231)		–	–	◎	◎	◎	○	○	◎		○	○		
Rotary tube sampling (JGS 3211)		Multi-tube	66 to 116			○						◎	○	

◎Most appropriate, ○Appropriate

Table 3.6.3 Unstandardized Typical Sampling and Samplers[2)]

Sampling and sampler	Structure	Piston		Pressing		Type of ground	State of specimen		Specimen diameter	Boring diameter (mm)	Remarks
		With	Without	Static	Rotary		Less disturbed	Disturbed			
Frozen sampling						Sand, sand gravel	○		Arbitrary		
Wire line sampler	Single tube Double tube	○		○	○	All types of soil except rock	○		78, 90	135, 146	
Gravel layer sampler	Double tube	○			○	Rudaceous soil	○		97	146	Availability of large diameter sampler
Twist sampler	Double tube	○		○		Sandy soil, very soft soil	○		50, 70	86	
Sampler for very soft soil	Double tube		○	○		Very soft soil		○	50		
Thin-walled sampler with free piston	Single tube	○		○		Cohesive soil		○	75	86	
Composite sampler	Double tube	○		○		Cohesive soil	○		75	116	
Foil sampler	Single tube	○		○		Cohesive soil, loose sandy soil	○		68		
Double tube sampler with fixed piston	Double tube	○		○		Rudaceous soil, waste		○	70	116	
GP sampler	Single tube		○		○	Rudaceous soil	○		100	127	Availability of large diameter sampler

○Applicable

The following section summarizes samplers and frozen samplings which have been frequently used in ground surveys.

(1) Thin-walled sampler with fixed piston

Thin-walled samplers with fixed pistons collect less disturbed specimens for soil tests using a sampling tube statically pressed into loose sandy ground with high contents of soft cohesive soil or fine fractions. Depending on the mechanisms for pressing the sampling tube into the ground, the thin-walled sampler is classified into hydraulic and extension rod types. The extension rod type is applicable to cohesive soil with degrees of hardness and compactness in terms of *SPT-N values* of about 0 to 4 and sandy soil with *SPT-N values* of about 0 to 8. The hydraulic type is applicable to harder cohesive soil and denser sandy soil than those to which the extension rod type is applicable.

① **Hydraulic type sampler**

The hydraulic type sampler consists of a sampling tube pressed into the ground using water pressure with a piston fixed with the sampler head. The structure of the hydraulic type sampler is shown in **Fig. 3.6.1**. The hydraulic type sampler enables the piston to be simply and reliably fixed by having it automatically put in place when the sampler head is fixed through the boring rod. The operation of the hydraulic type sampler is simple and efficient because it does not require the use of an extension rod. The hydraulic type sampler is particularly efficient for sampling specimens at large depths or under severe work environments such as offshore sampling. In addition, there are cases where the hydraulic type sampler can collect specimens from harder cohesive soil or denser sandy soil than the extension rod type sampler, because large pressing force can be obtained when sufficient reaction force and high-performance pump become available. The diameter of the hydraulic type sampler is normally ϕ86 mm.

② **Extension rod type sampler**

The extension rod type sampler consists of a sampling tube pressed into the ground through a boring rod with a piston fixed through a piston extension rod preliminarily assembled on the ground surface. The structure of the extension rod type sampler is shown in **Fig. 3.6.2**, and its diameter is normally ϕ86 mm.

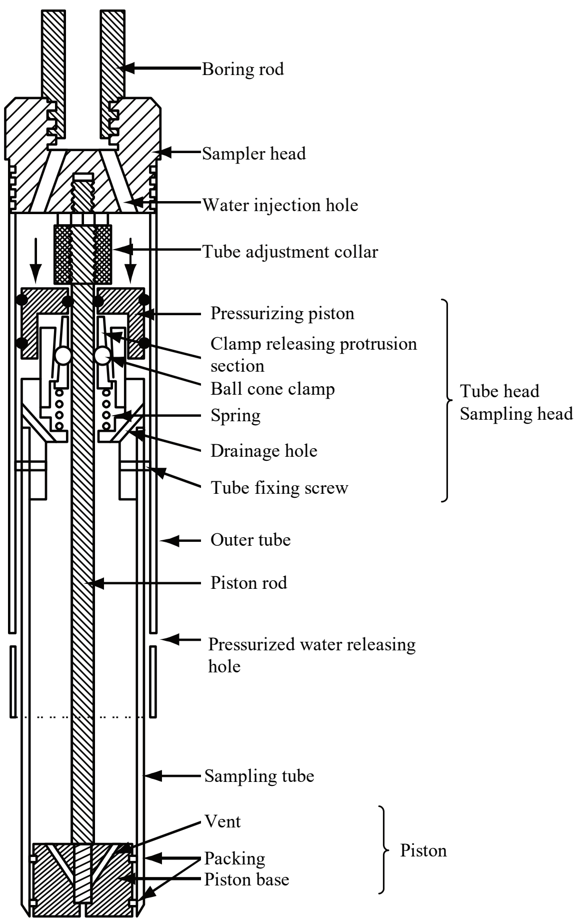


Fig. 3.6.1 Schematic Drawing of a Hydraulic Type Thin-Walled Sampler[2)]

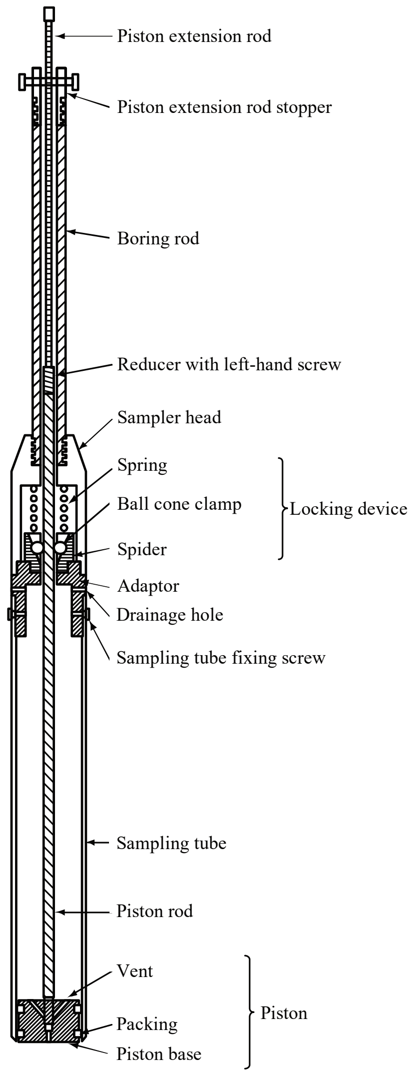


Fig. 3.6.2 Schematic Drawing of an Extension Rod Type Thin-Walled Sampler[2)]

(2) Rotary double tube sampler

The rotary double tube sampler consists of a rotatable outer tube that cuts soil while pressing an inner sampling tube, which is not rotatable, into the ground to collect less disturbed specimens. The structure of the rotary double tube sampler is shown in **Fig. 3.6.3**.

The rotary double tube sampler is used for collecting less disturbed specimens for laboratory mechanical tests and is applicable to medium to hard cohesive soil with a range of hardness in terms of *SPT-N values* from 4 to 14. It is also configured to enable the tip shoe protrusion length of the sampling tube to be automatically adjusted with a spring according to the hardness of the ground. The rotary double tube sampler is also called a Denison sampler. Thin-walled tubes with a diameter of ϕ75 mm are normally used for sampling tubes, and the diameter of the borehole is generally ϕ116 mm.

(3) Rotary triple tube sampler

The rotary triple tube sampler consists of a rotatable outer tube that cuts soil while pressing an inner sampling tube, which is not rotatable, into the ground and samples soil in a liner mounted inside the inner tube. The structure of the rotary triple tube sampler is shown in **Fig. 3.6.4**. The rotary triple tube sampler is also simply called a triple tube sampler and is classified as a type of sand sampler. The rotary triple tube sampler is used for collecting less disturbed specimens for laboratory mechanical tests and is applicable to medium hard and hard cohesive soil and to moderately and densely compacted sandy soil with corresponding *SPT-N values* of 4 or more for cohesive soil and 10 or more for sandy soil. However, the applicability of the rotary triple tube sampler needs to be sufficiently examined because of possible difficulty in collecting specimens due to the falling of specimens, etc. in the case of uniform sandy soil with low fine fraction contents or sandy soil mixed with gravel. The rotary triple tube sampler has a mechanism to enable the protrusion length of the shoe to be automatically adjusted and vibrations associated with the rotation of the outer tube to be attenuated with a steel spring mounted in the sampler head. Because of its mechanism for collecting specimens while cutting soil around the sampler, the rotary triple tube sample is widely applicable to hard cohesive soil and loose to very dense sandy soil, and, therefore, has recently been used in many ground surveys. The diameter of the borehole is normally ϕ116 mm.

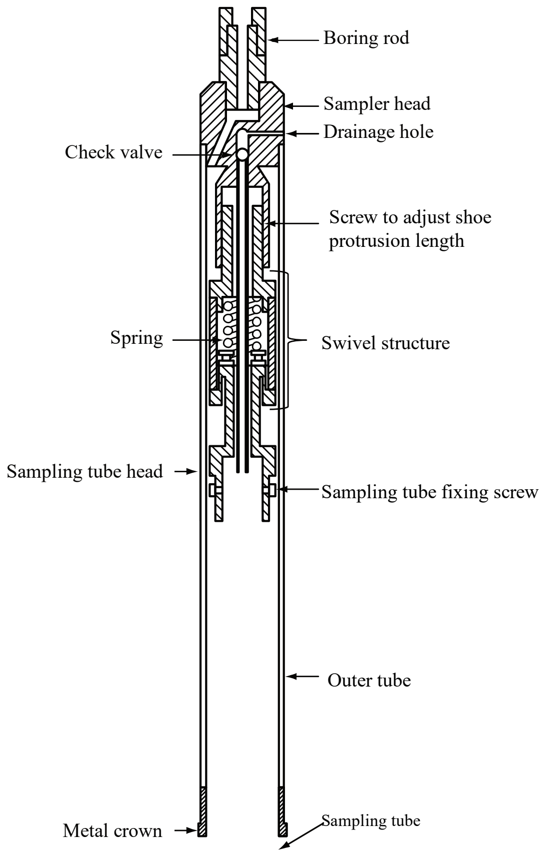


Fig. 3.6.3 Schematic Drawing of a Rotary Double Tube Sampler[2)]

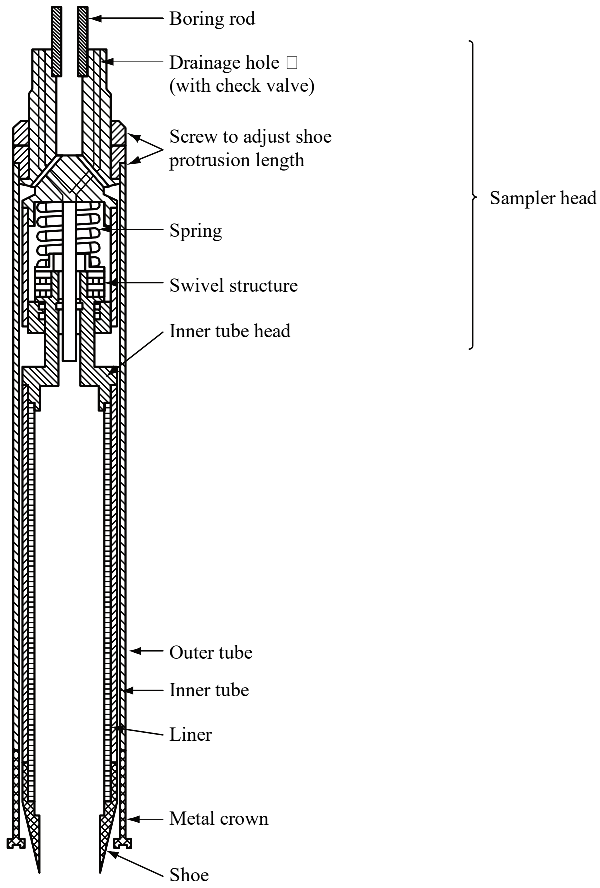
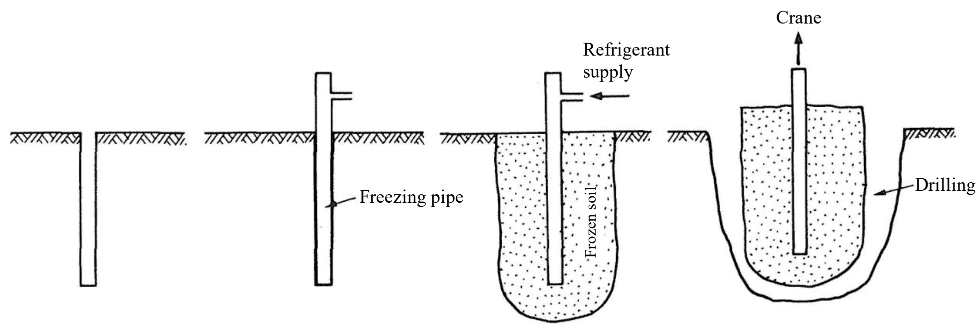


Fig. 3.6.4 Schematic Drawing of a Rotary Triple Tube Sampler[2)]

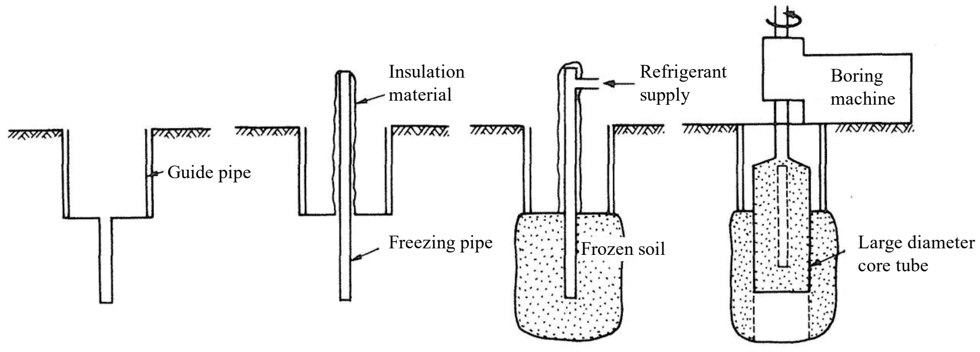
(4) Frozen sampling

Frozen sampling is for collecting less disturbed specimens of sandy soil with low fine fraction contents or rudaceous soil to which standardized samplings are hardly applicable in a state where the ground is frozen in-situ. Frozen sampling is largely classified into three types, as shown in **Fig. 3.6.5**, and in each type, the sampling is conducted by first installing a freezing pipe in the ground, circulating a refrigerant in the pipe and then taking a specimen sample after the ground around the freezing pipe is frozen. The first of the three types of frozen sampling pulls out frozen soil (a frozen soil column) entirely from the ground. This type is easily implemented with a crane or jack and is effective for collecting specimens from shallow ground. The second type, over coring, is used for collecting specimens at large depths in a manner that pulls the specimens with a freezing pipe as its central shaft through a single core tube having a large diameter. The third type, partial core sampling, effectively samples a core with a diameter equal to the test piece for soil tests from a frozen soil column and is used mainly for sampling specimens of rudaceous soil, which is difficult for forming specimens. Frozen sampling has been technically established as a reliable sampling method and is advantageous in that the specimens collected through frozen sampling are less disturbed than those collected through general sampling. However, because it requires extensive facilities, long preparation times and high costs, frozen sampling has been scarcely used for general ground surveys. In addition, when used for soil with high fine fraction contents and low permeability, frozen sampling tends to disturb the specimens.



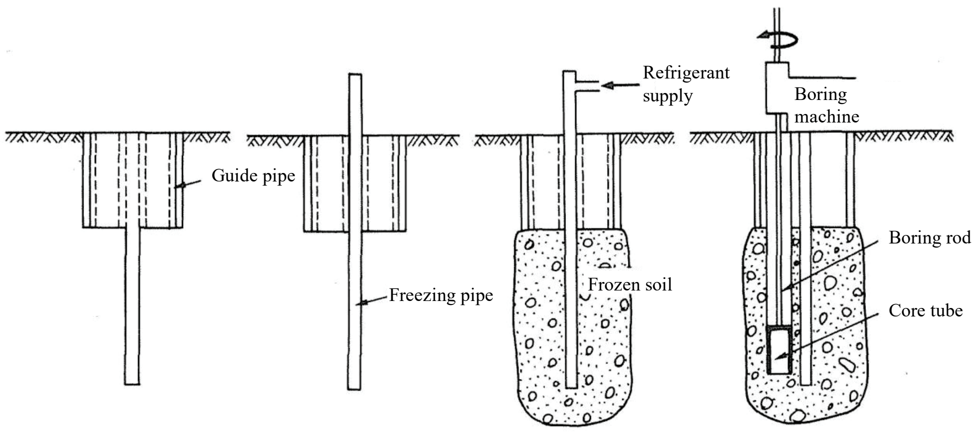
(a) Drilling of a freezing pipe installation hole
(b) Installation of a freezing pipe
(c) Freezing of ground
(d) Pulling out of a frozen soil column

(1) Frozen sampling to pull out frozen soil entirely (frozen soil column)



(a) Drilling of a freezing pipe installation hole and excavation of the upper section of the hole
(b) Installation of a freezing pipe
(c) Freezing of ground
(d) Pulling out of a specimen after over coring

(2) Over coring



(a) Drilling of a freezing pipe installation hole and excavation for guide pipe installation
(b) Installation of a freezing pipe
(c) Freezing of ground
(d) Sampling of a core specimen

(3) Partial core sampling

Fig. 3.6.5 Types of Frozen Sampling

(5) Double tube sampler with fixed piston

The double tube sampler with a fixed piston is a combination of a rotary double sampler and a fixed piston where the piston is fixed through an extension rod on the ground surface (as shown in **Fig. 3.6.6**).

The inner tube to store the specimen is made of an acrylic tube and is prevented from being rotated with the outer tube by the friction between the inner tube and the fixed piston. The double tube sampler with a fixed piston generally uses mud water, foam, compressed air, etc. as drilling fluid and is capable of collecting less disturbed specimens of rudaceous soil, soil in fracture zones, waste, etc.

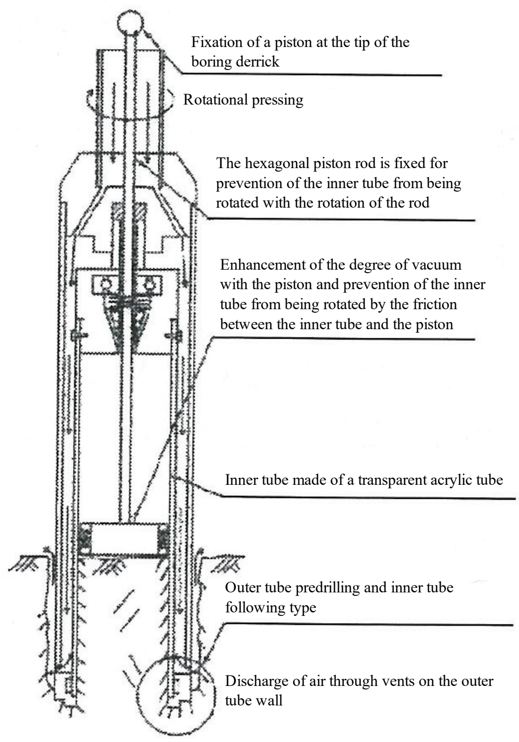


Fig. 3.6.6 Double Tube Sampler with Fixed Piston[2)]

(6) GP sampler

GP samplers are used for collecting specimens from rudaceous soil, soil mixed with coral gravel, etc. GP samplers collect specimens in a manner that preliminarily fills a core barrel with high concentration polymer solution and takes the specimen inside the core barrel while allowing the polymer solution to be extruded through a gap between the specimen and the core barrel, thereby protecting the side face of the specimen. The high concentration polymer solution enables a less disturbed specimen to be collected by reducing friction between the specimen and the sampler, preventing the matrix from flowing out, curbing slaking, etc. (refer to **Fig. 3.6.7**).

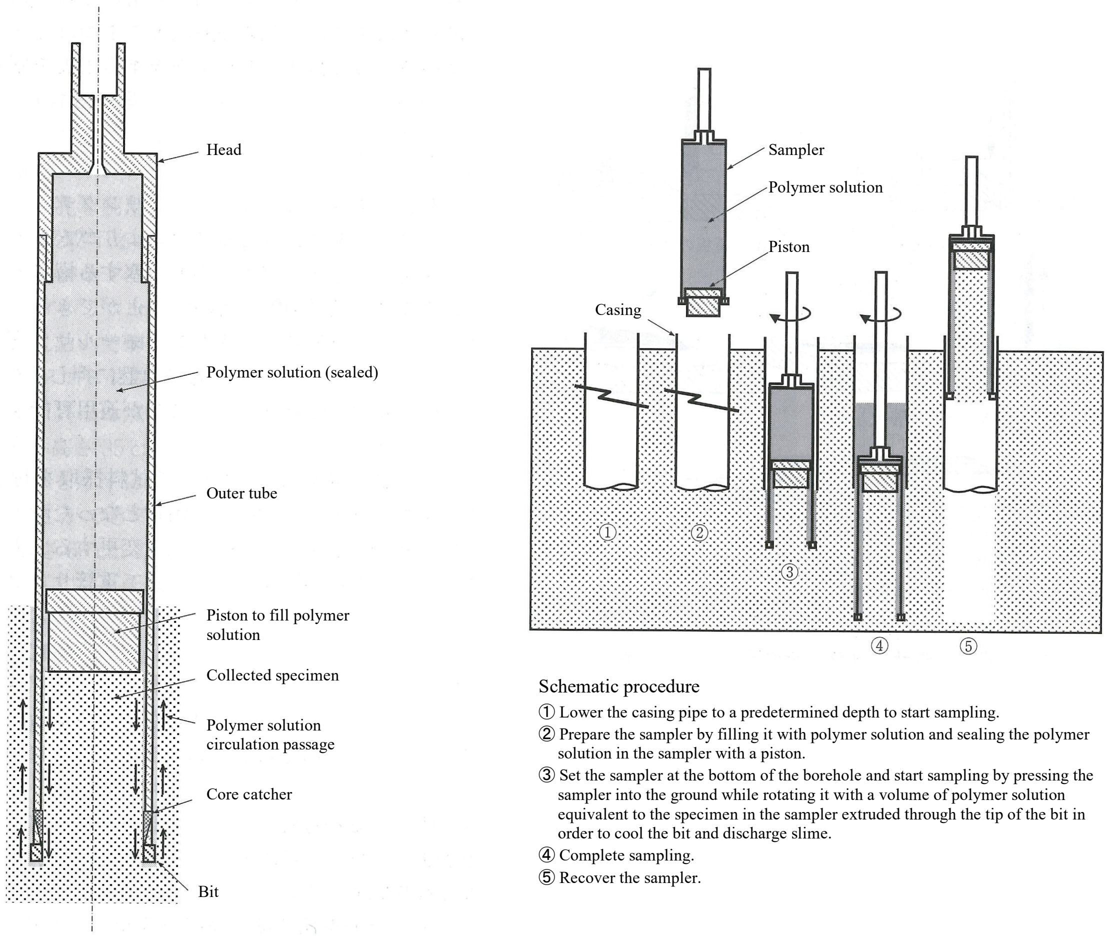


Fig. 3.6.7 Procedure of Collecting Specimens with a GP Sampler[2)]

In addition to the sampling introduced in (5) and (6) above, there is another sampling method which uses aerated mud water as drilling fluid for sampling less disturbed specimens of rudaceous soil, soil mixed with coral gravel, etc. in a manner that circulates mud water at a low pressure and low flow rate while preventing fine fractions from being washed out with air bubbles that expand as the mud water flows upward and effectively utilizes the bubbles to discharge slime.

3.6.2 Purposes of Sampling and Disturbances of Specimens

Ground surveys are generally implemented in stages; for example, a general survey followed by a more detailed one. As the purposes of the ground surveys and required accuracy in these surveys differ depending on the stages, so do the quantities of the samplings, the types and quantities of tests, etc. to be conducted. Thus, it is necessary to implement ground surveys under thorough and careful sampling plans so as to achieve the purposes set for each stage. There are three important items in sampling plans: ① the intervals and quantities of boring, ② the sampling depths and number of specimens, and ③ the types of sampers to be used and the diameters of the specimens.

Although it is ideal to implement samplings of specimens with the in-situ engineering properties of the original soil completely preserved in the specimens, such samplings are difficult to achieve in reality. In contrast, depending on the purposes of design and construction, there are cases which do not require very high quality specimens. Thus, it is important to determine the allowable amount of decrease in quality of the specimens to be sampled and the degree of actual quality of the sampled specimens. Furthermore, because the disturbance of specimens associated with sampling directly affects the test results and has great influences not only on the accuracy of design and construction but also on construction costs and safety, it is necessary to have a basic understanding about the disturbance of specimens. The factors causing the disturbances during samplings at sites are summarized in **Table 3.6.4**.

Table 3.6.4 Factors Causing Disturbances to Sampled Specimens

<table>
<tr><th colspan="3">Soil specimen</th><th colspan="2">Soil and soft rock specimen</th></tr>
<tr><td rowspan="5">Ground related phenomenon that causes a disturbance</td><td colspan="2">Swelling or constriction (volume change)</td><td rowspan="9">Drilling process</td><td>Rolling phenomenon associated with the rotation of the boring rod and sampler</td></tr>
<tr><td colspan="2">Shear deformation (form change)</td><td>Excess supply of pressure to the bit</td></tr>
<tr><td colspan="2">Temperature change</td><td>Mismatch between the protrusion length of the shoe and the soil hardness</td></tr>
<tr><td colspan="2">Chemical change</td><td>Abrupt increase in the pressure of drilling fluid due to the stagnation of slime around the bit</td></tr>
<tr><td colspan="2">Change in moisture content and degree of saturation</td><td>Insufficient supply of drilling fluid</td></tr>
<tr><td rowspan="7">Sampling process that causes a disturbance</td><td colspan="2">Release of stress in ground</td><td>Rock pieces wedged between the inner face of the bit and the core</td></tr>
<tr><td rowspan="6">Mechanical disturbance</td><td>Shear and compression by a boring bit</td><td>Rock pieces wedged in a circular pattern between the inner tube of the sampler and the core</td></tr>
<tr><td>Shear and compression by a sampling tube</td><td>Outflow of fine fractions due to drilling fluid</td></tr>
<tr><td>Suction, tension or torsion when a sampler is pulled up</td><td>Rolling of a block when drilled by a bit</td></tr>
<tr><td>Impact and vibration when a sampler is dismantled or sealed</td><td>Specimen handling</td><td>Improper handling of a specimen when taken out of a sampler, transported or stored, etc.</td></tr>
<tr><td>Impact when a specimen is transported</td><td>Other</td><td>Release of a stress or slaking</td></tr>
<tr><td>Shear and compression when a specimen is extruded or formed</td><td></td><td></td></tr>
</table>

3.6.3 Quality Evaluation at Sites and Handling of Disturbed Specimens

The evaluation of the quality of specimens means the determination of their degree of disturbance. The determination of the degree of disturbance at sites is to determine whether or not additional samplings of specimens are required. The determination criteria include the visual inspection of specimens and specimen extraction rates (collection rates). In addition, the determination and evaluation criteria for soft rock include visual inspections of cores and comparisons of the visual inspection results with wall observation images. When using less disturbed specimens as test pieces for laboratory tests, it is necessary to minimize the influence of disturbances on specimens by giving due consideration to the handling of the specimens, such as collection (sampling), transportation, storage and shape forming. When collected specimens are determined to be disturbed, they cannot be used for tests to examine mechanical properties in general but can be used for tests to understand physical properties. Even disturbed specimens can be sufficiently utilized depending on the use purposes and properties of the soil. It is also necessary to understand that disturbances cannot be completely eliminated even after making every effort to minimize them.

3.7 Laboratory Soil Tests

The laboratory soil tests to be conducted for collected specimens are listed in **Table 3.7.1**.

Table 3.7.1 List of Laboratory Test Items to be Conducted for Collected Specimens by the Type of Soil

Classification	Test item	Standard	Disturbed specimen		Less disturbed specimen	
			Sandy soil	Cohesive soil	Sandy soil	Cohesive soil
Physical test	Density of soil particles	JIS A 1202 JGS 0111	○	○	○	○
	Moisture content	JIS A 1203 JGS 0121	△ *1	○	○	○
	Grain size (sieving)	JIS A 1204 JGS 0131	○		○	
	Grain size, sieving + sedimentation	JIS A 1204 JGS 0131	△ *1	○	△ *1	○
	Liquid and plastic limits	JIS A 1205 JGS 0141	△ *1	○	△ *1	○
	Wet density	JIS A 1225 JGS 0191			○	○
Mechanical test	Uniaxial compression	JIS A 1216 JGS 0511				○ *2
	Simplified CU test					○
	Triaxial compression and tension (under UU, CU, CD conditions, etc.)	JGS 0521 to 6			○ *3	○ *4
	Cyclic triaxial test: Liquefaction properties	JGS 0541			○ *5	○ *5
	Cyclic triaxial test: Dynamic deformation properties	JGS 0542			○ *6	○ *6
	Cyclic hollow torsional shear (dynamic deformation properties)	JGS 0543			○	○
	Consolidation (staged loading)	JIS A 1217 JGS 0411				○ *7
	Consolidation (constant strain rate loading)	JIS A 1227 JGS 0412				○ *7

○: Test item normally required △: Test item required if necessary

Note:

*1: In the case of the necessity of obtaining D_{10} through sedimentation analysis tests for the calculation of uniformity coefficients, moisture and sedimentation analysis tests shall be conducted. In addition, for soil with fine fraction contents of 15% or more, liquid and plastic limit tests shall be conducted to correct the equivalent *SPT-N values* using plastic indexes.

*2: Two to three test pieces shall be prepared from one specimen for each uniaxial compression test.

*3: For triaxial compression tests of sandy soil, specimens representing each sandy layer constituting the ground shall be collected, and three to four test pieces shall be prepared from one specimen.

*4: For triaxial test compression tests of cohesive soil, specimens representing each cohesive layer constituting the ground shall be collected, and three to four test pieces shall be prepared from one specimen. When it is necessary to consider anisotropy, triaxial tensile tests shall be conducted in addition to triaxial compression tests.

*5: In the case of liquefaction determination through FLIP analyses or cyclic triaxial tests, four or more test pieces shall be prepared from each specimen collected from the object layers.

*6: For tests to obtain the nonlinear properties of ground required for earthquake response analyses, etc, one test piece shall be prepared from each specimen representing each object layer in general.

*7: For complex ground or ground having particular difficulties with consolidation settlement, laboratory tests shall be conducted for the increased number of specimens and test pieces. Generally, one test piece is prepared from each specimen; however, multiple test pieces shall be prepared from each specimen when it is necessary to evaluate the variation in soil data in each layer by obtaining a sufficient number of test results. Furthermore, staged loading and constant strain rate loading shall be combined when testing intermediate soil (low plasticity cohesive soil), pseudo-over-consolidated cohesive soil and specimens at large depths.

The soil test items required for each type of ground survey and surveying purpose are summarized in **Table 3.7.2**.

Table 3.7.2 Soil Test Items Required for Each Type of Ground Survey and Surveying Purpose

Soil test item		Type of ground survey and main surveying purpose						
		General			Ground improvement (cement based hardener, etc.)		Earthquake countermeasure	
		Foundation	Stability	Settlement	Preliminary mixed proportion test	Confirmation of improvement result	Prediction and determination of liquefaction	Examination of residual deformation after an earthquake
Density of soil particles, moisture content, grain size		○	○	○	○	○	○	○
Liquid and plastic limits		○	○	○	○	○	○	○
Wet density		○	○	○	○	○	○	○
Uniaxial compression		○	○	-	○	○	△	○
Triaxial compression and tension		○	○	-	△	△	△	○
Consolidation (staged loading, constant strain rate loading)		○	-	○	△	△	-	-
Cyclic undrained triaxial (liquefaction)		-	-	-	-	-	○	○
Dynamic deformation	Cyclic undrained triaxial	-	-	-	-	-	○	○
	Hollow torsional	-	-	-	-	-	△	△

Note: ○: Test item normally required △: Test item required if necessary －: Not required

3.8 Geophysical Exploration

Geophysical exploration is a collective term describing survey technologies that indirectly analyze ground properties using physical quantities measured in the ground or in the sea. When implementing geophysical exploration, it is most important to select geophysical exploration methods that are appropriate for the survey purposes and the depths of the survey objects, taking into consideration the characteristics and limitations of the respective methods. Because the available exploration depths vary method by method, it is important to select the appropriate methods corresponding to the object depths.

The major geophysical exploration methods used in ground surveys are listed in **Table 3.8.1**.

Table 3.8.1 Geophysical Exploration Methods Used in Ground Surveys

Name of geophysical exploration method	Physical quantity to be measured	Physical quantity to be obtained	Information to be surveyed	Applicable depth			Exploration efficiency	Main purpose	Remarks
				Less than 10 m	Less than 100 m	100 m or deeper			
Seismic exploration	First arrival time of seismic wave	Elastic wave velocity	Cross-sectional structure	○	◎	○	○	Foundation ground property survey	Evaluation of mechanical properties such as rock classification
Surface wave exploration	Surface wave	Surface wave velocity	Cross-sectional structure	○	○		◎	Foundation ground property survey, liquefaction prediction, porosity survey	2 types: impulse generator type and multichannel type

Name of geophysical exploration method	Physical quantity to be measured	Physical quantity to be obtained	Information to be surveyed	Applicable depth			Exploration efficiency	Main purpose	Remarks
				Less than 10 m	Less than 100 m	100 m or deeper			
Microtremor Exploration	Microtremor travel time	Frequency responses, ground structure	Ground frequency responses	○	○	△	◎	Evaluation of ground frequency responses, structure of underground around facility	For earthquake resistance design of structures
Electric exploration (Resistivity method)	Electric potential distribution	Electric resistance	Cross section	○	◎	○	○	Groundwater, landslides, tunnel routes	Availability of alternative methods obtaining IP or self-potential in place of specific resistance
Ground penetrating radar	Electro-magnetic wave propagation travel time	Reflection plane depth	Cross-sectional anomaly	◎	△		◎	Cavities, underground pipes, underground objects, digs	Development of special types of equipment mounted on vehicles etc.
Electro-magnetic exploration	Electro-magnetic induction field	Electric resistance, electric conductivity	Planar anomaly	△	△		○	Groundwater, landslides, brief survey of faults	Development of many types including air exploration
Sonic exploration	Reflection time	Reflection plane depth	Cross sectional layer boundary		○	△	○	Sediments, bottom ground structures such as faults	Applicable only to sea (water) surveys
(Offshore) magnetic exploration	Magnetic field	Magnetic anomaly	Planar anomaly	○			○	Underground metal objects such as bombs	Also applicable to ground surveys
Tomography	Elastic wave, artificial electric field, electro-magnetic wave	Elastic wave velocity, Electric resistance, electro-magnetic wave velocity	Cross section		○	○	△	Detailed ground survey for neighboring construction and other purposes	Application of X-ray CT technology, necessity of multiple boreholes

Note 1: Applicable depth: ◎ optimal, ○ moderately applicable, △ just applicable

Note 2: Exploration efficiency: ◎ readily executable, ○ moderately executable, △ executable with extensive preparation of equipment

Geophysical exploration is implemented with optimal measuring or analysis methods selected in accordance with the purposes of exploration and object ground. For the details of the respective methods, refer to the manuals[4) to 13)] published by relevant academic conferences and various specialized books. Among the geophysical exploration methods introduced above, the sonic and magnetic exploration methods are those frequently used in the geophysical exploration of marine areas, including ports, while the other exploration methods have been used mostly in the geophysical exploration of land areas.

In principle, geophysical exploration implemented on ground surfaces undergoes a degradation of resolution with an increase in exploration depths. Seismic wave and electric exploration methods can enhance the resolution by shortening the measuring intervals; however, their resolution is still affected by the exploration depths. Geo-tomography which uses boreholes is a geophysical exploration method capable of curbing the degradation of resolutions associated with deep exploration. However, in many cases, geo-tomography which uses multiple boreholes undergoes a degradation of resolution if the borehole intervals are widened because of the same reason as in the case of the degradation of resolution in a depth direction.

In geophysical exploration, obtaining the distribution of physicality through the “measurement” of physical quantities is defined as “analysis,” and estimating the ground properties using the analyzed distribution of physicality is defined as “interpretation.” The following section explains about the points of caution common to all aspects of geophysical exploration in terms of “measurement,” “analysis” and “interpretation.”

(1) Measurement

Geophysical exploration requires a large ratio (SN ratio) of signals to be detected by a sensor to noise in order to improve the reliability of the measurement results and search accuracy. It is necessary to confirm the quality of the data obtained through measurements at sites so as to ensure that remeasurements are to be made promptly once abnormalities are found in the data.

(2) Analysis

In geophysical exploration, a popular analysis method is the inverse analysis, which uses a computer in a manner that processes a large volume of data into the cross-sectional distribution of physicality below the measurement lines. The inverse analysis which has been frequently used first sets an initial model of a distribution of physicality and then repeats modifications of the model until the calculation results of the model become almost equal to the actual measurements. In the inverse analysis, however, it is necessary to pay attention to the following items so as not to possibly draw a false analysis result called a “false image.”

① Avoidance of unnecessarily fine segmentalization of the analysis model
② Inclusion of an area outside the exploration area in the analysis model
③ Setting of an appropriate initial model
④ Selection of appropriate analysis parameters
⑤ Elimination of measurement results with unfavorable SN ratios from the reference values
⑥ Avoidance of unnecessary repetitions

Even in an automated analysis, it is necessary to comprehensively determine the appropriateness of each analysis result because the number is not only one but multiple results, as is the case with conventional analyses.

(3) Interpretation

The analysis results of geophysical exploration are only a part of many pieces of the physical properties (e.g., elastic wave velocities in seismic wave exploration), and, therefore, multiple geological areas with identical physical properties do not always have a common geology. When estimating the ground properties from the distribution of physical properties, it is necessary to comprehensively determine the ground properties by referring to other ground survey results such as boring. Generally, there are ranges in the values that ground properties can take, but the ranges are affected by several conditions such as the presence or absence of groundwater and the degrees of weathering, and they vary place by place. In actual situations, the information obtained through the observation of geophysical exploration is important in that it shall be used together with other survey results for the identification of the physical property values suitable for the respective sites and the interpretation of the distribution of the physical properties.

3.8.1 Elastic Wave Exploration

Elastic wave exploration is to obtain an underground velocity structure by measuring elastic waves (P or S waves) artificially generated at locations close to the ground surface. A schematic layout of the measuring instruments is shown in **Fig. 3.8.1**. Because of its proven performance records and ability to extensively obtain planar information, elastic wave exploration has often been used for surveys in the early stages of planning and in the schematic design stage. Although elastic wave exploration is an effective and economical survey method in the schematic design stage, the application of the method shall be determined by taking into consideration its compatibility with the following conditions.

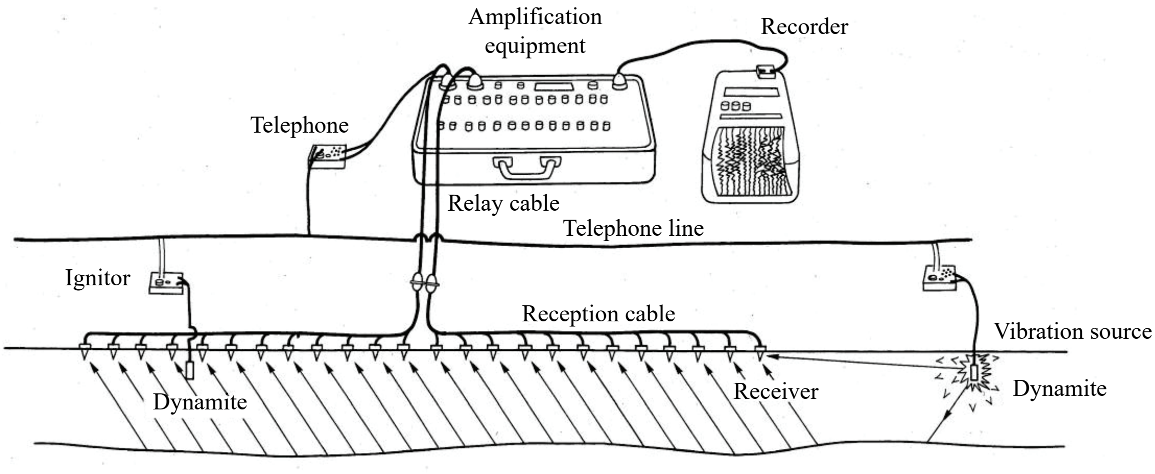


Fig. 3.8.1 Schematic Drawing of Measurements Using Elastic Wave Exploration

(1) Exploration depth

When increasing the exploration depth, it is necessary to extend the length of the measuring line. However, dynamite with a strong vibratory force can increase the exploration depths up to 100 to 200 m. In the case of vibration generation without using dynamite, the exploration depths are up to 30 m. The length of the measuring line needs to be 5 to 10 times longer than the exploration depths. The measuring line is basically laid immediately above the point for exploring the velocity structure in a direction preferably perpendicular to the contour lines which show the elevation of the site.

(2) Intervals of geophones

Seismic energy released from vibration source and arriving at the surface of the ground is detected by geophones. The intervals of geophones are generally 5 and 10 m in the case of shallow and deep exploration depths, respectively. When exploring at a small scale or in shallow areas in detail, the intervals are 1 to 2 m in many cases. The vibration generation sources are generally installed at intervals of 30 to 60 m, but as is the case with geophones, shorter intervals can be used as needed. The quality of the measured data shall be confirmed before setting up instruments for subsequent measurements and remeasurements shall be implemented as needed.

(3) Analysis results

The velocity distribution in the ground can be obtained by analyzing the measured data. The following items can be understood through the geotechnical interpretation of ground structures based on comparisons of the velocity distribution with the boring survey results and existing geological reports.

① Types and hardness or softness of soil and rock, as well as degrees of cracks, weathering and transformation
② The presence or absence and sizes of fault fracture zones
③ Geotechnical evaluation of ground conditions and the selection of foundation ground

These items have close links with engineering properties such as ground deformation and strength characteristics, which explains why elastic wave velocities have been used as engineering indexes.

(4) Use of exploration results

Elastic wave velocities are used for rock (geological) classification, the stability evaluation of cut slopes, the evaluation of difficulties in excavation, and the evaluation of the foundation ground of structures. In addition, because the relationships of the elastic wave velocities of ground with engineering properties of the ground such as deformation and strength have been known, elastic wave velocities can be used as index values for the engineering properties in the design and construction stages.

(5) Points of cautions when applying elastic wave exploration

When applying elastic wave exploration, it is necessary to pay attention to the following points.

① The analyses are based on a layered velocity structure where elastic wave velocities are increased with increasing depth, and, therefore, elastic wave exploration may not be applied to ground having a distribution of hard rock layers on soft rock layers with elastic wave velocities high in the hard rock layers and low in the soft rock layers.

② Even ground with a layered structure where elastic wave velocities are increased with increasing depths, there may be problems with blind layers where the existence of thin layers may not be detected because waves refracted and propagated through the thin layers cannot be observed as initial movements by receivers on the ground surface. In such cases, the depths of the foundation layers may be estimated as being shallower than the actual layers.

③ When high velocity layers are distributed parallel to or at a sharp angle with the measuring line, it is necessary to improve the measuring accuracy such as by installing an additional measuring line perpendicular to the original line.

3.8.2 Surface Wave Exploration

When artificially generating elastic waves using impacts from a hammer or a vibrator on the ground surface, surface waves propagate along the ground surface in addition to the propagation of body waves (P or S waves) in the ground. An exploration using the surface waves (Rayleigh and Love waves) is called surface wave exploration and Rayleigh waves are used in many cases.

The propagation velocities of Rayleigh waves V_r along ground surfaces are generally 0.9 to 0.95 times the velocities of S waves V_s. Rayleigh waves widely vibrate in a direction perpendicular to the ground surface and reflect the velocities of S waves V_s in the ground up to the depth corresponding to the wavelengths of the Rayleigh waves. Because the velocities of surface waves propagating the ground having horizontally layered structures vary depending on the wavelengths (frequency), the S wave velocity structures of ground can be obtained by measuring and analyzing the surface waves. The velocities of S waves V_s in the ground are the values directly related to shear modulus, indispensable for understanding and examining the dynamic properties of the ground and are important input values in the earthquake resistant design of structures.

Surface wave exploration analyzes ground by measuring artificially generated surface waves propagating through the ground. In surface wave exploration, there are two measuring systems: a surface wave exploration measuring system using a motion exciter and a multichannel surface wave exploration measuring system. In the former system, surface waves are generated by a motion exciter while changing frequency and measured with two receivers (as shown in **Fig. 3.8.2**). In the latter system, impact waves are generated by a hammer or a heavy bob, and the surface waves propagated through the ground are measured with a large number of receivers arranged on a measuring line (as shown in **Fig. 3.8.3**).

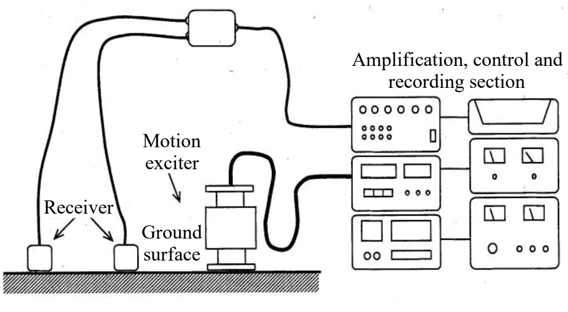


Fig. 3.8.2 Surface Wave Exploration System Using a Motion Exciter

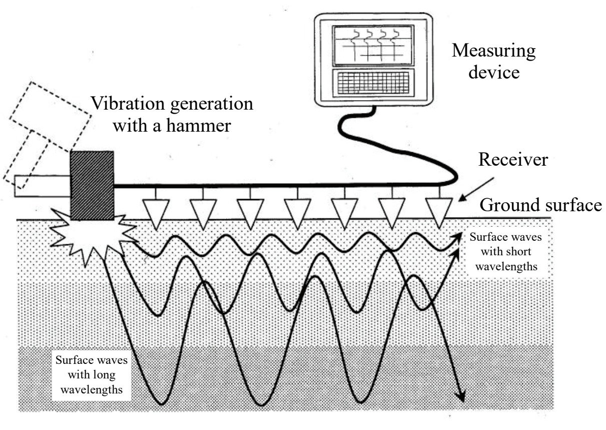


Fig. 3.8.3 Multichannel Surface Wave Exploration System

Surface wave exploration has the following characteristics and conveniences which are not available through normal elastic wave exploration.

① The S wave velocity structures obtainable through surface wave exploration have been widely used in engineering evaluations of ground and are particularly effective in ground surveys for earthquake resistant designs.

② Surface wave exploration is applicable to ground with alternate layers of soft and hard rock to which the elastic wave exploration is not applicable. Thus, surface wave exploration is effective for surveys of alluvial, reclaimed and filled ground, and underground cavities. In addition, surface wave exploration can be implemented even on paved roads.

③ Because of its high impulse generation efficiency and the availability of large signals, surface wave exploration can be implemented in noisy urban areas with a large number of miscellaneous vibrations.

Taking advantage of the characteristics mentioned above, surface wave exploration can be used for a wide range of survey objects including such special cases as ground surveys in urban areas, and surveys for laying underground pipe lines, liquefaction predictions, embankment diagnosis, ground improvement evaluations, underground cavities and pavement subgrades. **Fig. 3.8.4** shows an example of the results of a surface wave exploration at a survey point in reclaimed ground using a motion exciter. In the figure, the measured data and an S wave velocity structure are shown in the right and central columns, respectively. As can be seen in the figure, a boring, a standard penetration test and velocity logging were conducted at the survey point, and correlations can be shown when comparing the *SPT-N values* with the S wave velocities obtained through velocity logging. Furthermore, the S wave velocity structure clearly detects the existence of a low velocity intermediary layer. Generally, the propagation velocities of Rayleigh wave V_r correspond to depths equivalent to 1/2 to 1/3 of the wavelengths.

Fig. 3.8.5 shows an example of ground survey results obtained through multichannel surface wave exploration, which can be used for obtaining two-dimensional S wave velocity structures of ground. As can be seen in the figure, there is a low velocity region in the first half of the measuring line (left side), and even a single survey object site has a significantly varied ground structure. Because S wave velocities have correlations with *SPT-N value*s, surface wave exploration can provide effective data to clarify three-dimensional ground conditions in entire survey areas by interpolating the data at the boring survey points.

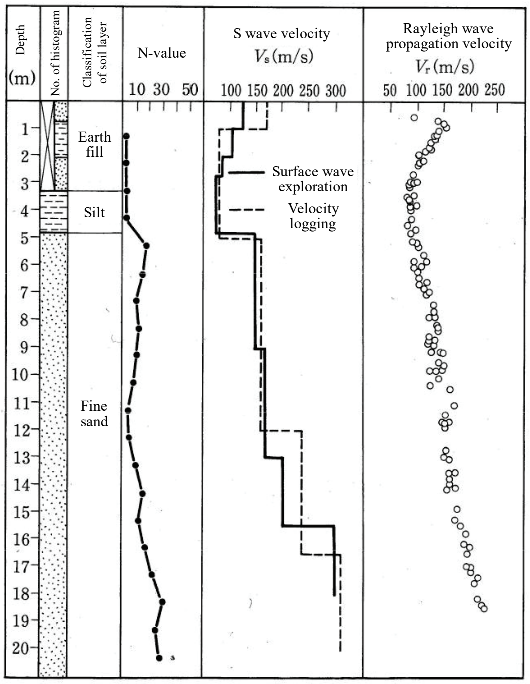


Fig. 3.8.4 Comparison of the Results of a Surface Wave Exploration Using a Motion Exciter, Boring and Velocity Logging

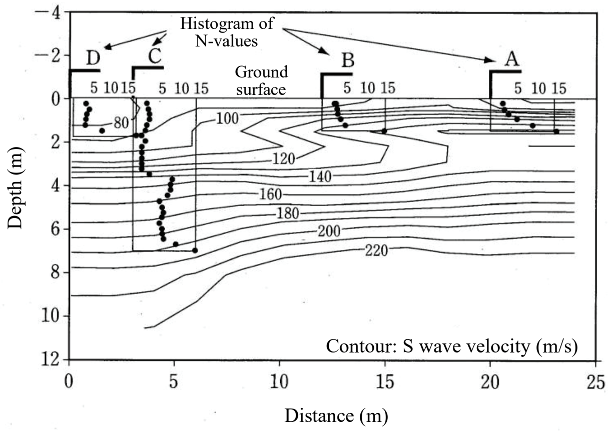


Fig. 3.8.5 Example of a Cross-Sectional Velocity Structure Using Multichannel Surface Wave Exploration

3.8.3 Microtremor Observation

Microtremors are defined as a collective entity of wave motions propagated from distant artificial vibration sources such as distant transportation facilities and factory machines, and natural vibration sources such as winds, tides, waves and volcanic activities. Microtremors are relatively stable waves with a predominant period of 0.1 to 2.0 seconds (and a frequency of 0.5 to 10 Hz) and are alternatively called short-period tremors. Microtremors can be effectively used for the estimation and evaluation of the oscillation properties of the object ground such as the predominant periods and amplification properties.

Because microtremors can reflect the specific properties of the ground during their propagation process, they can be used as important materials for representing the oscillation properties of the ground. That is, it has been known that the predominant periods of microtremors are consistent with earthquake ground motion properties, and, therefore, spectral analyses of the waveforms obtained through microtremor observation enable the predominant periods of surface layers and amplification properties of the object ground to be estimated. Thus, microtremors have been used as important sources of earthquake resistant design materials in the earthquake engineering field. Recently, it has also been made clear that microtremors behave like surface waves. Taking advantage of the behavior of microtremors, the array exploration method (for simultaneous observation of microtremors at multiple locations on a ground surface) has been used for geological structure surveys (refer to **Reference [Part II], Chapter 1, 3.8.4 Microtremor Array Exploration**).

There are four microtremor observation methods, as shown below.

① Method for observing microtremors only on ground surfaces

② Method for simultaneously observing microtremors on ground surfaces and inside boreholes or at multiple locations in foundation ground

③ The array exploration method for geological structure surveys

④ Intra-structure observation method to understand the earthquake ground motion properties of structures

Microtremor observations are generally applicable to ground up to a depth of about 100 m. There are also cases of the array exploration method being used for geological structure surveys at depths exceeding 1,000 m.

The following information is available through the measurement and analysis results of microtremors.

① Respective spectrum analysis results, predominant periods of surface layers and the classifications of ground in the case of microtremor observations on ground surfaces, or on both the ground surface and inside boreholes

② Dispersion curves of surface waves (relationship between periods and phase velocities) and the estimation results of S wave velocity structures in the case of the array exploration method

③ The characteristic periods and damping constants of structures in the case of intra-structure observation

The waveform data of microtremors at observation points is the most important material obtainable through field surveys. It is necessary to display waveforms chronologically recorded with a recorder and confirm whether or not the waveforms are applicable to analysis processes. Because there is a high possibility that the measurement data includes not only the oscillation properties of the ground but also the effects of the propagation paths of the microtremors, it is important to create measurement environment conditions to obtain stable analysis results and to evaluate the oscillation properties while taking into consideration the influence of the propagation paths of microtremors.

Microtremor observations are particularly effective when the contrast between the surface and foundation ground is clear. Furthermore, since microtremors are extremely small ground oscillations, careful attention is required because traffic, factories and construction or other unexpected artificial impulses around the survey sites can result in possible noise in the microtremor observation data. The environmental conditions such as strong winds and rain can also be sources of noise that intervene with normal microtremor observations. Thus, it is necessary to select appropriate observation dates.

For the use of microtremor observations in the evaluation of earthquake ground motions for verification, refer to **Reference [Part II], Chapter 1, 4 Observation Related to the Evaluation of Earthquake Ground Motions for Verification**.

3.8.4 Microtremor Array Exploration

Microtremor array exploration is to observe permanent vibrations (oscillations) on ground surfaces and to estimate geological structures using the dispersion qualities of wave motions. The surface of the earth has been subjected to microtremors due to natural phenomena (winds, tides, waves, volcanic activity, etc.) and artificial noise (vehicles, factories, etc.). In the theory of elasticity, microtremors are defined as a collective entity of body waves (P and S waves) and surface waves (Rayleigh and Love waves). Among these body waves, microtremor array exploration focuses on predominant surface waves, detects them through planar observation networks and estimates underground velocity structures through inverse analyses of the dispersing qualities of the surface waves. The dispersing qualities of surface waves is a phenomenon where waveforms seem to disperse with increasing propagation distances because the propagation velocities of surface waves differ depending on frequency. The dispersion curves (the relationship between the phase velocities and frequency) which show forms reflecting the ground structures (as shown in **Fig. 3.8.6**) can be used for estimating the structure of the object ground. However, because microtremor array exploration is based on horizontally layered structures in principle, it is difficult to apply it to steep sloping ground and complex ground with fault structures.

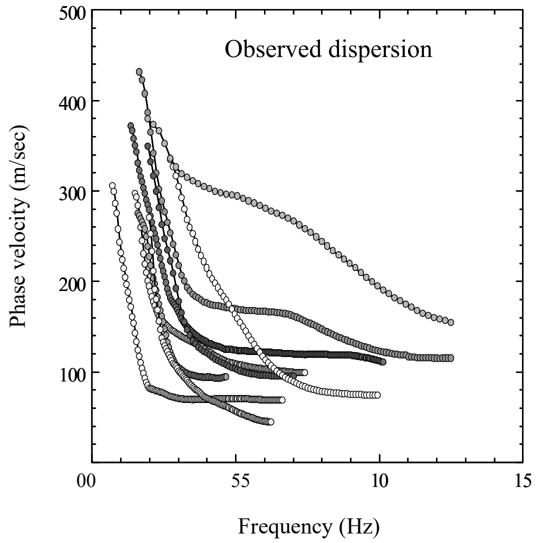


Fig. 3.8.6 Example of Dispersion Curves

(1) Exploration procedure

The procedure for microtremor array exploration is described below (**Fig. 3.8.7**).

① Observation of microtremor arrays: observation of microtremors propagated from all directions with seismographs put in a planar arrangement on the ground surface

② Detection of surface wave phase velocities: creation of dispersion curves through observed microtremors (observed dispersion curves)

③ Estimation of S wave velocity structures: estimation of S wave velocity structure models corresponding to observed dispersion curves through inverse analyses

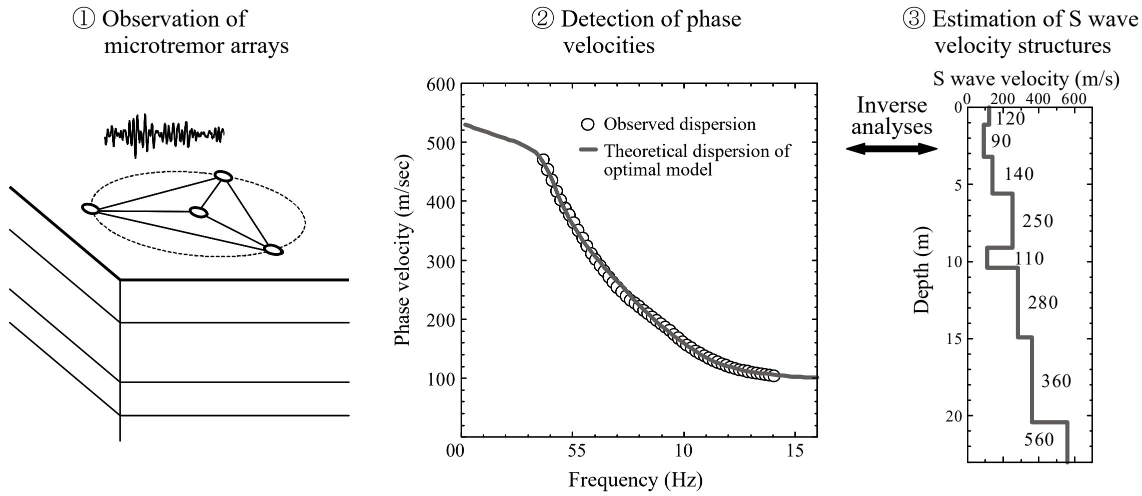


Fig. 3.8.7 Schematic Drawing of the Microtremor Array Exploration Procedure

(2) Arrangement of seismometers

① **Triangle array**

Microtremors are observed through an observation network (hereinafter referred to as an "array") with seismometers put in a planar arrangement. For extracting the dispersing qualities of surface waves, seismometers are arranged in a concentric fashion at equal intervals with one seismometer as the center. Generally, an arrangement of the minimum number of seismometers is a regular triangle array (one at the center and three at each corner of the triangle, as shown in **Fig. 3.8.8**). There are also cases of microtremor array exploration which combine large and small regular triangle arrays sharing a center point depending on the depths and ground conditions.

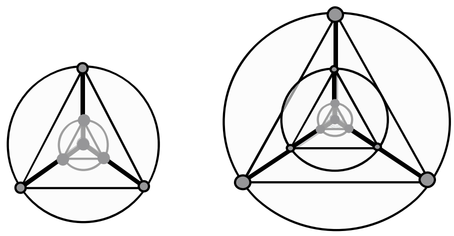


Fig. 3.8.8 Basic Arrangement of Seismometers in Microtremor Array Exploration

② **Chain array**

The chain array comprises seismometers arranged in two rows and a central seismometer shifted in sequence to observation points between the rows. Microtremors are observed while shifting the central seismometer and dispersion curves are created using the measurements of three neighboring seismometers. An example of the arrangement of seismometers and schematic drawing of a chain array is shown in **Fig. 3.8.9**.

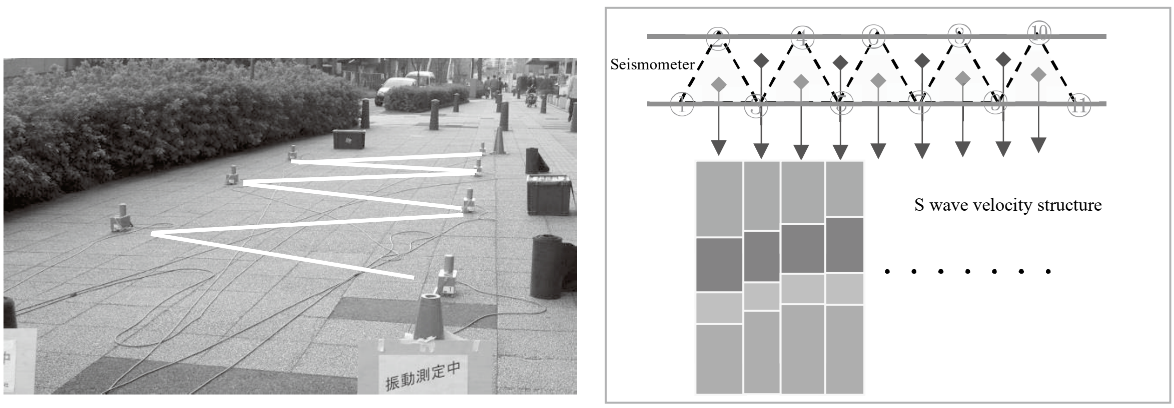


Fig. 3.8.9 Seismometer Arrangement and Schematic Drawing of a Chain Array

(3) Exploration results

A microtremor array exploration can be used for obtaining the structural drawings representing S wave velocities within array radii, which enable the estimation of geological compositions and the identification of the engineering foundation depths ($V_s \geqq 300$ m/s). **Fig. 3.8.10** shows an example of a structural drawing of S wave velocities. As can be seen in **Fig. 3.8.11**, a microtremor array exploration also enables the distribution patterns of engineering foundations to be understood.

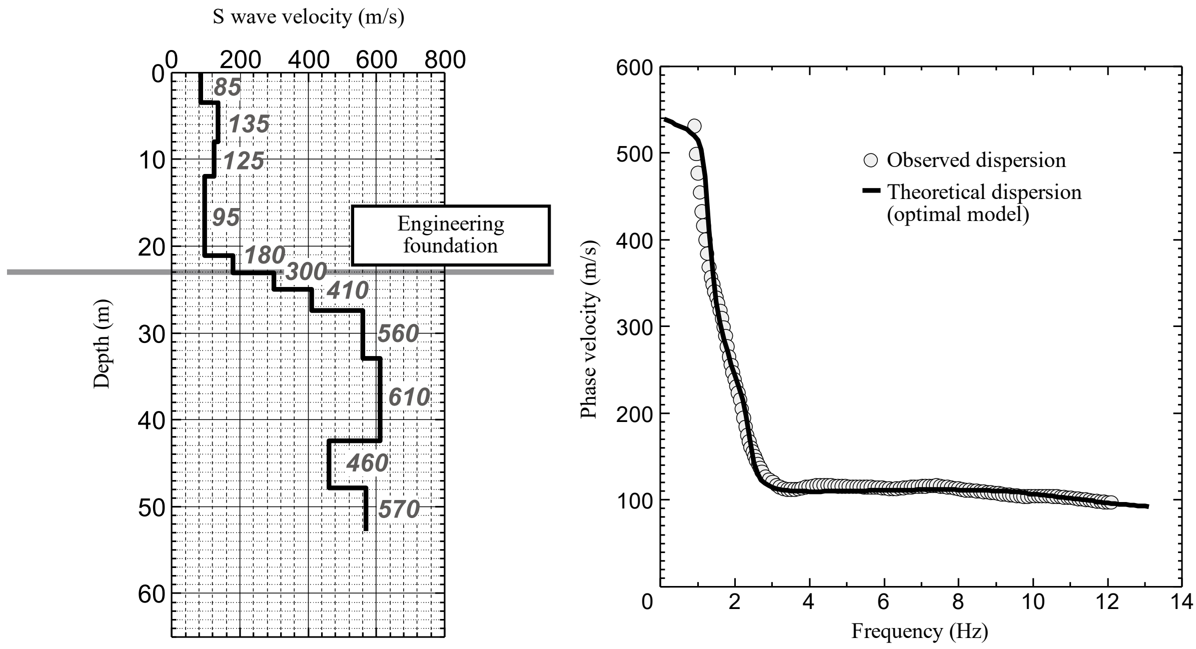


Fig. 3.8.10 Structural Drawing of S Wave Velocities

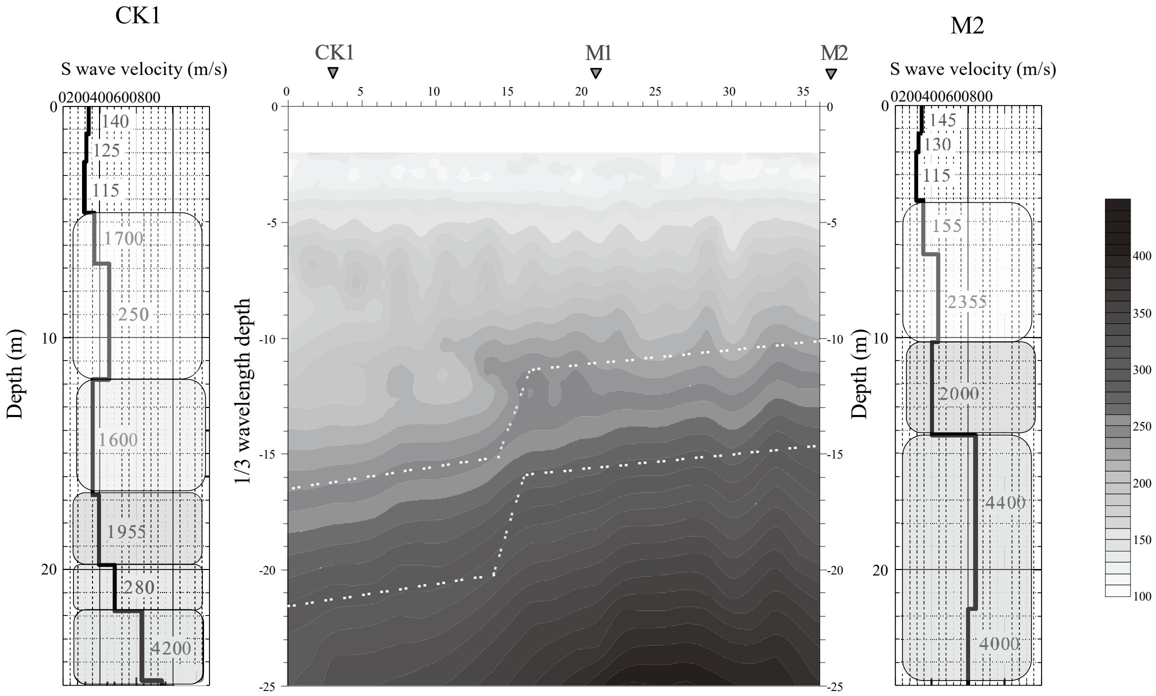


Fig. 3.8.11 Distribution Pattern of an Engineering Foundation Obtained through Chain Array Exploration
(The figure in the middle shows the depths converted from wavelengths measured through a chain array exploration, and figures on both sides show the inverse analysis results.)

3.8.5 Electric Exploration

Depending on their properties and surrounding environments, some portions of geological layers allow electric current to easily pass, while other portions do not. Electric current conductivity is expressed by the physical property called specific resistance (in the unit of Ω·m), and the electric current conductivity is enhanced with decreasing specific resistance. When applying electric current to ground, the electric potential distribution is generated according to the distribution of specific resistance in the ground. The electric exploration specific resistance method is one of the electric explorations which estimates specific resistance distribution from the electric potential distribution data measured on ground surfaces. The specific resistance method has been the popular electric exploration method for ground surveys. The specific resistance method is largely classified into ① the vertical exploration method, ② the horizontal exploration method, and ③ the two-dimensional exploration method (including the specific resistance image method, high density electric exploration, the ρa-ρu method, etc.). These methods have been selectively used according to the exploration purposes and their applicability to the geological structures. Among these methods, the two-dimensional exploration method has been generally used.

Particularly, the two-dimensional exploration method is used in the case of electric exploration to be conducted for the purpose of obtaining continuous geological structures along exploration measuring lines, and when the object ground is expected to have two-dimensional structures without significant structural changes in the directions perpendicular to the cross sections to be explored. In contrast, the vertical exploration method is used in the case of obtaining information only in the depth direction of the ground expected to have horizontally layered structures. In addition, the horizontal exploration method is used in the case of obtaining information only on planes or in directions along the measuring lines at specific depths. Electric exploration requires electrode bars to be inserted into the ground so as to measure the electric potential with an electric current applied to the ground. The diagrams of the arrangements of measuring instruments are shown in **Fig. 3.8.12**.

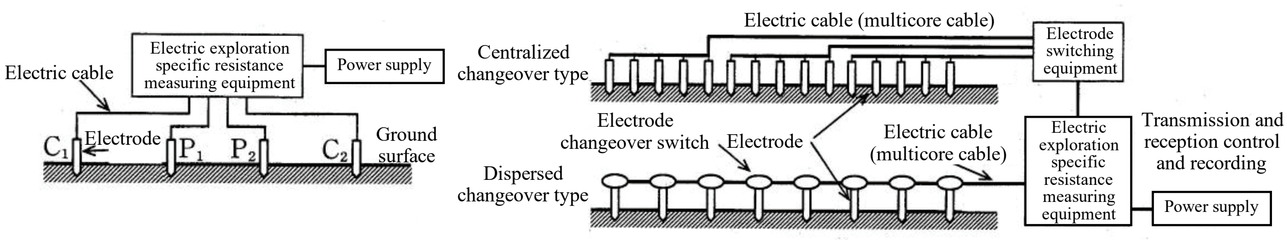


Fig 3.8.12 Diagrams of Electric Exploration Measurement

As shown in **Fig. 3.8.12 (a)**, the vertical exploration method fixes a central point of electrodes and measures potential while expanding the distances between the electrodes, which increases the depths that can be explored. In contrast, the horizontal exploration method fixes the distances between the electrodes and measures potential while moving along the measuring lines.

As can be seen in **Table 3.8.2**, the specific resistance of ground is affected by many factors such as the porosity of geological layers, water saturation, pore water specific resistance, clay mineral contents, and temperature. Generally, ground with higher clay mineral contents and volume water contents (porosity × water saturation) tends to have lower specific resistance.

Table 3.8.2 Factors Affecting the Specific Resistance of Ground and Changes in Specific Resistance Due to These Factors

Low	←	Specific resistance	→	High
High	←	Porosity (saturated state)	→	Low
High	←	Water saturation	→	Low
Low	←	Pore water specific resistance	→	High
High	←	Clay mineral content	→	Low
High	←	Conductive mineral content	→	Low
High	←	Temperature	→	Low

(1) Exploration depths

Exploration depths have a close relationship with the distances of electrodes and the lengths of measuring lines. In plain regions, the depths to be explored by the vertical and horizontal exploration methods are 200 to 300 m with a maximum distance of 400 to 600 m between the electrodes. Generally, electric exploration is applied to ground at a depth of up to 100 m. The depths to be explored by the two-dimensional exploration method are about 300 m. The resolution performance and the analysis accuracy of electric exploration are reduced with an increase in exploration depths. In cases of exploring ground at depths deeper than those mentioned above, it is advantageous to use other types of exploration such as electromagnetic exploration.

(2) Electrode distances

Distances of about 1/10 to 1/15 of the exploration depths are considered to be the minimum electrode distances which can become a determinant factor of the resolution performance of exploration.

(3) Analysis results

The information obtainable through electric exploration is the underground distribution of specific resistance. Although electric exploration cannot directly provide information on the strength of ground, which is an important parameter in geotechnology, it enables geological classification of the object ground to be made in a manner that comprehensively interprets specific resistance distribution patterns in combination with the existing information such as the expected geological structures and boring survey results. The specific resistance distribution maps obtained through the analyses enable the distribution of geological structures, aquifers, fault fracture zones, hydrothermally affected zones, and weathered zones to be estimated. In many cases, fault fracture and hydrothermally affected zones form low specific resistance zones as a result of the inclusion of clay minerals, the increase in porosity due to rock fractions, and the existence of groundwater. Low specific resistance zones can be clearly detected in the cases of ground with granite layers which generally have high specific resistance and ground with Mesozoic and Paleozoic layers. When the low velocity zones obtained through seismic wave exploration correspond to low specific resistance zones, it is highly possible that these zones are fault fracture or hydrothermally affected zones. Furthermore, the two-dimensional exploration method may enable the inclination directions of fault fracture and hydrothermally affected zones to be estimated from those of low specific resistance zones.

(4) Use of exploration results

Geology cannot be unambiguously determined by specific resistance, but the data in **Table 3.8.3** can be used as a guide to determine geological classifications from the specific resistance distribution. Here, the data obtained through the electric logging in **Reference [Part II], Chapter 1, 3.5.3 Logging in Boreholes (Physical Logging)** can be effectively used for improving the accuracy of geological classifications based on specific resistance. One of the characteristics of electric exploration is that specific resistance distribution enables abnormal areas in the ground to be identified in a manner that locates fault fracture zones, alteration zones and high permeability sections in rock by focusing on abnormally low specific resistance areas, and locates cavities by focusing on abnormally high specific resistance areas. In addition, there has been an approach to evaluate ground using the correlation between the specific resistance and elastic wave velocities.

Table 3.8.3 Specific Resistance of Major Rocks, Soil and Water

Type of rock and soil	Specific resistance (Ω·m)	Type of rock, soil or water	Specific resistance (Ω·m)
Granite	3×10^{2}–1×10^{4}	Shale	1×10^{0}–1×10^{1}
Diorite	1×10^{2}–5×10^{4}	Gravel	1×10^{2}–1×10^{3}
Andesite	1×10^{2}–1×10^{1}	Sand	1×10^{0}–1×10^{3}
Basalt	1×10^{3}–1×10^{5}	Clay	8×10^{-1}–1×10^{2}
Loam	1×10^{2}–1×10^{3}	Surface soil	2×10^{2}–1×10^{3}
Conglomerate	1×10^{1}–1×10^{4}	Limestone reek	6×10^{1}–5×10^{5}
Sandstone	3×10^{1}–1×10^{3}	Groundwater (freshwater)	2×10^{1}–8×10^{1}
Mudstone	1×10^{0}–1×10^{2}	Seawater	3×10^{-1}

(5) Points of caution when applying electric exploration

The analysis of the vertical exploration method is based on the assumption that the ground has a horizontal layer structure, and that of the two-dimensional exploration method is based on the assumption that the ground has a two-dimensional geological structure (no changes in the directions perpendicular to the measuring lines). Thus, in the case of ground with significant changes in topographical and geological structures at the side of the measuring line, it is necessary to obtain data along an additional measuring line and to compare the data of both measuring lines. Furthermore, the measuring lines shall be arranged away from electric power transmission lines, railroads and steel structures so as to prevent the exploration results from being affected by noise and abnormal measurements. In the two-dimensional exploration method, it is necessary to expand exploration areas wider than the object areas in consideration of reduced analysis accuracy at the bottoms of the analysis cross section and at both ends of the measuring lines.

3.8.6 Acoustic Exploration

(1) Outline of acoustic exploration

Depending on the exploration systems, acoustic exploration is classified into reflection and refraction systems. The outlines of these systems are shown in **Fig. 3.8.13**.

In the reflection system, for the purpose of obtaining geological structures below the seabed, an oscillator close to the water surface oscillates acoustic waves of a few Hz to a few kHz at constant time intervals, and a receiver receives the acoustic waves reflected and returned from the seafloor and layer boundaries. By continuously repeating the above process, the reflection system obtains the continuity of the seafloor and layer boundaries and the distribution of boundary surfaces in a depth direction through the velocities of the acoustic waves. The reflection system is characterized by the ability to extract only the reflected acoustic waves and to express undulations on layer boundaries with enhanced visual effects.

The reflection system is further classified into single and multichannel methods. The single channel method includes an analogue type, which displays the intensity of the reflected acoustic waves as grayscale images on a recorder, and a digital type, which records the reflected acoustic waves as digital data. The multichannel method includes a streamer type, which oscillates acoustic waves from an exploration vessel and receives the reflected acoustic waves using multiple receivers (a streamer cable) towed by the vessel, and a bay cable type, which receives acoustic waves by laying a cable with multiple receivers attached to it on the seafloor. Generally, the single channel method is used with acoustic wave sources producing low oscillation energy, and the multichannel method is used with acoustic wave sources producing large oscillation energy for obtaining geological structures in deep portions below the seafloor.

The single channel method can be easily implemented and produce reflection cross sections relatively easily. However, in the case of the analogue type, it is difficult to eliminate noises caused by multipath reflection between the seafloor and sea surface and signals transferring information other than geological data. In contrast, the multichannel method measures reflected acoustic waves simultaneously with multicomponent receivers (generally 24 to 48 components), thereby enabling only the necessary information to be extracted with any unnecessary noises eliminated through the digital processing of multicomponent records.

The single channel method is applicable to exploration depths of up to about two times the water depth, and, when applied to greater depths, encounters problems with increasing noise due to multiple acoustic waves reflected off the seafloor. The multichannel method is applicable to sedimentary layers with thicknesses of 2,000 to 3,000 m (when using air guns), and the method’s resolution performance decreases with increasing depths.

In contrast to the reflection system which observes reflected waves, the system which obtains the velocity structures of layers below the seafloor by observing refracted waves using ocean-bottom seismometers or sonobuoys is classified as the refraction system. The refraction system obtains geological structures based on the propagation velocities of acoustic waves in geological layers calculated from the distances between the acoustic wave sources and observation points as well as the differences in the arrival times of the acoustic waves. The refraction system is the application of elastic wave exploration (refer to **Reference [Part II], Chapter 1, 3.8.1 Elastic Wave Exploration**) to marine areas.

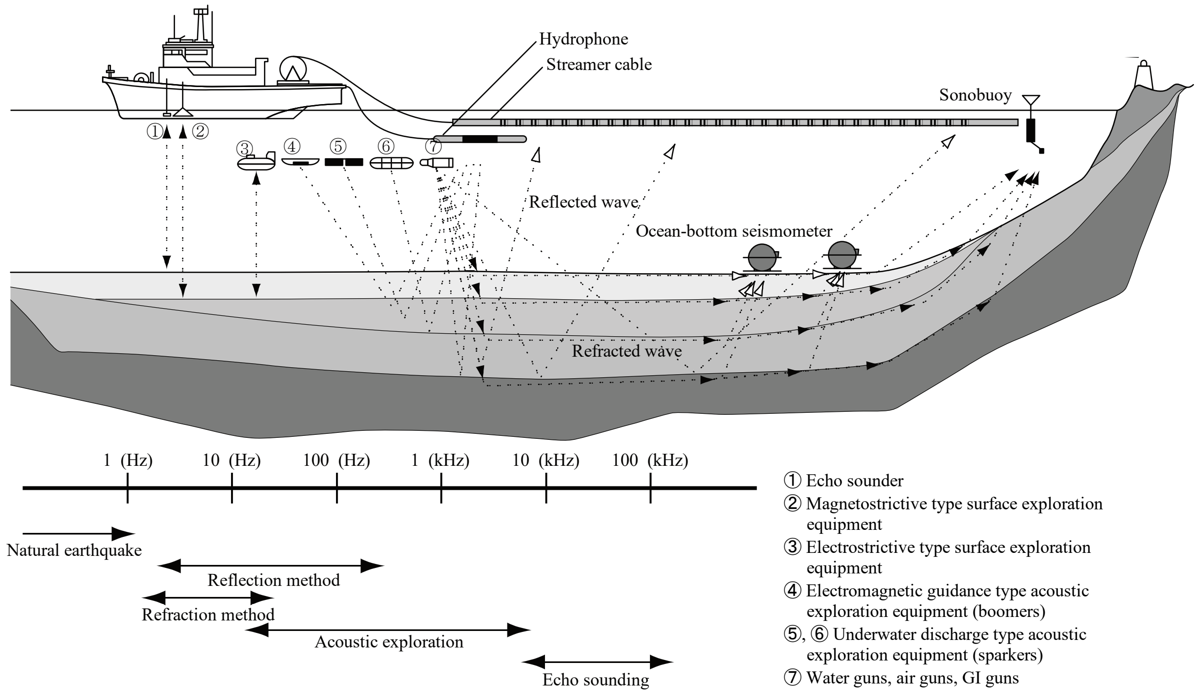


Fig. 3.8.13 Schematic Drawings of Reflection and Refraction Methods

The results of acoustic exploration are used for the determination of the undulations (depth distribution) of foundations below the seafloor, the sedimentary structures such as the thicknesses of sedimentary layers, and geological structures such as faults and folds, and applied to surveys to identify the positions of faults close to coastal facilities. The values obtainable through the reflection method are not the distances but the reflection times of compression waves. Therefore, it is necessary to reliably combine acoustic exploration with boring or sounding surveys so as to confirm which layer boundaries correspond to the reflection surfaces obtained through acoustic exploration.

(2) Properties of acoustic waves

① Propagation velocity

There are two types of acoustic waves: longitudinal waves (P waves) and lateral waves (S waves). Both types of acoustic waves can propagate through solid substances, but only longitudinal wave can propagate through water. The propagation velocity of acoustic waves in seawater is about 1,500 m/s, although it slightly varies depending on the depths, temperatures and salinity. In the multichannel method, the propagation velocities of acoustic waves in each layer are calculated through velocity analyses by collecting traces reflected off common reflecting points.

② Propagation loss

Acoustic waves emitted into seawater propagate as spherical waves. The intensity of the acoustic waves is connected with the distances from the acoustic wave sources and the frequencies. Given that the acoustic waves have identical initial amplitudes, those having low frequencies attenuate less than those having high frequencies. In contrast, in relation to the wavelengths and frequencies of acoustic waves, the wavelengths get longer as the frequency decreases, thereby degrading the measuring accuracy. Thus, although acoustic wave sources with high frequencies are desirable to achieve accurate measurements, the acoustic waves cannot reach distant locations in such cases. Even when trying to increase the amplitudes of the acoustic sound sources so as to allow acoustic waves to propagate with high frequencies, there are limitations. Thus, it is necessary to determine the frequency of the acoustic exploration equipment by taking into consideration the object water depths (exploration depths) and the desired accuracy.

③ Reflection and refraction

When acoustic waves pass through two different media, they are partially reflected and refracted at a boundary between the media. The product ($\rho \times v$) of the density of a medium ρ and a propagation velocity v is called acoustic impedance. The reflectivity coefficient becomes larger with an increase in the difference between the acoustic impedance of the two media.

④ **Directivity of acoustic waves**

The acoustic pressure is maximized on the axis of the vibration plane. The angle from the axis to a point where the acoustic pressure in the axial direction becomes 1/2 of the maximum (half value angle) is normally used as an index expressing the sharpness of the directivity of the acoustic waves. The types of acoustic exploration equipment with small oscillation energy, such as electro-strictive type equipment using electro-strictive vibrators and magneto-strictive type equipment using magneto-strictive vibrators, have directivity to some extent. In the cases of electromagnetic guidance and underwater discharge type equipment, as well as the types of equipment using impulsive waves of compressed air, oscillated acoustic waves propagate in all directions. In these cases, receivers use piezoelectric elements (hydrophones) which are also omnidirectional.

⑤ **Resolution**

The resolution in acoustic exploration is classified into a vertical resolution, representing the limitations on the detection of differences between substances arranged one above the other, and a horizontal resolution, representing the limitations on the detection of differences between substances arranged next to each other.

(a) Vertical resolution

The limitations on distinguishing substances with identical polarities using reflected acoustic waves is a half cycle of the predominant frequency. Thus, when considering two-direction observation, the vertical resolution is 1/4 of the wavelengths. For example, given that the acoustic wave source has a predominant frequency of 1,000 Hz and the propagation velocity of P waves in the ground is 2,000 m/s, the wavelength λ becomes 2 m and the vertical resolution, 1/4 of the wavelength, is calculated to be 0.5 m. However, this is a theoretical example, and, in reality, it is difficult to oscillate a waveform of a complete single pulse, and the actual vertical resolution is about 1 to 1/2 the wavelength.

(b) Horizontal resolution

The horizontal resolution varies depending on the directivity of the transmitters and receivers as well as the density of the reflection points. In terms of the directivity, electro-strictive and magneto-strictive type equipment has half-value angles of 30° to 60°, while the other equipment is omnidirectional. The resolution to distinguish objects next to each other can be enhanced by using transmitters and receivers with a high directivity. The density of the reflection points is affected by the intervals of oscillation and the speeds of the exploration vessels. The horizontal resolution can be enhanced by decreasing the intervals of the reflecting points in a manner that slows the vessel speeds and shortens the oscillation intervals. The oscillation intervals depend on the time to charge energy and the types as well as the structures of the acoustic wave sources.

(3) Acoustic exploration equipment

When acoustic waves propagate through water or in ground below the seafloor, the acoustic waves from sources with high frequencies have high resolutions but undergo strong attenuation, or the acoustic waves from sources with low frequencies have low resolutions but undergo weak attenuation. The acoustic exploration equipment includes, in a descending order of the frequency of the acoustic wave sources, the electro-strictive type (transducers), magneto-strictive type (sonoprobes), electromagnetic guidance type (boomers), underwater discharge type (sparkers), and compressed air type (air and water guns). In single channel acoustic exploration, all the equipment mentioned above is used, but in multichannel acoustic exploration, water and air guns are used except in certain coastal areas. Furthermore, in the case of multichannel acoustic exploration implemented on a large scale, for example, in resource surveys, spike-like waveforms are generated by synchronizing the oscillation of air guns using the chained array air gun system (combining a large number of air guns).

① **Electro-strictive vibration type acoustic exploration equipment**

Electro-strictive vibrators are high in the conversion efficiency of electric energy to acoustic energy and can be used as both transmitters and receivers because of their ability to convert external pressure to electric energy. They are also used as transmitters and receivers for echo sounders or fishfinders with a frequency range from a few dozen kHz to a few hundred kHz. Because echo sounding uses low frequencies in the range of 1 to 10 kHz, electro-strictive vibrators need to have a large output capacity. Therefore, many types of electro-strictive vibration type acoustic exploration equipment have multiple electro-strictive vibrators in a parallel configuration.

(a) Pulse method

Equipment using the pulse method transmits pulse waveforms, with transducers attached to the bottoms of the exploration vessels, which can obtain geological data of superficial layers a few dozen meters below the seafloor of mid-to-deep water.

(b) Chirp method

Equipment using the chirp method transmits waves with frequencies that continuously change over time (chirp signals) and is capable of the cross-correlation processing of transmitted and received waveforms, and, therefore, can explore deeper ground than the pulse method. This method can obtain geological data of superficial layers up to 100 m below the seafloor of shallow-to-deep water.

(c) Parametric method

When acoustic waves with two different frequencies (primary waves) from a transducer are oscillated in an identical direction, the sum frequency (secondary high frequency waves) and difference frequency (secondary low frequency waves) are created (e.g., the primary frequencies of 21 kHz and 18 kHz create secondary high and low frequencies of 39 kHz and 3 kHz, respectively). The sum frequency is high and shows strong attenuation, but the difference frequency created by a nonlinear cross-interaction is low and shows weak attenuation, thereby enhancing the resolution with the amplitude of acoustic waves intensified as they propagate. Depending on the model specifications, equipment using the parametric method can obtain the geological data of superficial layers a few dozen to 150 meters below the seafloor of shallow-to-deep water.

Fig. 3.8.14 shows an example of a record obtained through the parametric method together with an example of a record obtained through the electromagnetic guidance type acoustic exploration equipment introduced in ② below for comparison. In both examples, the vertical axes represent the depths of the reflecting surfaces.

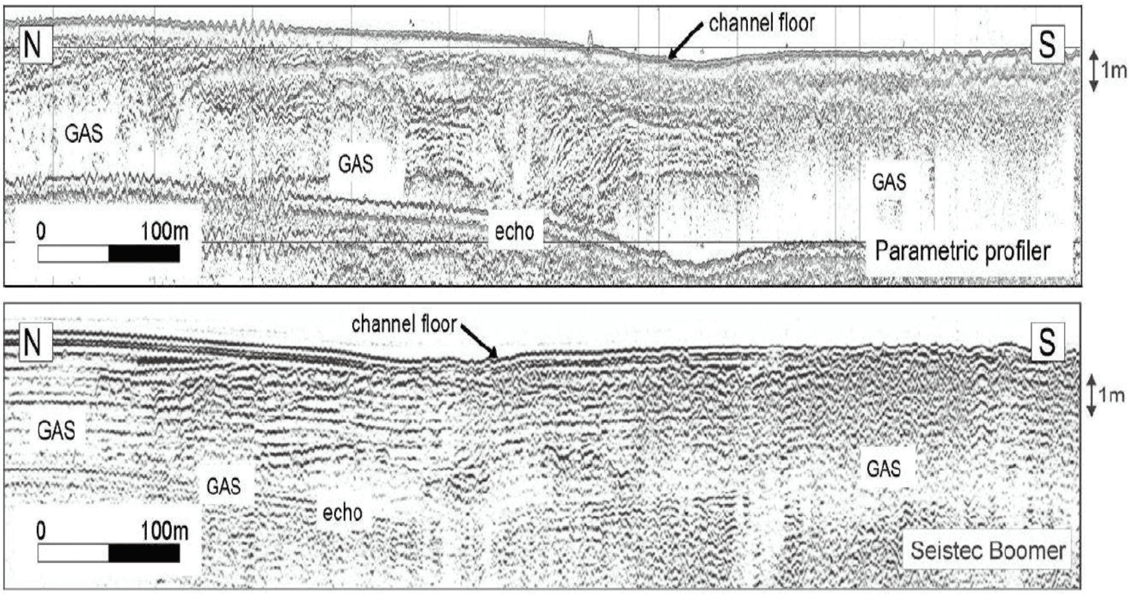


Fig. 3.8.14 Examples of Acoustic Exploration Records
(Top: Electro-strictive vibration type acoustic exploration using the parametric method;
Bottom: Electromagnetic guidance type acoustic exploration equipment)

② **Electromagnetic guidance type acoustic exploration equipment**

When a magnetic field created with high current instantaneously is applied to a submerged coil and an aluminum plate is installed close to the magnetic field, it generates eddy current. Using this mechanism, the equipment generates acoustic waves with the aluminum plate rapidly oscillated by a repulsion force acting between the eddy current and the magnetic field. The electromagnetic repulsion force is proportional to the square of the current, and the predominant frequencies of the generated acoustic waves are a secondary high frequency which is twice as high as that of fundamental waves of the current and others higher than the secondary high frequency. A capacitor is charged with the power supplied from a generator after increasing voltage with a transformer. Then, the high voltage current is instantaneously applied to a submerged coil attached to the lower section of a catamaran-like towed device by activating three point caps, an ignitron or a thyristor so as to oscillate the acoustic waves with an insulated metal plate oscillated by a repulsion force, and to receive reflected acoustic waves with hydrophones used as receivers. Generally, the equipment has a transmission energy of 200 to 300 J and a transmission frequency in the range of 400 Hz to 14 kHz. In addition, the equipment is capable of oscillating acoustic waves close to single pulse waveforms, thereby obtaining high resolution records. Unlike the underwater discharge type, because the electromagnetic guidance type can oscillate acoustic waves in freshwater, as is the case in seawater, it has been used in the exploration of rivers and lakes. **Fig. 3.8.15** shows an outline of electromagnetic guidance type acoustic exploration.

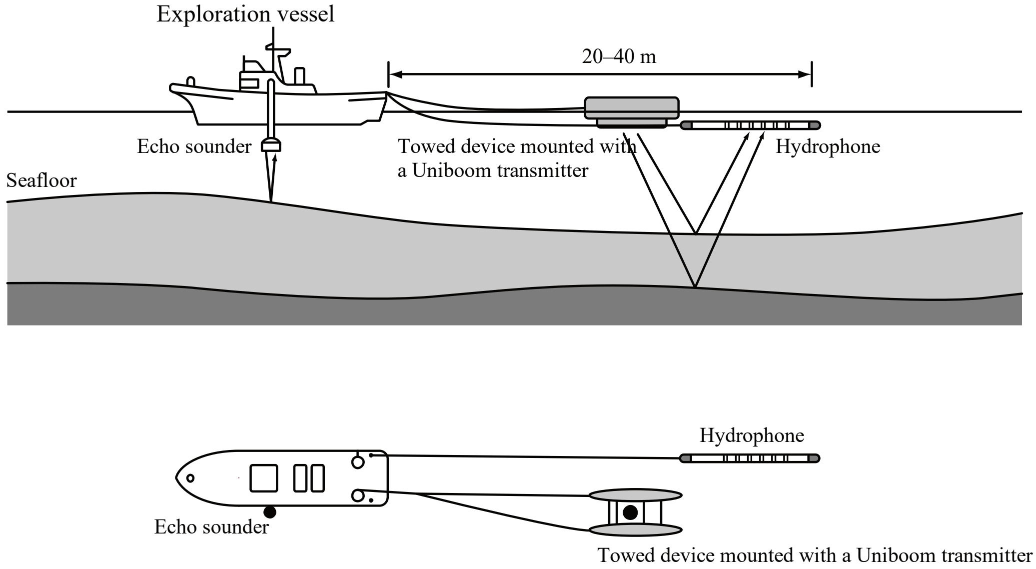


Fig. 3.8.15 Outline of Electromagnetic Guidance Type Acoustic Exploration

③ **Underwater discharge type acoustic exploration equipment**

Underwater discharge type acoustic exploration equipment is generally called a sparker. Similar to the electromagnetic guidance type, the equipment charges a high voltage capacitor and instantaneously discharges high voltage current through underwater discharge electrodes. The high current between the electrodes rapidly increases the temperature of the seawater and produces bubbles, and acoustic waves are generated when the bubbles expand and burst. The equipment can increase the exploration depth and enhance the resolution with an increase in acoustic pressure and a decrease in pulse widths in the oscillation waveforms. The configuration of the equipment is similar to the electromagnetic type as shown in **Fig. 3.8.15**, and the equipment uses underwater electrodes in place of a towed device mounted with a Uniboom oscillator.

④ **Acoustic exploration with a compressed air type acoustic wave source**

Acoustic wave sources activated by compressed air include water guns, air guns and GI guns. An air gun synchronized array which combines multiple air guns and controls the oscillation waveforms is used in surveys such as oil explorations that explore deep layers below the seafloor. Air is compressed by a compressor to about 15 MPa while being supplied to a chamber in the gun through a manifold. By opening a solenoid valve on the chamber using electric signals, air, in the case of an air gun, or water, in the case of a water gun, is rapidly discharged to generate acoustic waves. An air gun is a typical acoustic wave source used in ocean bottom oil exploration and ground surveys, and it oscillates acoustic waves with low frequencies in the range of a few Hz to 100 Hz.

(4) Multichannel acoustic exploration

Multichannel acoustic exploration has been developed as a technology for large scale explorations of geological structures, such as oil and natural gas explorations, and has been applied widely from active fault surveys in coastal areas to geological structure surveys of deep sea bottoms for the demarcation of continental shelves. After the Great

Hanshin Earthquake, the multichannel acoustic exploration method was used for the active fault surveys in Tokyo, Ise and Osaka Bays.

In contrast to single channel acoustic exploration, which receives reflected waves from the seafloor and the layers below the seafloor with one receiver, multichannel acoustic exploration generally receives reflected waves with multiple receivers arranged on a receiving cable (streamer cable). One of the types used in open ocean has a 6,000 m streamer cable and 480 channels (with receivers arranged at 12.5 m intervals). However, it is difficult to tow conventional streamer cables with lengths of a few hundred meters in water areas with busy ship navigation and in coastal areas. Thus, a compact type using electromagnetic acoustic wave sources and short streamer cables with a length of 15 m and 12 channels (with receivers arranged at 1.25 m intervals) has been developed so that it can be towed by small vessels. An outline of multichannel acoustic exploration is shown in **Fig. 3.8.16**.

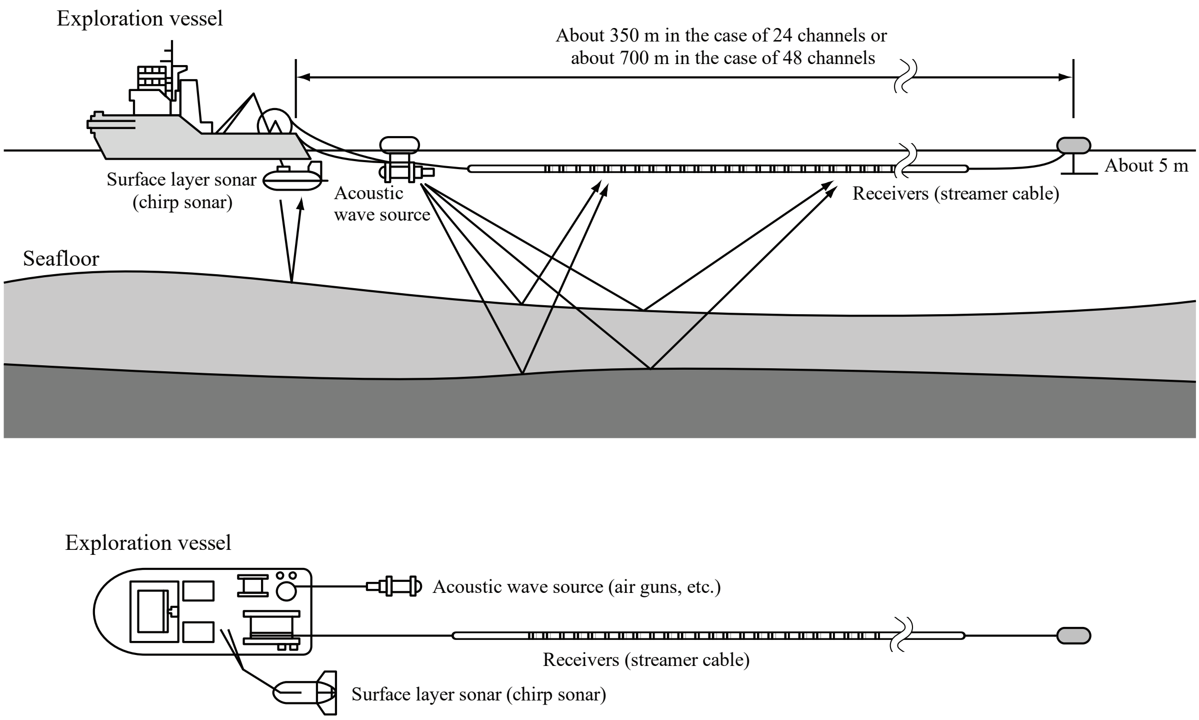


Fig. 3.8.16 Outline of Multichannel Acoustic Exploration

(5) Analyses of measured records

① **Analyses of acoustic exploration records**

Acoustic exploration records enable cross-sectional information of depositional and geological structures below the seafloor to be visually examined, but they also contain information other than geological data, such as multiple reflection and refracted waves associated with the propagation of acoustic waves. The horizontal axes in the acoustic exploration records represent the distances corresponding to the number of oscillations, and the vertical axes represent durations from when the acoustic waves are oscillated until when the reflected waves are received. Thus, the acoustic exploration records show time cross sections. In the case of single channel acoustic exploration where propagation velocities inside the layers are unobtainable, there are cases of establishing depth sections by replacing these velocities with the underwater propagation velocity (1,500 m/s). Using the underwater propagation velocity, which is generally slower than the velocity in layers, causes the depth sections to show shallower depths than in actual cases. Thus, as is the case for designing structure foundations with a focus on the accuracy of the depth records, it is necessary to appropriately set the propagation velocities of the acoustic waves in layers based on the boring histograms and P wave logging results.

② **Organization of record analysis results**

Layer classifications shall be analyzed while identifying the intersections of the measuring lines and plotting the distribution ranges on track charts with due consideration to their relationship with the seafloor topography and faults. Then, the analysis results of the layer classification shall be organized in terms of the characteristics of the reflection patterns, the degrees of inclination, the relationship between layers, and the distribution ranges as well as depths. Based on these results, the order of stratification shall be determined in combination with the geological structures of land areas and boring survey reports.

Regarding faults, folds and flexures, their continuity and the extent of their influences on the surrounding layers shall be analyzed by plotting their positions on track charts. Based on these analysis results, the cross sections of marine geology, sounding maps of foundation layers and marine geological structure maps shall be established. There may be cases where it is necessary to establish isopachous maps that show the thicknesses of specific layers.

③ **Sound-scattering layers**

According to **the terminological dictionary of marine acoustics**[14)], sound-scattering layers are defined as those layers which scatter low frequency acoustic waves. Here, low frequency acoustic waves mean those used for acoustic exploration and with frequencies lower than those used for echo sounders. There may be cases where acoustic exploration fails to obtain geological information below the seafloor in some areas. Some sound-scattering layers scatter acoustic waves immediately below the seafloor and others scatter acoustic waves at certain depths below the seafloor.

Sound-scattering layers are located widely in inner bay areas, such as in the inner part of Tokyo Bay, in Ishinomaki Bay, in the Ariake Sea off the coast of Kumamoto Prefecture, and in the northern part of the Yatsushiro Sea. In open ocean, a sound-scattering layer is widely distributed from an area off the coast of Niigata Prefecture to an area south of Awa Island. In freshwater areas, sound-scattering layers can be found in the Sumida and Ara Rivers and in shallow lakes such as Lake Kasumigaura, Lake Suwa, Lake Kisaki, the inner part of Lake Hamana, Lake Shinji and the southern part of Lake Biwa. In contrast, normal data without the influence of sound-scattering layers is available in deep lakes such as Lake Toya, Biwa, Aoki and Nojiri.

Sound-scattering layers can be found in sedimentation valleys and shallow lakes with muddy sediment. It is estimated that gas which is produced through the decomposition of organic substances in sedimentation layers and trapped in cohesive layers fully reflects the acoustic waves or absorbs the energy of the acoustic waves at the depths of the cohesive soil layers. In boring surveys conducted in water areas with sound-scattering layers during the development of the Tokyo Bay Aqua-Line and the Kansai International Airport, no phase changes were identified at the depths where acoustic waves were scattered, but there were distributions of humus below the sound-scattering layers. Thus, gas was considered to be the cause of the scattering of acoustic waves.

3.8.7 Hazardous Material Exploration

Many of the mines laid during World War II have been left undetected in the ports and coasts of Japan. In addition, a large number of unexploded bombs and shells which were dropped and fired in air raids and naval bombardments have been buried in areas near revetments and river mouths deposits. Hereinafter, these mines, bombs and shells are referred to as mines and bombs. In the water areas which have already been cleared out by the Ministry of Defense, there is no danger that ship navigation could result in exploding unrecovered mines or bombs. However, there is a possible danger of severe damage due to exploding mines or bombs when dredging or revetment construction work results in accidental collisions with unexploded ordnance. Hazardous marine exploration is a type of exploration that is conducted to remove mines and bombs to ensure the safe implementation of port construction works.

An outline of the mines and bombs is as follows.

Mine: An explosive which is installed close to the seafloor and explodes to destroy or submerge ships when they navigate nearby.

Bomb: An explosive which is dropped from a bombing plane or a combat plane on a target object and explodes on impact to destroy them.

Shell: An explosive which is shot out of a cannon (on a battleship) for the same purpose as a bomb.

Abandoned bomb: A bomb or shell abandoned by the former Imperial Japanese Army at the end of the war.

Fire bomb: An explosive filled with flammable material such as oil for setting fires or destroying a target by fire.

In principle, detected mines and bombs are removed and disposed by the Japanese Self-Defense Forces.

(1) Outline of Hazardous Material Exploration

Because the shells of mines and bombs used in World War II were made of iron and were magnetic, magnetic exploration is the most effective way to detect them. The locations of buried objects made of iron (hereinafter referred to as “magnetically irregular points”) can be identified through magnetic exploration; however, it cannot determine whether or not the buried objects are mines or bombs. In most cases, magnetically irregular points have been caused by ironware other than mines and bombs.

① Types and methods of hazardous material exploration

Hazardous material exploration is basically implemented in two stages: first, to detect the magnetically irregular points, then to excavate them to identify the buried objects (or submersible exploration in the case of water areas).

Magnetic exploration is largely classified into horizontal magnetic exploration and vertical magnetic exploration.

Horizontal magnetic exploration is implemented for planar measurements along seafloor and ground surfaces, and for detecting objects at certain depths below these surfaces. The measuring methods, required measuring instrument and the depths to be explored differ between water areas (including rivers and lakes) and land areas.

Vertical magnetic exploration uses boreholes at survey points to detect objects down to the targeted depths.

The types of hazardous material exploration are summarized in the terms of exploration areas and directions in **Table 3.8.4**, and the applicability of the respective types of magnetic measurement is shown in **Table 3.8.5**.

The appropriate types of hazardous material exploration shall be selected while taking into consideration the exploration purposes (the scope of exploration related to security), surrounding environments and noise sources such as the existing structures.

Table 3.8.4 Types of Hazardous Material Exploration

<table>
<tr><th rowspan="2"></th><th rowspan="2">Exploration direction</th><th colspan="2">Area and type of magnetic exploration</th></tr>
<tr><th>Water area</th><th>Land area</th></tr>
<tr><td rowspan="5">Magnetic exploration</td><td rowspan="4">Horizontal</td><td>Offshore horizontal magnetic exploration</td><td rowspan="3">Onshore horizontal magnetic exploration and excavation of magnetically irregular points</td></tr>
<tr><td>Submersible exploration</td></tr>
<tr><td>Confirmation exploration</td></tr>
<tr><td>Combined horizontal-excavation exploration</td><td>Combined horizontal-excavation exploration</td></tr>
<tr><td>Vertical</td><td>Vertical magnetic exploration</td><td>Vertical magnetic exploration</td></tr>
<tr><td>Other</td><td>Horizontal</td><td>Diver exploration</td><td>Exploration with metal detectors</td></tr>
</table>

Table 3.8.5 Applicability of the Respective Types of Magnetic Exploration

Type	Applicability
Offshore horizontal magnetic exploration	When it is necessary to explore wide water areas for dredging works
Onshore horizontal magnetic exploration	When it is necessary to explore shallow ground for civil engineering and building works
Combined horizontal-excavation exploration	Secondary offshore or onshore horizontal magnetic exploration to be implemented to supplement primary exploration in a manner that increases the exploration depths by excavating the seafloor or ground surface to the depths for which safety is confirmed through primary magnetic exploration. Vertical magnetic exploration can be used as an alternative when the local conditions do not allow combined horizontal-excavation exploration to be implemented.

Type	Applicability
Vertical magnetic exploration	When it is necessary to explore at great depths to confirm the safety of the areas for pile driving works and geological boring surveys (for both offshore and onshore areas). Because the effective area in single magnetic exploration is limited, the exploration periods and costs are increased in the case of extensive exploration. Thus, the applicability of the combined horizontal-excavation exploration shall be examined while taking into consideration the required depths and subsequent construction works.

② **Offshore exploration**

Offshore magnetic exploration is basically horizontal magnetic exploration using vessels.

Actual excavation work is required to confirm whether or not the magnetically irregular points detected through magnetic exploration are mines or bombs. Submersible exploration is underwater excavation work executed by divers to confirm the presence or absence of mines or bombs at magnetically irregular points below the seafloor. Relatively wide exploration areas are set for submersible exploration, because, in the case of offshore magnetic exploration, the positional accuracy of the magnetically irregular points is lower than that of onshore magnetic exploration.

A confirmation exploration (in the form of horizontal magnetic exploration) is implemented in the areas subjected to submersible exploration for the purpose of confirming that the magnetically irregular points have been eliminated, with all buried iron objects identified and removed from under the sea after the submersible exploration.

It shall be noted that there are two types of offshore areas: areas where safety can be secured by single magnetic exploration, and areas where the safety needs to be confirmed by repeatedly implementing magnetic exploration every time before any construction work, because of the possibility that rivers and tides will allow mines and bombs to flow into the area.

Diver exploration (or simplified exploration) is underwater work conducted by divers at locations where there is difficulty in implementing magnetic exploration in a manner that allows the divers to detect objects with simplified magnetometers or brass tamping rods and to excavate the objects in order to identify them.

An outline of offshore magnetic exploration is summarized in **Table 3.8.6**.

Table 3.8.6 Outline of Offshore Magnetic Exploration

Type	Outline	Remarks
Offshore horizontal magnetic exploration	Detection of magnetically irregular points corresponding to the exploration objects (mines and bombs) through area-wide magnetic measurements	Diver exploration is implemented in very shallow coastal areas where there are difficulties in carrying out measurements using vessels
Submersible exploration	Excavation and confirmation by divers of magnetically irregular points corresponding to the exploration objects detected through area-wide magnetic exploration	Excavations shall be conducted with care while monitoring the magnetic reactions using magnetometers. In general, the readings from the magnetometers are not recorded.
Confirmation exploration	Magnetic exploration implemented to confirm that the magnetically irregular points corresponding to the exploration objects have been eliminated after submersible exploration	Offshore horizontal exploration implemented for the exploration areas of submersible exploration
Diver exploration	Exploration of the entire area, which cannot be explored through magnetic exploration, by divers using magnetometers or brass tamping rods	Substitution of the magnetic exploration implemented, in most cases, together with excavation confirmation. It is rare to plan diver explorations independently. In general, the results from the magnetometers are not saved.

③ **Onshore and offshore exploration close to revetments**

In many cases of onshore and offshore magnetic exploration close to revetments, the exploration objects are not mines but bombs and shells. Depending on the purposes, horizontal magnetic exploration and vertical magnetic exploration are implemented for exploration places close to ground surfaces and at relatively large depths, respectively.

In these explorations, the magnetically irregular points can be detected with a positional accuracy of ±1 m, but bombs dropped from high altitudes may have been buried at deep places. Thus, depending on the buried depths and the groundwater levels, countermeasures such as enclosing the excavation areas with steel sheet piles and constructing vertical shafts may be required when excavating ground to identify the magnetically irregular points.

There may be cases where an exploration using metal detectors can be effectively implemented in place of magnetic exploration when it is determined that bombs and shells are too small to be magnetically detected or the magnetic exploration is not effectively applicable to local situations. Generally, in an exploration using metal detectors, the data obtained through the metal detectors is not saved as is the case with offshore diver exploration.

(2) Preparation Plan

Unlike other geophysical explorations, preparation plans shall be established for hazardous material exploration on the basis of its unique purpose to detect objects such as mines and bombs.

① **Magnetic quantity of mines and bombs**

It has been known that specific types and sizes of mines and bombs have magnetic quantities in predetermined ranges. For example, mines and 250 kg bombs have a magnetic quantity of 17.5 μWb or more and 7.0 μWb or more, respectively. Thus, when the exploration objects are mines, a magnetic exploration with a capacity for detecting all the magnetically irregular points of 17.5 μWb or more is satisfactory. In reality, however, the detectable magnetic quantities are set as shown below to be on the safe side.

For mines: Detectable magnetic quantity of 7.0 μWb or more for analyses and reporting
Detectable magnetic quantity of 17.5 μWb or more for confirming magnetically irregular points through submersible exploration

For 250 kg bombs: Detectable magnetic quantity of 3.5 μWb or more for analyses and reporting
Detectable magnetic quantity of 7.0 μWb or more for confirming magnetically irregular points through submersible exploration

In the case of magnetic exploration with the exploration objects clearly specified, there is no need to change the measuring methods at the sites, but the analytical workloads will be slightly increased. For example, in magnetic exploration only for mines, there is no purpose in analyzing and reporting magnetically irregular points with small magnetic quantities of 0.7 μWb. Thus, the thresholds of the magnetic quantities shall be appropriately set by evaluating the exploration objects and local situations in a comprehensive manner. When implementing submersible exploration, it is also necessary to clarify the exploration objects and appropriately set the threshold magnetic quantities corresponding to the exploration objects.

Table 3.8.7 shows the sizes and magnetic quantities of major mines and bombs.

Table 3.8.7 Sizes and Magnetic Quantities of Mines and Bombs

Mines and bombs	Standard size		Magnetic quantity (μWb)
	Diameter (mm)	Length (mm)	
Mine (Type I and II)	570 to 580	2050 to 2090	17.5 or more
Mine (Type III)	470	1710	
1t bomb	480 to 610	1800	14.0 or more
250 kg bomb	300 to 380	1200	7.0 or more

② **Exploration objects and depths**

The depths explored through magnetic exploration can be obtained from the SN ratios of the capacity of the magnetic probes and magnetic recording. Here, S represents the signals and is dependent on the magnetic quantity of the exploration objects. N represents the noise and is dependent on the performance of the instrument, the magnetism of the ground and miscellaneous buried iron. Thus, the noise levels vary with the region and location.

In the case of offshore horizontal magnetic exploration implemented under favorable conditions, the effective exploration depths are about 4.0 m and 2.5 m below the seafloor for mines and 250 kg bombs, respectively. In other words, offshore horizontal magnetic exploration is effective for exploring magnetically irregular points with a magnetic quantity of 17.5 μWb or more and 7.0 μWb or more at the depths up to 4.0 m and 2.5 m below the seafloor, respectively.

When implementing magnetic exploration with mines and 250 kg bombs as exploration objects, measurements can be done at one time because the instrument and methods to be used are generally common. In this case, it is necessary to clearly set the exploration depths with due consideration to the differences in the effective exploration depths between mines (4.0 m) and 250 kg bombs (2.5 m).

Onshore horizontal magnetic exploration is subjected to larger measurements and ground noise than offshore exploration. Thus, the available exploration depths of the onshore horizontal magnetic exploration are generally shallower.

Vertical magnetic exploration is implemented inside exploration holes (boreholes) and the available exploration depths are determined by the borehole depths. The exploration ranges in horizontal direction of vertical magnetic exploration should be set in a circle from the center of the boreholes with a certain radius.

③ **Ranges of submersible exploration**

In offshore horizontal magnetic exploration, the accuracy of the measured positions of the magnetically irregular points is not very high because the measurements are made with sensor frames suspended from moving exploration vessels. Thus, it is necessary to set wide exploration areas for submersible exploration with the coordinates of the magnetically irregular points as the centers. In particular, the exploration areas are, in many cases, either:

• Circles having a radius of 15 m with magnetically irregular points as the centers; or

• Squares having a side of 30 m with magnetically irregular points as the centers.

④ **Timings of implementing the magnetic and submersible explorations**

There is a possibility that mines and bombs can flow into water areas with rushing rivers or strong currents. Thus, magnetic exploration or diver exploration in water areas where there is a possible inflow of mines and bombs shall be implemented immediately before the commencement of any planned construction works.

Submersible exploration shall be implemented immediately after magnetic exploration in order to avoid a possible displacement of the existing magnetically irregular points or a migration of new ones as a result of being not only swept up by rivers or currents but also dragged by fishing nets or anchors after being dumped.

⑤ **Influences of existing structures**

Magnetic exploration is largely affected by existing structures (buildings, steel sheet piles, bridge piers and H-section steel) made of iron or containing iron. Therefore, magnetic exploration for the detection of mines and bombs cannot be implemented in areas close to these structures. The extent of the influences of these structures on magnetic exploration is approximately several to ten meters depending on the sizes of the structures. In the magnetic exploration results, these areas that are influenced by existing structures are reported as abnormally dense zones or unexplorable zones.

In principle, diver exploration is implemented in areas to which magnetic exploration is not applicable due to strong influences from existing structures.

⑥ **Exploration periods**

Offshore horizontal magnetic exploration and submersible exploration require a large amount of time for planning, preparing measuring instrument, fitting out exploration vessels and taking procedures.

The exploration periods vary depending on the local conditions. Particularly, the execution plans of offshore works shall be established based on the meteorological and hydraulic conditions unique to the exploration areas such as ports and construction works conducted depending on the season, as well as the large differences in wave conditions between water areas inside and outside of ports.

In the case of vertical borehole magnetic exploration, the types of local ground largely affect the time required for drilling boreholes.

In addition, because of the necessity to organize not only the magnetic records but also the ship positions and echo sounding data, the analysis work of offshore horizontal magnetic exploration takes more time than that of onshore horizontal magnetic exploration and vertical magnetic exploration.

(3) Exploration Procedure

① Offshore exploration

(a) Overall procedure

The overall procedure for offshore exploration is shown in **Fig. 3.8.17**.

In principle, in offshore exploration, additional magnetic exploration is implemented at the places around the magnetically irregular points after the submersible exploration so as to confirm that the magnetically irregular points that have a possibility of being mines or bombs have been reliably eliminated. The magnetic exploration implemented for this purpose is called confirmation exploration. When reminds of the magnetically irregular points are detected through confirmation exploration, the cycle of submersible and confirmation exploration needs to be repeated until all the magnetically irregular points are eliminated.

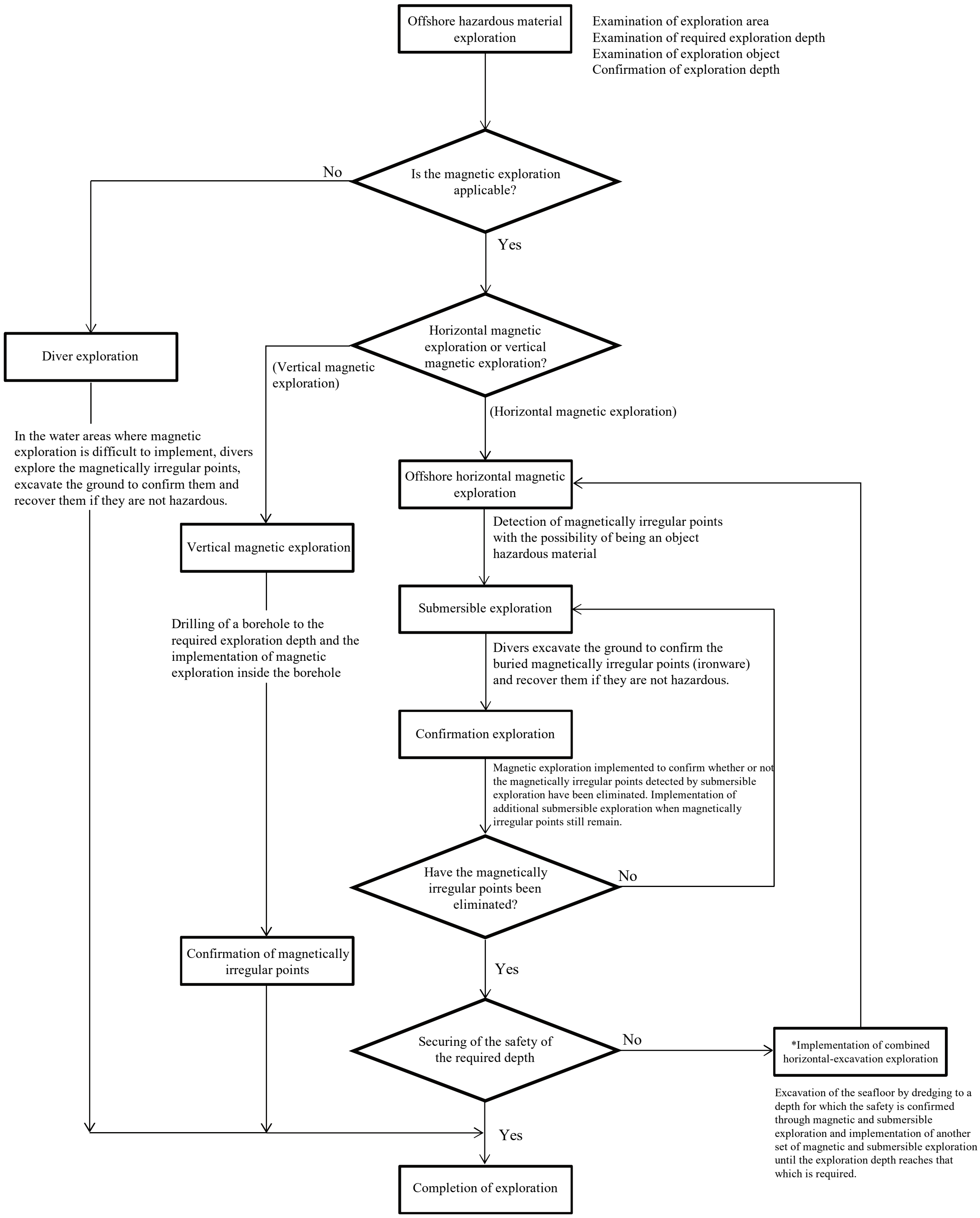


Fig. 3.8.17 Overall Procedure for Offshore Hazardous Material Exploration

(b) Procedure for offshore horizontal magnetic exploration

The procedure for offshore horizontal magnetic exploration is shown in **Fig. 3.8.18**. In addition, the implementation situations of offshore horizontal magnetic exploration are shown in **Figs. 3.8.19** to **3.8.21**.

Offshore horizontal magnetic exploration

Planning and preparation

- Field survey
- Planning
- Procedures and explanation meetings with concerned parties
- Assembling of a sensor frame
- Fitting out of exploration vessels
- Checking of equipment, etc.

↓

Field measurement

- Magnetic measurement
- Vessel position surveying
- Measurement of the sensor frame height
- Confirmation of the presence or absence of unexplored areas

↓

Analysis and discussion

- Creation of track charts
- Analysis of magnetic records
- Readout of echo sounding records
- Analysis of magnetically irregular points (Magnetic quantities, positions, buried depths, etc.)
- Discussion

↓

Creation of deliverables

- Exploration result
    - Position map of magnetically irregular points
    - List of magnetically irregular points
    - Track charts
- Exploration method
- Reference materials and other documents

Fig. 3.8.18 Procedure for Offshore Horizontal Magnetic Exploration

Fig. 3.8.19 Sensor Frame Suspended from a Steel Barge (Example Using Nine Sensors)

Fig. 3.8.20 Measuring State (Small Boat Towing Method)

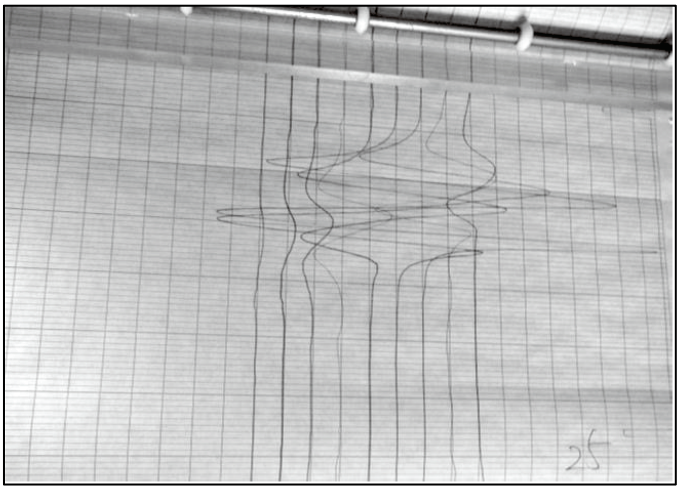


Fig. 3.8.21 Example of Magnetic Recording

(c) Procedure of submersible and diver exploration

Submersible exploration is generally defined as an exploration conducted by divers in a manner that excavates the seafloor to identify the magnetically irregular points detected through the preceding offshore horizontal magnetic exploration. The procedure for submersible exploration is shown in **Fig. 3.8.22**.

In contrast, diver exploration (simplified exploration) is an exploration conducted by divers in a manner that explores magnetically irregular points and excavates the seafloor to confirm the points in water areas that are too narrow or too shallow for magnetic exploration to be implemented, or in water areas with hard seafloor where mines are considered less likely to be buried. The procedure for diver exploration is shown in Fig. **3.8.23**.

Submersible exploration

Planning and preparation

- Field survey
- Planning
- Procedures and explanation meetings with concerned parties
- Preparation of diver boat
- Checking of equipment, etc

↓

Confirmation of excavation

- Marking of the positions of the magnetically irregular points
- Confirmation of excavation by divers
- Recovery of confirmed abnormal materials (not hazardous)
- Confirmation of the presence or absence of any remaining magnetically irregular points

↓

Discussion

- Verification of recovered abnormal materials
  - Whether or not the recovered abnormal materials correspond to the detected magnetically irregular points

↓

Creation of deliverables

- Result
  - Position map
  - List of confirmed abnormal materials
  - Photographs of abnormal materials
- Exploration method
- Reference materials and other documents

Fig. 3.8.22 Procedure for Submersible Exploration

Diver exploration

Planning and preparation

- Field survey
- Planning
- Procedures and explanation meetings with concerned parties
- Preparation of diver boat
- Checking of equipment, etc

↓

Confirmation of exploration and excavation

- Clarification and marking of exploration area
- Exploration by divers
- Excavation and confirmation of detection points
- Recovery of abnormal materials (not hazardous materials)
- Confirmation of the presence or absence of any remaining magnetically irregular points

↓

Creation of deliverables

- Result
  - Position map
  - List of confirmed abnormal materials
  - Photographs of abnormal materials
- Exploration method
- Reference materials and other documents

Fig. 3.8.23 Procedure for Diver Exploration

② **Onshore exploration**

The procedure for onshore exploration is shown in **Fig. 3.8.24**.

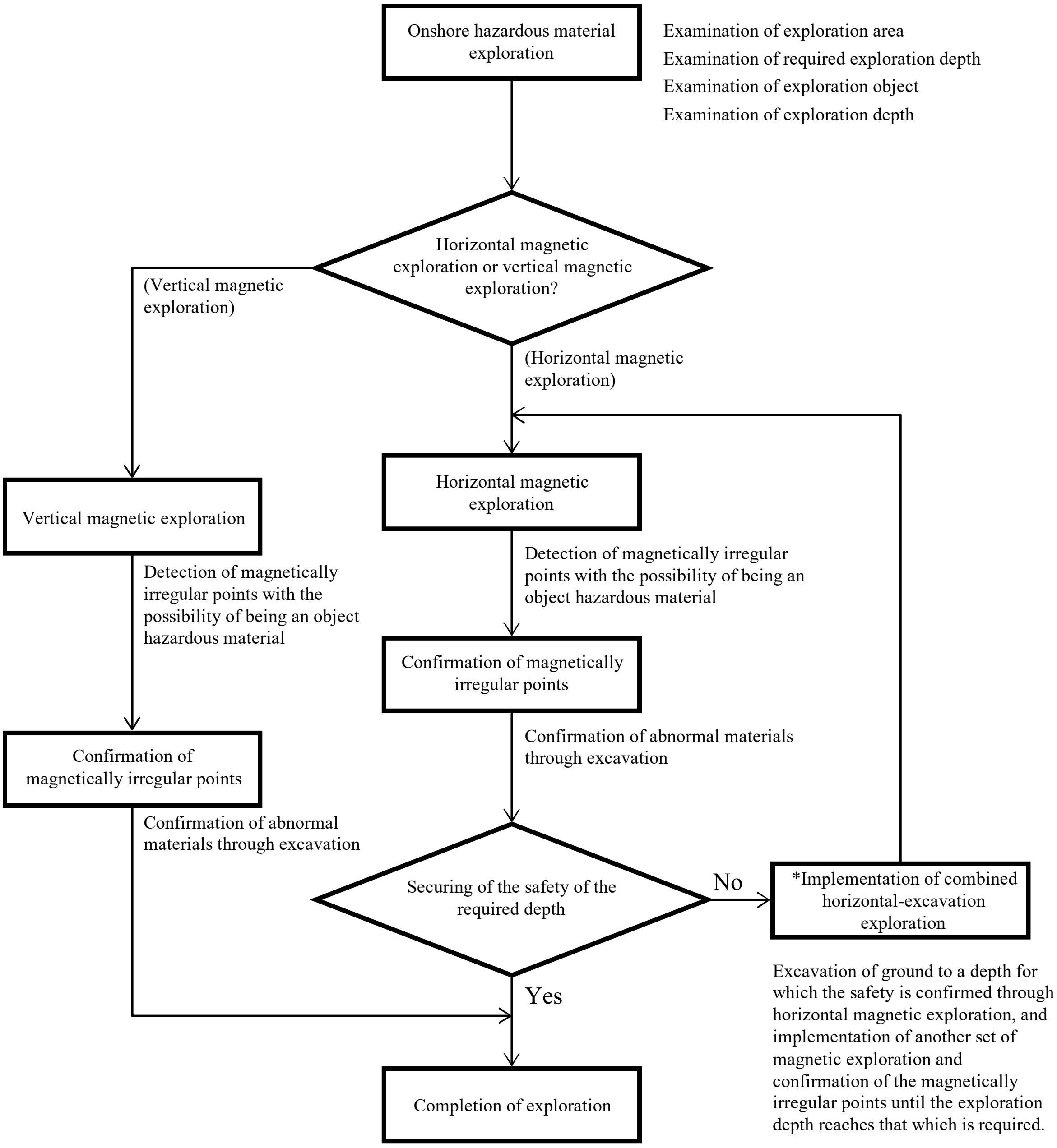


Fig. 3.8.24 Procedure for Onshore Hazardous Material Exploration

The procedure for onshore magnetic exploration and excavation as well as confirmation of the magnetically irregular points is shown in **Fig. 3.8.25**. In addition, the implementation situations are shown in **Figs. 3.8.26** and **3.8.27**.

Onshore exploration

Planning and preparation

- Field survey
- Planning
- Procedures and explanation meetings with concerned parties
- Checking of equipment, etc.

↓

Field measurement

- Positioning of exploration areas and points
- Weeding (if necessary)
- Magnetic measurement
  - Horizontal or vertical exploration

↓

Analysis and discussion

- Analysis of magnetic records
- Analysis of magnetically irregular points
  - (Magnetic quantities, positions, buried depths, etc.)
- Discussion

↓

Confirmation of excavation

- Clarification of magnetically irregular points
- Excavation and confirmation of abnormal materials
- Removal of abnormal materials
  - (Not hazardous materials)
- Confirmation of the presence or absence of any remaining magnetically irregular points

↓

Creation of deliverables

- Result
  - Position map of magnetically irregular points
  - List of magnetically irregular points
  - List of confirmed abnormal materials
  - Photographs of abnormal materials
- Exploration method
- Reference materials and other documents

Fig. 3.8.25 Procedure for Onshore Work

Fig. 3.8.26 State of Onshore Horizontal Magnetic Exploration

Fig. 3.8.27 State of Vertical Borehole Magnetic Exploration

(4) Measuring Instrument

① Magnetic probes for magnetic exploration

There are several types of magnetic probes, which are basically used to measure the magnetization parameters (magnetic flux density) at the measuring points, but not to measure the responses (reflected waves or electromagnetic responses) of the exploration objects with signals emitted from the probes.

The following three types of magnetic probes have been frequently used for hazardous material exploration.

(a) Dual coil type magnetic gradiometer

This instrument measures variations of the unidirectional component of magnetic flux density. It detects signals only when a sensor section moves.

(b) Differential type fluxgate magnetometer (uniaxial or triaxial)

This instrument measures the magnetic flux density values at measuring points. A widely used model is the uniaxial probe that measures the unidirectional component.

There are cases of using the triaxial model which measures the total magnetic parameters by simultaneously measuring three directional components.

(c) Optical pumping magnetometer (cesium-vapor magnetometer, etc.)

This instrument measures the magnetic flux density values (total magnetic intensity) at measuring points but cannot define the direction of the components.

There are no major differences in the performance of these types of magnetic measurements instrument as long as they are used for the magnetic exploration of mines and bombs.

Magnetic exploration explores iron materials in a manner that detects minute magnetic fields formed around the iron materials while moving sensors. In the cases of magnetic gradiometers and fluxgate magnetic probes which measure directional components, the geomagnetic field of the earth can be a major noise source. Thus, these types of instrument detect signals of buried iron materials while eliminating the geomagnetic field signal by simultaneously receiving signals with two magnetic detection sensors in a single probe having identical performance arranged at certain intervals and obtaining the differences. These magnetic probes are called the dual-coil type or differential type instrument.

In the case of optical pumping magnetic probes represented by the cesium magnetometer, because they measure the total magnetic intensity, their measurements are not affected by noise from the geomagnetic field of the earth associated with the movement of the magnetic sensors. However, the measurements need to be corrected for the geomagnetic field, which permanently fluctuates, by making stationary measurements or using differential type sensors.

The proton magnetic probe, which is also one of the typical magnetic probes and generates a strong magnetic field around the sensor during measurements, cannot be used for hazardous material prospecting because the magnet field may detonate mines.

Table 3.8.8 shows the standard exploration performance required for the magnetic probes to be used in hazardous material exploration.

Table 3.8.8 Standard Exploration Performance of Magnetic Probes

Exploration method	Exploration object	Standard exploration performance
Offshore horizontal magnetic exploration	Mine	Ability to detect mines at depths up to 4.0 m below the seafloor
	250 kg bomb	Ability to detect 250 kg bombs at depths up to 2.5 m below the seafloor
Onshore horizontal magnetic exploration	250 kg bomb	Ability to detect 250 kg bombs at depths up to 1.5 m below the ground surface
Vertical magnetic exploration	250 kg bomb	Ability to detect 250 kg bombs at distances up to 1.5 m from the center of the exploration boreholes

(Note) The exploration performance in the table above is standard and may differ depending on local noise levels.

② **Magnetic probes for submersible and diver exploration**

The devices that divers use in submersible and diver exploration are simplified magnetic probes called magnetometers in many cases. The principle of the magnetometers is the same as that of general magnetic probes; however, while the magnetic probes output measured data in the form of electric signals and record in pen recorders or data loggers, the magnetometers convert the intensity of the output signals to sound so that the divers can determine the positions of the iron materials by listening to the sound.

With most magnetometers, sensors are integrated with the main bodies, thereby eliminating the necessity to extend cables from the sensors up to the exploration vessels. Thus, the improved usability of magnetometers can enhance the safety of underwater exploration work.

③ **Other measuring devices**

(a) Ship position surveying devices

Ship position surveying devices are used while measuring in the case of offshore magnetic exploration and while marking magnetically irregular points in the case of submersible exploration. Recently, the positional accuracy has been largely improved owing to the popularization of GNSS, supported by the improvement in related devices and software.

However, devices other than those mounted with GNSS may be used in the following cases.

- Explorations in areas where electric signals from GNSS satellites are difficult to receive: radio positioning devices
- Explorations in narrow areas close to land: transits

(b) Echo sounders

In offshore horizontal magnetic exploration, echo sounders are used for measuring the heights of sensor frames from the seafloor in a manner that simultaneously measures the depths (water depths) of the sensor frames and the seafloor.

(c) Metal detectors

Metal detectors are used mostly for onshore exploration to which magnetic exploration is not applicable. By detecting the positions of metal responses using electromagnetic waves, metal detectors can be used for detecting not only iron material but all other metallic materials. As is the case with magnetometers, metal detectors convert the intensity of the metal response signals to sound so as to enable operators to determine the positions of buried metallic materials while listening to the sound.

(5) Exploration Methods

① **Offshore horizontal magnetic exploration**

Offshore horizontal magnetic exploration is implemented using exploration vessels in water areas to detect magnetically irregular points that require the excavation of the seafloor in order to be identified.

(a) Preparation

1) Field surveys and reference material collection

The preparations for offshore horizontal magnetic exploration include observations of the conditions of the exploration areas and their surroundings as well as the collection of any necessary reference materials.

Furthermore, it is necessary to confirm at least the following items.

- Meteorological and hydrographic conditions
- Water depths and seafloor topography of the exploration area
- Presence or absence of construction works in and around the exploration area while implementing offshore horizontal magnetic exploration
- Presence or absence of existing structures that may have possible influences on magnetic recording
- Related organizations requiring explanations and consultations
- Scheduled ship navigation such as regular liners

2) Selection of exploration vessels

For the navigation methods, exploration vessels are largely classified into towed vessels and self-propelled vessels. Self-propelled vessels are used in shallow water areas and are made of nonmagnetic materials such as FRP or wood. In the case of towed vessels, different types of vessels are used depending on the water depths of the exploration areas as shown in **Table 3.8.9**.

Table 3.8.9 Classification of Exploration Vessels

Navigation method	Vessel type	Applicable water depth	Characteristics
Towed vessel	Steel barge	10 m or more	Suitable for explorations in deep water areas where the influence of steel barges on the measurement records can be eliminated. Not suitable for narrow areas due to low maneuvering performance but relatively resistant to waves.
	Non-magnetic barge	5 to 20 m	Suitable for explorations in moderately deep water areas where steel barges have influence on the measurement records. Not resistant to waves compared to steel barges.
	Non-magnetic small boat	7 to 12 m	Suitable for explorations in narrow and shallow water areas because of favorable maneuvering performance.
Self-propelled vessel	Non-magnetic small boat	1 to 7 m	Suitable for shallow water areas and does not require lowering a sensor frame to deeper depths.

(Note) The values of the applicable depths are approximations.

3) Sensor frame assembling

In offshore horizontal magnetic exploration, a sensor frame mounted with multiple magnetic exploration sensors is lowered close to the seafloor and towed by an exploration vessel so as to simultaneously obtain data on multiple measuring lines.

The width that can be measured in a single navigation of an exploration vessel is called an exploration effective width. Widening the exploration effective width can be achieved by increasing the number of sensors but this causes the sensor frame to be enlarged, thereby making the control of the posture, position, and depths of the sensor frame difficult. In many cases, when using dual-coil type magnetic gradiometer or differential type fluxgate magnetic probes for detecting mines, five sensors are arranged at 2 m intervals on the sensor frame. In this case, the intervals of the measuring lines are 2 m and the exploration effective width is 10 m.

When widening the intervals of the sensors (measuring lines), the exploration effective depths get shallower. Thus, the intervals shall be determined with due consideration to exploration objects, the performance of the measuring devices and required exploration depths.

4) Fitting out of exploration vessels and setting of measuring devices

The exploration vessels shall be mounted with measuring devices to be ready for measurement and a winch to suspend the sensor frame so as to enable its depth to be adjusted in accordance with the water depth. An echo sounder shall be set at a location suitable for simultaneously measuring the depths of the seafloor and the sensor frame. When using GNSS for the positioning of the vessels, an antenna needs to be fixed to a place that facilitates the reception of radio waves from satellites above the sensor frame.

Operation checks of all measuring devices shall be conducted after connecting them to confirm that they can properly obtain magnetic records, data on vessel positions and echo sounding records, and that they are synchronized.

Vessels towing sensor frames shall be provided with lamps or signs showing that the vessels have limited maneuverability.

(b) Measurement

Measurements shall be conducted continuously while navigating the exploration vessels in the exploration areas. The positions of the vessels are measured using GNSS in many cases. The accuracy of the positions of the vessels is specified in the **Common Specifications for Design, Surveying and Research Business**.[15)]

Track charts shall be created based on the positioning data so as to navigate the exploration vessels while confirming that the vessel tracks cover the entire exploration areas. The intervals of the vessel tracks that are wider than the exploration effective width mean that there are still unexplored areas remaining. Thus, measurements shall be continued until the intervals become narrower than the exploration effective width everywhere in the exploration areas.

The scope of the application of the exploration effective depth is that the sensor frame is within a certain distance from the seafloor (in many cases, within 1.0 m from the seafloor). It is necessary to confirm whether or not there are periods when the sensor frame has been kept higher than a certain height using the echo sounding records and to reject the measurements obtained during these periods.

(c) Analysis and discussion

Magnetic irregularities need to be extracted from the magnetic records obtained through the measurements, and the positions and magnetic parameters of the magnetic irregularities as well as their distances from the sensors need to be analyzed. The positions with magnetic parameters equal to or higher than the specified values shall be plotted on track charts. Then, the magnetic irregularities which are detected on multiple measuring lines and considered to have an identical cause are collectively defined as a magnetically irregular point. The buried depth of the magnetically irregular point needs to be obtained from the analysis results of the magnetic records and the heights of the sensor frame when the magnetic irregularities are obtained.

The discussion about the analysis results shall address the examination and determination of whether the possible cause of the magnetically irregular point is the influence of existing structures, an object much larger than the exploration objects or a long material such as a chain based on the detection ranges of the magnetic irregularities.

② **Submersible and diver exploration**

(a) Preparation

1) Field surveys and reference material collection

The preparations for submersible and diver exploration include observations of the conditions of the exploration areas and their surroundings as well as the collection of any necessary reference materials.

It is necessary to confirm at least the following items.

- Meteorological and hydrographic conditions
- Water depths and seafloor topography of the exploration areas and the properties of the seafloor geology
- Presence or absence of construction works in and around the exploration areas while implementing submersible or diver exploration
- Presence or absence of existing structures with possible influences on the exploration and confirmation of abnormal materials using magnetometers
- Presence or absence of existing structures subjected to the influence of the excavation at abnormal points
- Related organizations requiring explanations and consultations
- Scheduled ship navigation such as regular liners

2) Magnetometers and excavation devices

Magnetometers suitable for the local conditions shall be used.

Excavation devices (water jets, air lifts, etc.) suitable for the local conditions shall be selected while taking into consideration the properties of the seafloor soil. In the case of deep magnetically irregular points, mechanical excavation shall be considered. Furthermore, it is necessary to avoid the use of strongly magnetized devices which pose a problem for exploration and a risk to trigger accidental explosions.

It is also necessary to avoid the use of iron diving goods.

(b) Exploration

1) Submersible exploration

The positions of the magnetically irregular points shall be marked with buoys or flag poles.

The exploration areas shall be divided into strips having widths of 1 to 2 m, and exploration of the strips shall be thoroughly conducted in series so as not to leave any unexplored areas. Then, the points of the magnetic responses shall be excavated in order to identify magnetically abnormal materials.

Here, it is necessary to select the appropriate excavation methods while taking into consideration the seafloor topography and current directions. Abnormal materials which turn out not to be mines or bombs through excavation shall be recovered in principle, then examined to determine whether or not the recovered materials have magnetic parameters equivalent to those detected as magnetically irregular materials.

After the confirmation and recovery of the magnetically irregular materials, the excavated seafloor shall be explored once again with magnetometers for the possible omission of other abnormal materials below or around those already recovered. Then, exploration shall be shifted to the remaining areas.

When no abnormal materials are found through submersible exploration, it is necessary to reconfirm the exploration positions and examine the possibility that iron sand, magnetic rock or noise has caused the detection of magnetically irregular points.

2) Diver exploration

The exploration areas shall be marked with buoys or flag poles.

The exploration areas shall be divided into strips having widths of 1 to 2 m, and exploration of the strips shall be thoroughly conducted in series so as not to leave any unexplored areas. In diver exploration, there are many cases where existing structures cause difficulties in the analyses of magnetic records, and, therefore, all of the magnetic irregular points are generally subjected to the confirmation by excavating the seafloor.

In the case of difficulties in implementing the diver exploration with magnetometers, tamping rods or visual explorations (only when underwater visibility is favorable) can be used as substitutes.

3) Recovered materials

Recovered abnormal materials shall be stored in predetermined places and appropriately disposed of as industrial waste.

In the case of difficulties in recovering detected abnormal materials because of weights or other reasons, the measures to recover them shall be additionally examined.

4) Measures to be taken when mines or bombs are found

When detected abnormal materials are considered to be or have the possibility of being mines or bombs, excavation works shall be suspended and the locations of the suspected mines and bombs shall be marked. Then, necessary measures shall be taken so as to prevent the abnormal materials from being displaced.

③ **Onshore horizontal magnetic exploration**

Onshore horizontal magnetic exploration is to detect mines and bombs buried at shallow places close to ground surfaces. The exploration objects are normally bombs and shells, but there are cases of exploring mines.

(a) Preparation

1) Field surveys and reference material collection

The preparation for onshore horizontal magnetic exploration includes observations of the conditions of the exploration areas and their surroundings as well as the collection of any necessary reference materials with particular focus on the following items.

- The conditions of the ground surfaces (flatness and the presence or absence of pavement)
- The presence or absence of underground pipes in the exploration areas
- The presence or absence of existing structures with a possible influence on magnetic records
- The presence or absence of high-voltage power lines and railroads (to be possible noise generation sources due to earth current)

2) Setting of measuring lines

Measuring lines shall be basically straight lines arranged at intervals appropriate for the exploration objects. Intervals of 1 m are normally set for the exploration of 250 kg bombs. In the case of exploring smaller bombs or shells that are smaller than 250 kg bombs, it is necessary to set appropriate intervals, for example, 0.5 m, which enable them to be reliably detected.

In addition, coordinate systems shall be determined so as to enable the magnetically irregular points to be accurately positioned when executing excavation confirmation after analyses of the exploration data.

(b) Measurement

Magnetic records shall be continuously measured by a measurer walking along the measuring lines with a magnetic sensor. In order to clearly identify the measuring positions on the measuring lines, measured records shall be provided with positional marks at appropriate intervals.

The height of the magnetic sensor during measurements shall be basically 10 to 20 cm above the ground surface. Increasing the height of the magnetic sensor can reduce ground noise but weakens the magnetic signals from the exploration objects, thereby causing the exploration depths to be shallower in many cases. Thus, the height of the magnetic sensor shall be appropriately determined in accordance with the local conditions.

(c) Analysis and discussion

The magnetic irregularities shall be extracted from the measured magnetic records for analyses of the positions, magnetic parameters and buried depths. In addition, a list of magnetic irregularities points shall be created in a manner that plots magnetic irregularities with magnetic parameters higher than the predetermined values on a plan view.

The discussion about the analysis results shall address the examination and determination of whether the possible causes of the magnetic irregularities are the influence of existing structures, an object much larger than the exploration objects (foundation of old structures) or long materials such as underground pipes based on the detection ranges of the magnetic irregularities.

④ Exploration with metal detectors

Metal detectors can be used for the exploration of mines and bombs buried at shallow places close to ground surfaces.

(a) Preparation

1) Field surveys and reference material collection

The preparation for exploration with metal detectors includes observations of the conditions of the exploration areas and their surroundings as well as the collection of any necessary reference materials with particular focus on the following items.

- The conditions of the ground surfaces (flatness and the presence or absence of pavement)
- The presence or absence of underground pipes in the exploration areas
- The presence or absence of existing structures with a possible influence on exploration

2) Setting of measuring lines

The intervals of measuring lines are basically about 1 m and need to be appropriately set while taking into consideration the sizes of the exploration objects and the local conditions.

In addition, coordinate systems shall be determined so as to enable the positions of the metal responses to be clearly identified.

(b) Measurement

A metal detector shall be slowly moved along the measuring lines and the positions of the metal responses shall be marked.

It is difficult for metal detectors to determine the buried depths of metallic objects. Metal detectors simply suggest whether or not objects with metal responses are buried extremely close to ground surfaces.

⑤ Confirmation of onshore magnetic irregularities

(a) Preparation

1) Field surveys and reference material collection

The preparation for confirmation of onshore magnetic irregularities includes observations of the conditions of the excavation areas and the collection of any necessary reference materials with particular focus on the following items.

- Ground conditions and groundwater levels
- The presence or absence of underground pipes in the excavation areas
- The presence or absence of the influence of excavations on existing structures nearby

When excavation depths are deep, it is necessary to enclose the excavation areas with sheet piles. Vertical magnetic exploration can be implemented, as needed, to confirm that the sheet pile installation positions are free of mines and bombs.

(b) Excavation confirmation

When an excavation depth is deep, heavy excavation machines shall be used. It is necessary to frequently confirm the position of the abnormal materials with a magnetometer or a metal detector during excavation, and, when determining if the excavation has come close to the abnormal materials, further excavation shall be carefully executed by hand.

When abnormal materials become visible, it is necessary to identify whether or not the magnetic irregularity is a mine or bomb at the earliest possible time. Then, the magnetic irregularity is removed when it is determined to be removable and that it is not a mine or bomb.

(c) Measures to be taken when identifying mines and bombs

When an excavated magnetic irregularity is identified or suspected to be a mine or bomb, excavation work shall be stopped and measures shall be taken to keep the abnormal materials immobile and enclosed with a fence to keep people away.

⑥ Vertical magnetic exploration

Vertical magnetic exploration is planned and implemented to detect mines and bombs buried at the depths to which horizontal magnetic exploration is difficult. Vertical magnetic exploration is normally implemented for bombs and shells, but there are also cases of using it for mines.

(a) Preparation

1) Field surveys and reference material collection

The preparation for vertical magnetic exploration includes observations of the conditions of the exploration areas and their surroundings as well as the collection of any necessary reference materials with particular focus on the following items.

- The conditions of the ground surfaces (the presence or absence of pavement)
- The presence or absence of underground pipes in the exploration areas
- The presence or absence of existing structures with a possible influence on magnetic records
- The presence or absence of high-voltage power lines and railroads
- Methods for supplying drilling water and processing waste water after drilling

Because vertical magnetic exploration requires boring work, it is important to preliminarily confirm the existence of underground installations.

The soil properties of the exploration areas shall be surveyed to obtain materials for drilling exploration boreholes. In the case of loose sand which is insensitive to the disturbance of boring work and allows a large volume of water to be secured, the jet boring method can be used for drilling boreholes. In other cases, the rotary boring method is generally used.

(b) Measurement

Exploration boreholes shall be drilled at the exploration points (exploration holes) identified through surveying.

When using the rotary boring method, in order to prevent dangerous contact of the tip of the boring bits with any bombs, the safety of the drilling exploration boreholes needs to be secured by conducting magnetic measurements at the appropriate depth intervals during boring. In the case of exploring for 250 kg bombs, magnetic measurements are generally conducted at the bottom of the borehole every time the borehole is deepened by 1 m.

When using steel rods and bits for boring exploration boreholes, they shall be removed from the boreholes while magnetic measurements are conducted inside, and the borehole walls may be supported by PVC pipes if necessary. In many cases, nonmagnetic rods (stainless rods) are used so as not to interfere with the magnetic measurements.

After drilling down to a predetermined depth, magnetic measurements shall be conducted for the entire borehole.

The magnetic exploration instrument used in vertical magnetic exploration is small diameter magnetic sensors developed specifically for use in boreholes.

(c) Analyses and discussion

The magnetic irregularities shall be extracted from the measured magnetic records for analyses of the depths of the magnetically irregular points, the magnetic parameters and the distances from the exploration boreholes. In addition, a list of magnetic irregularities shall be created in a manner that plots the magnetic irregularities with magnetic parameters higher than the predetermined values on a plan view.

The discussion about the analysis results shall address the examination and determination of whether the possible causes of the magnetically irregular points are the influence of existing structures, an object much larger than the exploration objects or underground installations.

Additional magnetic measurements may be conducted by drilling additional boreholes when necessary for identifying the positions of the magnetically irregular points.

(6) Other

① Magnetic parameters

The International System of Units for magnetic parameter is [μWb].

The unit of [gauss·cm^2] (NGP) was previously used but has been replaced by [μWb] since the revision of the Measurement Act. For reference, 1 [gauss·cm^2] (NGP) ≈ 0.7 [μWb].

For hazardous material exploration, the magnetic moment (in the unit of [gauss·cm^3]) was also used in addition to the magnetic parameter, but the two units have recently been integrated into the magnetic parameter.

② Exploration objects other than mines and bombs

In addition to the exploration of hazardous materials such as mines and bombs, magnetic exploration has also been used for other purposes. Typical cases are the detection of anchors and pipelines in the case of offshore magnetic exploration, and confirmation of the presence or absence of existing piles and their installation depths as well as the detection of underground pipes in the case of onshore magnetic exploration.

3.9 Subgrade and Base Course Tests

3.9.1 General

The subgrade and base course of the aprons of quaywalls and shallow draft wharves, container yards and port roads shall have sufficient stiffness so as to ensure bearing resistance to the loads acting on them and stability with respect to natural conditions such as meteorological phenomena. Thus, it is necessary that the design and construction methods of the subgrade and base course are determined, and the supervision of the construction work of the subgrade and base course is appropriately implemented based on an accurate understanding of the engineering characteristics of the soil and construction conditions at the sites through various types of tests.

The subgrade positioned below the pavement needs to be stable with respect to the repetitions of traffic loads and meteorological actions and have a function to be the formation level of the base course, which is part of the pavement and constitutes the lower pavement layers. The surveys and tests related to the structural design and subsequent construction and maintenance of pavement generally include the following in the cases of roads and airports.

- Field soil density test
- Field compaction test
- In-situ CBR test
- Plate loading test
- Compaction test using a rammer
- Laboratory CBR test

- Other tests

3.9.2 Field Soil Density Test

(1) Purpose

The compactness or stability of the soil can be estimated from its density. Thus, a field soil density test is conducted for estimating the compactness of the subgrade and base course as well as the soil at places where the subgrade and base course are to be constructed, thereby utilizing the compactness in the design and supervision of the construction work of the subgrade and base course.

The field soil density test shall be conducted through the sand replacement method in accordance with the **Test Method for Soil Density by the Sand Replacement Method (JIS A 1214)**. In addition to the above method, the **Test Method for Soil Density Using Nuclear Gauge (JGS 1614)** can be used as a reference.

(2) Organization of tests and data

The field soil density test is conducted in the following manner.

① **Test equipment**

The test equipment used in the field soil density test includes a density measurement device (comprising a jar, an attachment and a base plate), a glass plate, test sand and a scale (as shown in **Fig. 3.9.1**).

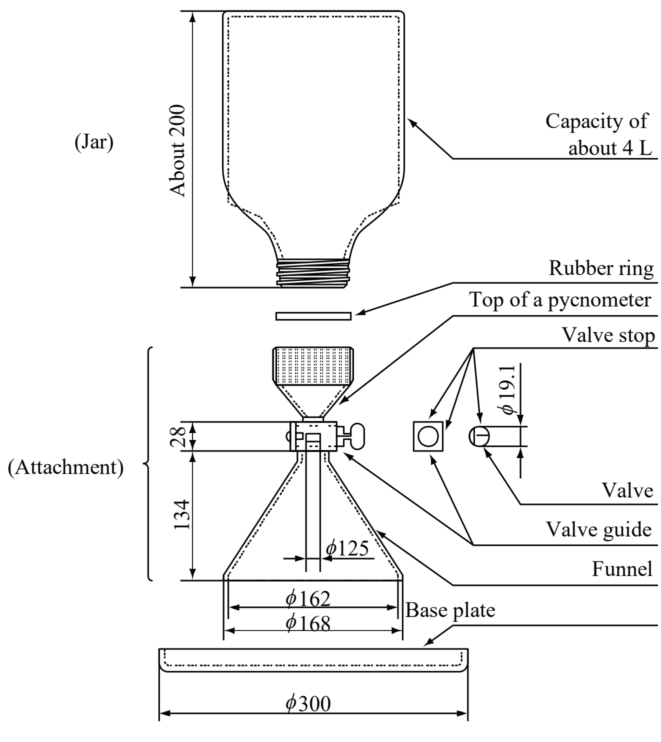


Fig. 3.9.1 Test Equipment Used in the Field Soil Density Test

② **Advance preparation**

As advance preparation for the field soil density test, the volume of the density measurement device and the density of the test sand need to be calibrated. The calibration of the sand density shall be conducted in a manner that obtains the mass of the test sand in the device by subtracting the mass of the device from the mass of the device filled with test sand and measured with particular care to cause vibrations which could result in an overestimated density for the test sand. Furthermore, because the water content and grain sizes of the sand cause variations in its density, thereby affecting the soil density test results, the test sand shall be managed in a manner that keeps the water content and grain sizes of the test sand at the time of calibration stable while the test sand is transported, stored and used in field tests.

③ **Test preparation**

The preparation of the field soil density test shall be conducted as follows.

i. The ground surface of the area to be tested for soil density shall be flattened with a blunt blade.

ii. A base plate shall be placed firmly on the flattened ground surface.

iii. The soil shall be dug out through a hole on the base plate in a manner that excavates a vertical hole without disturbing the soil to the extent possible using a test hole excavation tool. The entire volume of the soil dug out through the hole shall be kept in a closed container so as to prevent its water content from being changed.

④ **Test method**

The mass of the soil excavated from the test hole and the volume of the test hole are measured in the following order.

i. Measurement of the mass of soil excavated from the test hole

ii. Measurement of the water content of the soil which has been thoroughly mixed after the measurement of the mass

iii. Measurement of the mass of the sand required for filling the test hole in a manner that sets the measuring instrument with a funnel inserted into the hole on the base plate and pours sand stored in a jar into the test hole through the funnel

⑤ **Methods for organizing test results**

The density of the soil can be obtained by the following method.

- Wet density ρ_t (g/cm^3)

$$\rho_t = \frac{m_7}{V_0} \tag{3.9.1}$$

$$V_0 = \frac{m_{10}}{\rho_{ds}} \tag{3.9.2}$$

where

V_0 : the volume of the test hole (cm^3);

m_7 : the mass of the soil excavated from the test hole (g);

m_{10} : the mass of the sand required for filling the test hole (g); and

ρ_{ds} : the density of the test sand (g/cm^3).

- Dry density ρ_d (g/cm^3)

$$\rho_d = \frac{m_0}{V_0} \tag{3.9.3}$$

where

m_0 : the mass of the soil excavated from the test hole and dried in an oven (g).

(3) Method for utilizing test results

The determination results of the compaction degrees of the subgrade, base course, cut ground and filled ground shall be used for supervision of the compaction work.

3.9.3 Field Compaction Test

(1) Purpose

A field compaction test is conducted to determine the most suitable compaction method for the conditions at the construction site through the implementation of test construction. The specific examination items in the field compaction test are the types and combinations of compaction machines, soil to be used, compaction water content, spreading thicknesses, number of compactions, compaction speeds and compaction methods.

(2) Organization of tests and data

Because there have been no standardized field compaction test methods, the test items shall be appropriately selected from the following in accordance with the construction site conditions.

- The types and combinations of compaction machines
- Soil to be used
- Compaction water contents
- Spreading thicknesses
- The number of compactions
- Compaction speeds
- Compaction methods

① **Test equipment**

The equipment to be used in the field compaction test includes a level, a staff, celluloid plates or vinyl sheets, lime or settlement plates, a scale, an appropriate sounding device, an unconfined compression test machine and a stopwatch.

② **Test plan**

In the field compaction test plan, the following items shall be examined.

(a) Selection of compaction machines

The machines to be used in the field compaction test are subject to the scale of the construction and the types of soil. Some machines enable the compaction weight to be adjustable; in such cases, the field compaction test shall be conducted by changing the compaction weight in stages. In addition, it is necessary to conduct the field compaction test by changing the air pressure in tires when using machines provided with tire rollers, and changing the frequency or vibration force when using machines provided with vibrators.

(b) Selection of soil to be used

Typical soil materials to be used in the actual construction shall be selected for use in the field compaction test. When several types of soil are planned to be mixed in the actual construction, the mix proportion of the soil shall follow the one used in the field compaction test.

(c) Compaction water content

When the natural water content is lower than the optimal water content obtained through a laboratory compaction test (refer to **Reference [Part II], Chapter 1, 3.9.6 Compaction Tests Using a Rammer**), the moisture content shall be adjusted by sprinkling water and the field compaction test shall be conducted with the following three types of water content.

i. Natural water content: When the natural water content is extremely low, the water content shall be adjusted by sprinkling water.

ii. Water content close to the optimal water content obtained through laboratory compaction tests

iii. Water content slightly higher than the optimal water content obtained through laboratory compaction tests: When the natural water content is higher than the optimal water content obtained through the laboratory compaction test, the water content to be used for the field compaction test shall be either the natural water content or the water content determined after adjusted the natural water content by spreading soil under the sun to a level reasonably achievable or by other methods.

(d) Spreading thicknesses

A few types of spreading thicknesses shall be tested, taking into consideration the types of machines to be used and their expected work efficiency.

(e) Number of compactions

The field compaction test shall be planned so as to complete each test with the number of compactions kept at a range of 10 to 15 times. When the number of compactions needs to be more than the range, it is preferable to examine the possible use of alternative machines.

(f) Combinations of machines

The field compaction test shall be conducted several times on a trial basis for possible combinations of machines in full consideration of economic efficiency. When conducting the field compaction test for soil with a high proportion of gravel or rock debris, it is necessary to set a larger spreading thickness than in normal cases. In addition, the field compaction test shall be terminated when compacted surfaces undergo extreme settlement as the field compaction test progresses because the machines may be too heavy, or the soil may undergo lateral flow. In the case of applying particularly high compaction speeds, it is necessary to fully examine the appropriate compaction speeds.

③ **Setting of test locations**

When setting the test locations of the field compaction test, it is necessary to examine the following items.

(a) Typical test locations

Locations selected from areas which are expected to produce field compaction test results representing the construction sites and which are flat and well-drained to the extent possible.

(b) Boundaries between object soil layers and original ground

Identification of the original ground with lime spread over it or settlement plates so as to enable the degree of ground settlement to be evaluated after the field compaction test.

(c) Test preparation

Preliminary implementation of the measurement of the in-situ density and water content as well as penetration tests should be conducted in advance.

(d) Test section in accordance with test conditions

The test locations shall be appropriately determined in accordance with the machines to be used and the site conditions. The minimum dimensions of the compartments required for each field compaction test shall be a width of 4 m and a length of 10 m in general, but the length can be shortened to about 5 m when a rammer or vibration compactor is used. In cases where the test object is test earth fill having monotonous soil properties, the slopes of the earth fill shall be 1:1 when the earth fill height is less than 1 m, or 1:1.5 when the height is 1 m or higher.

(e) Edges of test sections

Test sections shall be provided with slopes at their edges so as to allow the test machines to be mobilized and demobilized through them. Flat sections with a length of at least 3 to 4 m shall be provided between the tops of the slopes and test sections to prevent the slopes from affecting the test sections and to allow the test machines to change their traveling directions.

④ **Test items and test methods**

The test items and test methods of the field compaction test are as follows.

(a) Density and water content

The density and water content of the test earth fill shall be measured at the appropriate depths. Because the measurements of the in-situ density have a large dispersion, it is necessary to repeat the measurements at least three times—preferably five times—at one location to improve the measurement accuracy. Furthermore, measurements shall be carried out intensively before and after compaction (prior to the spreading of successive layers).

(b) Amounts of settlements

The amounts of settlements on the surfaces and at insides of the test section shall be measured. The settlements on the surfaces shall be measured in a manner that uses levels and staffs, or uses scales to measure the distance of leveling lines stretched between piles at both sides of the test sections in locations free of the influences of compaction to the surfaces. When using levels, it is necessary to pay attention to their installation locations because levels installed too close to the test sections reduce the measurement accuracy and interfere with the operation of the test machines and the work of the personnel. In addition, measurements shall be carried out repeatedly at 10 or more fixed locations in the test sections. When using levels, it is also necessary to install temporary benchmarks close to the test sections and regularly check the elevations of the benchmarks during measurement.

Furthermore, when measuring the settlement at the insides of the test section, a few celluloid plates or vinyl sheets (about 5 cm × 5 cm) shall be preliminarily embedded in the earth fill in a depth of about 5 cm intervals and the soils above the plate shall be removed at a predetermined number of compactions to measure the settlement.

(c) In-situ strength

In-situ strength is generally measured with simple penetration test equipment. For the in-situ strength test methods, reference can be made to **Reference [Part II], Chapter 1, 3.9.8 (1) Simplified bearing test**. In the case of soil with a high proportion of gravel to which sounding cannot be applied, the in-situ strength of such soil shall be measured through an in-situ CBR test, a plate loading test, or an unconfined compression test with core samples taken out of the sites.

(d) Compaction speeds

The time required for the test machines to travel between the gauge marks with known distances in the respective test sections shall be measured with a stopwatch.

In addition, the earth pressure, pore water pressure, settlement of original ground, lateral flows and inclinations shall be measured as needed.

There are the following points of caution in the field compaction test.

- The spreading of compaction layers shall be carried out with homogeneous materials so as to achieve a uniform distribution of the soil density throughout the layers. Because even freshly spread compaction layers are subjected to compaction to some extent before the initial compaction due to the movement of heavy equipment (such as bulldozers) used for spreading materials, it is necessary to stabilize the conditions for the spread compaction layers before compaction throughout the field compaction test.
- The measurement points of the respective layers shall be evenly distributed to the test sections. When measuring the density many times, it is particularly necessary to clearly identify the locations of the measuring holes. Otherwise, measuring holes have to be repeatedly excavated at identical locations.
- In the case of test sections with minimal widths, it is necessary to strictly control the tracks of the compaction machines, preferably by guiding them along leveling lines or white lines painted with lime preliminarily provided on the earth fill together with sign poles. It shall be noted that the compaction machines might deviate from the tracks depending on the skills of the operators.
- Because the field compaction test takes a long time and the test results are likely to be subjected to weather conditions, it is necessary to conduct the field compaction test as quickly as possible. To that end, it is also necessary to establish field compaction test plans, thereby enhancing the test efficiency.
- It is not necessary to measure all the test items after every compaction. A single compaction can cause remarkable changes in the measured values while the number of compactions is still small, but barely causes changes in the measured values after several compactions. Thus, the time intervals of the measurements can be extended as the number of compactions is increased.

⑤ Organization of data

The field compaction test results shall be organized and utilized as follows. Here, similar to the difficulty in standardizing the field compaction test methods as mentioned above, it is also difficult to standardize the formats to organize the results of the field compaction test. Because there are too many test items and factors that affect them to describe in this limited space, only the correlations between the selected items and their influential factors are summarized as clearly as possible below.

(a) Relationship between the number of compactions and density

A figure with the number of compactions and dry density on the respective axes shall be established for the respective water contents or compaction machines. An example of such a figure is shown in **Fig. 3.9.2**.

(b) Relationship between the number of compactions and settlement

When the settlement at inside of the compaction layers are measured, a figure with the number of compactions and settlement on horizontal and vertical axes, respectively, shall be established for the compaction machines or spreading thicknesses. For the settlement inside the earth fill, a figure with the depths from the compaction surfaces and settlement amounts of the compaction layers on horizontal and vertical axes, respectively, shall be established. An example of a figure for the settlement inside of the earth fill is shown in **Fig. 3.9.3**.

(c) Relationship between the number of compactions and penetration test results

Figure with the number of compactions and penetration test results on horizontal and vertical axes, respectively, shall be established for the compaction machines and spreading thicknesses.

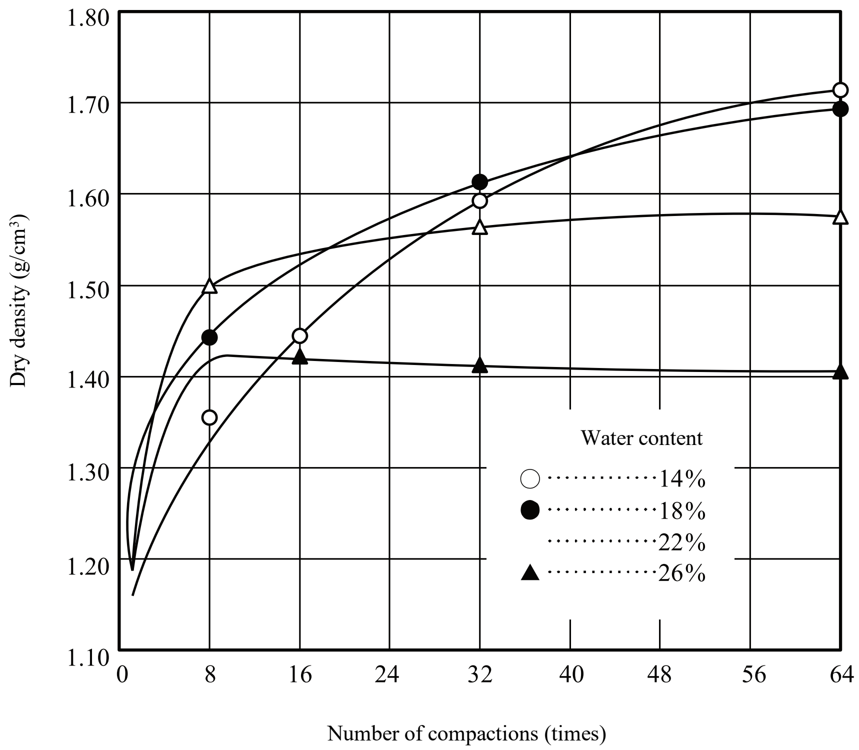


Fig. 3.9.2 Example of the Changes in Dry Density with an Increase in the Number of Compactions

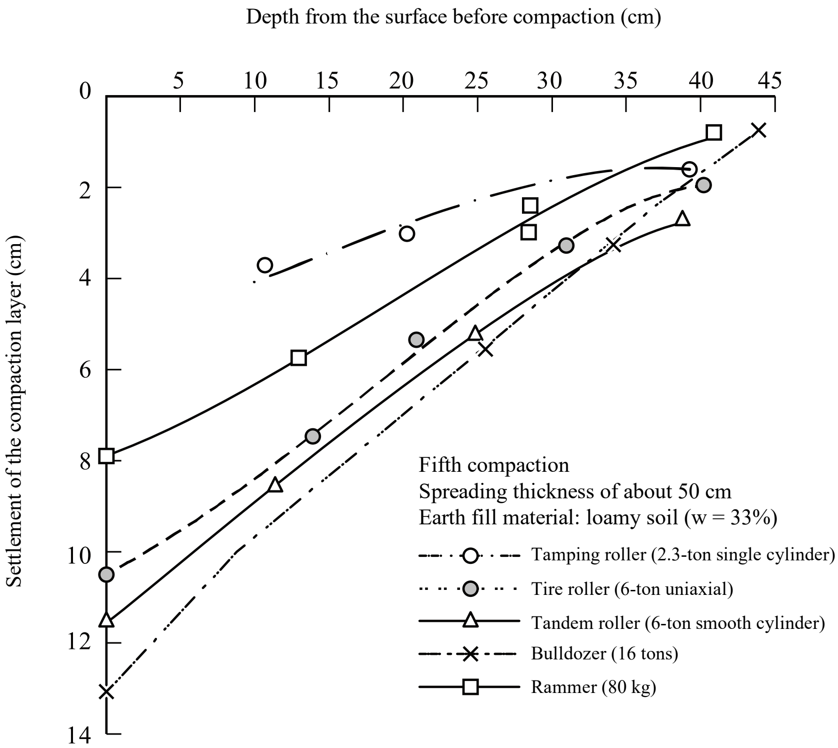


Fig. 3.9.3 Example of Measurements of the Settlement at inside of the Compaction Layers due to Compaction

(3) Methods for utilizing test results

The results of the field compaction test can be utilized in the following ways.

① **Number of compactions and settlement**

It has been known that the settlement on the surfaces and at inside of the compaction layers as well as the number of compactions by steel rollers satisfy the following equation.

$$\Delta h_N = \frac{N}{a + bN} \tag{3.9.4}$$

where

N : the number of compactions;

Δh_N : the settlement by the Nth compaction from the surface level after the previous compaction; and

a, b : coefficients.

With the increase in the number of compactions, the value of the above equation comes close to $1/b$. That is,

$$\lim_{N\to\infty} \Delta h_N = \lim_{N\to\infty} \frac{N}{a + bN} = \frac{1}{b} = \Delta h_\infty \tag{3.9.5}$$

Then, the following equation is obtained.

$$\frac{\Delta h_N}{\Delta h_\infty} = b\Delta h_N = \frac{N}{\dfrac{a}{b} + N} \tag{3.9.6}$$

Representing the degree of progress in the settlement after a certain number of compactions with respect to the final amount of settlement, the values of $\Delta h_N / \Delta h_\infty$ can be utilized in the determination of the compaction effects.

② **Number of compactions and density**

The number of compactions and density enable the compaction effects to be evaluated. In addition, they can be utilized to determine the number of compactions to achieve a sufficient degree of compaction and the types of compaction equipment to maximize the degree of compaction.

③ **Dry density and amount of settlement by depths**

The dry density and amount of settlement can be utilized in determining the spreading thicknesses suitable for the predetermined compaction methods.

In addition, they can by utilized in evaluating the degrees of compaction of the subgrade, base course, earth cut and earth fill, and in supervising the compaction work.

3.9.4 In-Situ CBR Test

(1) Purpose

The in-situ CBR test is conducted to measure the CBR values and the bearing capacity of the subgrade as well as the base course in situ as design materials. The test shall be conducted at six locations for each identical soil layer.

The in-situ CBR test shall be conducted in accordance with the **Test Method for the California Bearing Ratio (CBR) of In-Situ Soil (JIS A 1222)**.

(2) Organization of tests and data

① **Test equipment**

The test equipment used in the in-situ CBR test includes a load reaction (a truck or other simple movable object), a loading device (provided with a jack and a spherical support having a capacity corresponding to the expected load intensity and a mechanism capable of adjusting the piston penetration rate to 1 mm/min), a penetration amount measuring device (provided with a displacement meter, a fixture to fix it to the penetration piston and a support base), a penetration piston and a loading plate (as shown in **Fig. 3.9.4**).

② **Test method**

The in-situ CBR test is conducted in the following order.

i. A flat test surface with a diameter of about 30 cm shall be prepared at the test location. In cases where the ground surface cannot be leveled directly, a flat surface can be prepared by laying a thin layer of dry sand.

ii. A loading device shall be installed with a slight load not more than 49 N in order to bring the penetration piston into contact with the specimen.

iii. The penetration piston shall be inserted into the specimen at a rate of 1 mm/min by uniformly increasing the load until the penetration amount reaches 12.5 mm, and the value of the load required for achieving 12.5 mm shall be recorded.

iv. After the penetration test, a specimen shall be taken from the test location, and the water content and density shall be measured with the specimen.

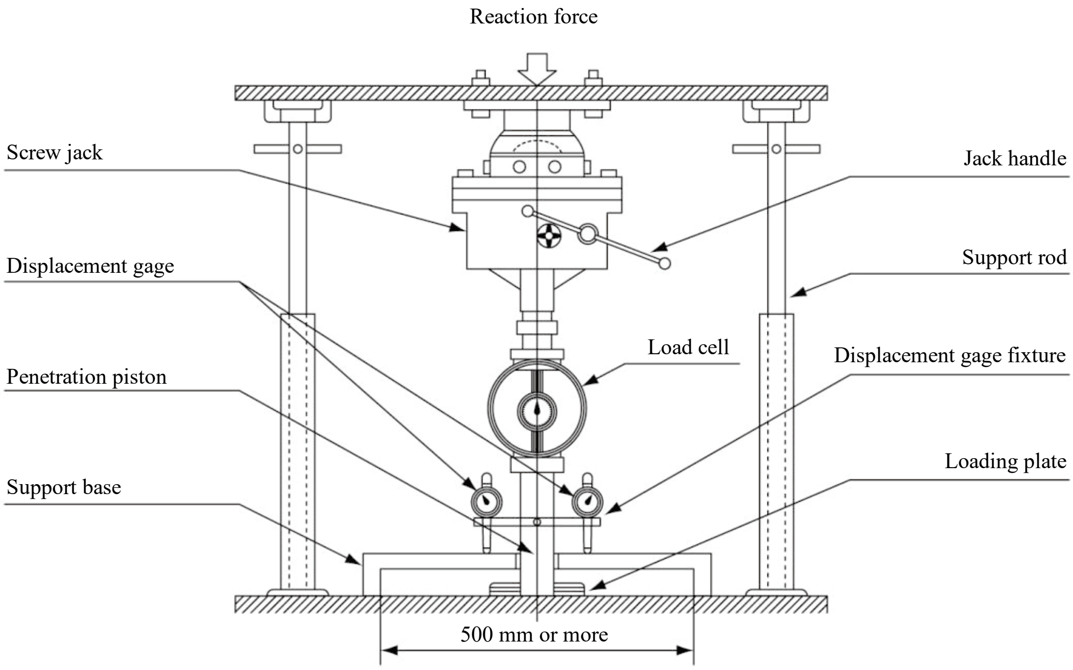


Fig. 3.9.4 In-Situ CBR Test Device

(3) Methods for utilizing test results

The CBR values shall be calculated as with the laboratory CBR test (**Chapter 1, 3.9.7**).

3.9.5 Plate Loading Test

(1) Purpose

The plate loading test is to obtain the bearing capacity of subgrade and base course from the loads applied to a loading plate and the settlement of the loading plate and to utilize the bearing capacity in the design of the subgrade and base course. The plate loading test shall be conducted in accordance with the **Method for Plate Load Test on Soil for Road (JIS A 1215)**.

(2) Organization of tests and data

① **Test equipment**

The equipment used in the plate loading test includes a loading plate (circular steel plates with a thickness of 22 mm or more and diameters of 30, 40 and 75 cm), a jack (with a capacity of 50 to 400 kN), a displacement gauge, a settlement measuring device (which comprises a support beam having a length of 3 m or more mounted with a displacement gauge fixing unit and its support legs, and which enables the support legs to be positioned 1 m or more away from the loading plate and the support points of the loading device), and a loading device (such as a vehicle or trailer which allows the required reaction force to be obtained and its supports points to be positioned 1 m or more away from the outer edge of the loading plates).

② **Test method**

The plate loading test is conducted in the following order.

i. Three test locations shall be selected for each identical soil layer by conducting auger boring.

ii. The test locations shall be selected from uniform areas with no coarse materials exposed or flattened out and will be covered with sand as needed.

iii. A loading plate shall be installed on the prepared test locations with a loading device and a settlement measuring device set up at the correct position.

iv. After completing the preliminary loading and unloading operation to stabilize the loading plate, the actual plate loading test shall be conducted until the settlement amount reaches 15 mm, or the applied load intensity exceeds the maximum contact pressure predicted at the site or the yield point of the ground.

There are the following points of caution for the plate loading test.

- The increments of loads during the plate loading test shall be 0.034 N/mm^2. The load intensity and settlement amount shall be recorded after the progress of settlement is stopped. Here, the progress of settlement is considered to be stopped when the ratio of the settlement amount in one minute with a certain load intensity to the total settlement amount becomes 1% or less.
- The support legs of the displacement fixing unit shall be set at positions 1 m or more away from the loading plates and ground contact points of the loading device so that they do not affect the loading.
- The plate loading test shall be conducted efficiently and smoothly so as to avoid any influences from changes in the weather conditions on the test locations with a particular focus on the availability of drainage at the test locations in anticipation of rain, snow or an inflow of groundwater.

③ **Organization of test results**

A curve showing the relationship between the loads and settlement as shown in **Fig. 3.9.5** shall be created using the test results.

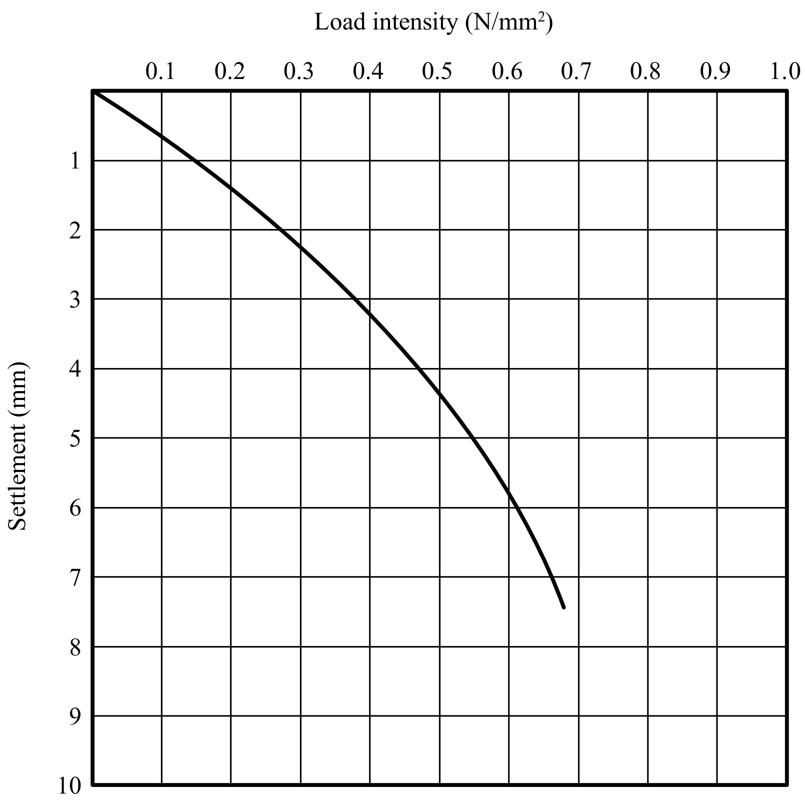


Fig. 3.9.5 Example of a Curve Showing the Relationship between the Loads and Settlement in the Plate Loading Test

The coefficient of bearing capacity K (N/mm^3) can be calculated by substituting the load intensity corresponding to a certain settlement amount obtained through the curve into the following equation.

$$K = \frac{\text{Load intensity (N/mm}^2\text{)}}{\text{Settlement (mm)}} \qquad \textbf{(3.9.7)}$$

Because the actual curves representing the relationship between the loads and settlement do not exactly show a linear relationship as shown in **Fig. 3.9.5**, the value of the bearing capacity coefficient varies depending on the methods for obtaining the settlement amount. Thus, the value of the bearing capacity coefficient shall be calculated in accordance with the allowable settlement to be determined depending on the type of pavement. Generally, the allowable settlement is set at 1.25 mm and 2.5 mm for concrete and asphalt pavement, respectively.

(3) Methods for utilizing test results

The results of the plate loading test can be utilized in the following ways.

- The coefficient of bearing capacity of the subgrade or base course obtained through the plate loading test represents the load bearing characteristics of the subgrade or base course and is generally utilized in designing the thicknesses of the concrete pavement. The plate loading test is also conducted for the supervision of the construction of the base course.
- The coefficient of bearing capacity varies depending on the sizes of the loading plates. In the case of concrete pavement for the aprons of quaywalls and shallow draft wharves, the standard method for the plate loading test is implemented to obtain a value corresponding to the allowable settlement of 1.25 mm with a diameter of 30 cm.
- The following equation is used when converting a coefficient corresponding to the loading plate with a diameter of 30 cm to that with a diameter of 75 cm.

$$K_{75} = \frac{1}{2.5} K_{30} \qquad \text{(Subgrade)} \tag{3.9.8}$$

$$K_{75} = \frac{1}{3.0} K_{30} \qquad \text{(Conventional base course)} \tag{3.9.9}$$

$$K_{75} = \frac{1}{5.0} K_{30} \qquad \text{(Stabilized base course)} \tag{3.9.10}$$

where

K_{30} : the coefficient of bearing capacity corresponding to the loading plate with a diameter of 30 cm (N/mm^3); and

K_{75} : the coefficient of bearing capacity corresponding to the loading plate with a diameter of 75 cm (N/mm^3).

- Generally, in the case of concrete pavement for the aprons of quaywalls and shallow draft wharves, the thickness and materials of the base course shall be determined so as to achieve the value of K_{30} on a base course surface that becomes 0.2 N/mm^3 or more. Then, it is preferable to construct a test base course based on the determined thickness and materials, and to confirm whether K_{30} can be reliably achieved on the test base course. When K_{30} does not reach 0.2 N/mm^3, the required thickness shall be determined using the test base course, or either the thickness or strength of the concrete shall be increased.
- In cases where the bearing capacity coefficient on the subgrade surface is susceptible to the water content of the subgrade soil, the actual measurement value shall be corrected using the following equation.

$$\begin{array}{c}\text{Coefficient of bearing} \\ \text{capacity (corrected)}\end{array} = \begin{array}{c}\text{Coefficient of bearing} \\ \text{capacity (actual)}\end{array} \times \frac{\text{CBR of undisturbed soil specimen (with 4-day immersion)}}{\text{CBR of undisturbed soil specimen (with natural water content)}} \tag{3.9.11}$$

3.9.6 Compaction Test Using a Rammer

(1) Purpose

The compaction degrees of soil largely vary depending on the water content and compaction methods and are particularly susceptible to the changes in water content. Thus, the actual compaction work is preferably implemented with a water content close to that which maximizes the compaction degree according to the in-situ soil properties. For this purpose, a compaction test using a rammer is conducted to experimentally obtain the compaction characteristics of the soil with respect to the changes in water content based on the soil and construction conditions at the sites, thereby determining the optimal water content and setting the standards for the supervision of the construction work.

A compaction test using a rammer shall be conducted in accordance with the **Test Method for Soil Compaction Using a Rammer (JIS A 1210)**.

(2) Organization of tests and data

① Outline

Depending on the differences in the compaction methods, a compaction test using a rammer is conducted through either the first or second method. The first and second methods are to compact the soil with a free dropping 2.5 kg rammer from a height of 30 cm and a free dropping 4.5 kg rammer from a height of 45 cm, respectively. Depending on the dimensions of the molds and the allowable maximum soil grain sizes in the specimens, the respective test methods are classified into several types which have different numbers of compaction layers and different numbers of compactions for the layers, as shown in **Table 3.9.1**. These methods are also classified into dry and non-dry methods depending on the differences in the preparation of the specimens, and repeated and non-repeated methods depending on the differences in the use of the specimens (**Table 3.9.2**). The dry method is to preliminarily dry all the specimens to a water content lower than that which is optimal and to adjust the water content to the required amount by adding water to the specimens. In contrast, the non-dry method is to adjust the natural water content to that which is required by drying the soils or adding water to them. The repeated method uses identical specimens several times by changing the water content, while the non-repeated method uses different soils for different water contents.

Table 3.9.1 Types of Compaction Tests Using a Rammer

Nominal designation of compaction test	Mass of rammer, kg	Inner diameter of mold, cm	Number of compaction layers	Number of compactions per layer	Allowable maximum grain size, mm
A	2.5	10	3	25	19
B	2.5	15	3	55	37.5
C	4.5	10	5	25	19
D	4.5	15	5	55	19
E	4.5	15	3	92	37.5

Table 3.9.2 Types of Specimen Preparation and Use Methods and Quantity of Specimens to Be Prepared

<table>
<tr><th>Nominal designation of combination</th><th>Combination of specimen preparation and use methods</th><th>Diameter of mold, cm</th><th>Allowable maximum grain size, mm</th><th>Minimum required quantity of specimen</th></tr>
<tr><td rowspan="3">a</td><td rowspan="3">Dry and repeated methods</td><td>10</td><td>19</td><td>5 kg</td></tr>
<tr><td>15</td><td>19</td><td>8 kg</td></tr>
<tr><td>15</td><td>37.5</td><td>15 kg</td></tr>
<tr><td>b</td><td>Dry and non-repeated methods</td><td>10
15</td><td>19
37.5</td><td>3 kg for each combination
6 kg for each combination</td></tr>
<tr><td>c</td><td>Non-dry and non-repeated methods</td><td>10
15</td><td>19
37.5</td><td>3 kg for each combination
6 kg for each combination</td></tr>
</table>

The compaction test using a rammer is conducted by combining the above methods, and the test results are attached with nominal designations indicating which combinations are used, as shown in **Tables 3.9.1** and **3.9.2**. The combinations for the actual implementation can be determined in accordance with the soil properties, construction methods, construction capacity and the methods for utilizing the test results with additional attention to the following points.

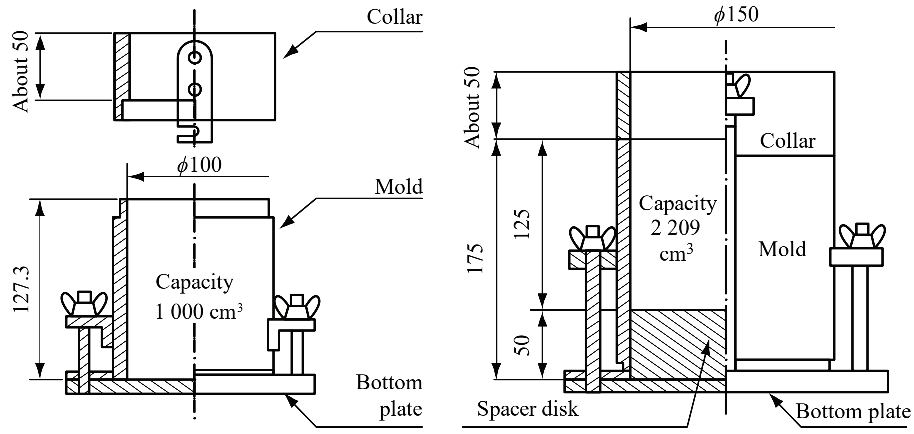
- The second method shall be used in cases where a compaction machine at the construction site has large compaction energy, or the specimens used for the compaction test using a rammer are also used as the specimens for the CBR test.
- The type "a" method shall be used in cases where the soil shows no differences in the test results between the dry and non-dry methods, and the test is conducted for the purpose of obtaining the reference values for determining the availability of materials and the supervision of the compaction work.

- The type "b" method shall be used in cases where the specimens of the test are also used as those for the CBR test and other mechanical and permeability tests even though the object soil is classified as general soil.
- The type "c" method shall be used in cases where the object soil has a high natural water content and is susceptible to dryness as is the case with volcanic cohesive soil.

② **Test equipment**

The equipment used in the compaction tests using a rammer include a mold, a collar, a bottom plate, a spacer disk, a rammer and a specimen extrusion device. The mold is made of steel and has a cylindrical shape with dimensions as shown in **Fig. 3.9.6**. The rammer has a circular outside edge face with a diameter of 5 cm, a shape and mass as shown in **Fig. 3.9.7**, and a mechanism enabling the dropping height to be adjusted to the prescribed height.

(Unit: mm)



a) 10 cm mold

b) 15 cm mold

Fig. 3.9.6 Mold

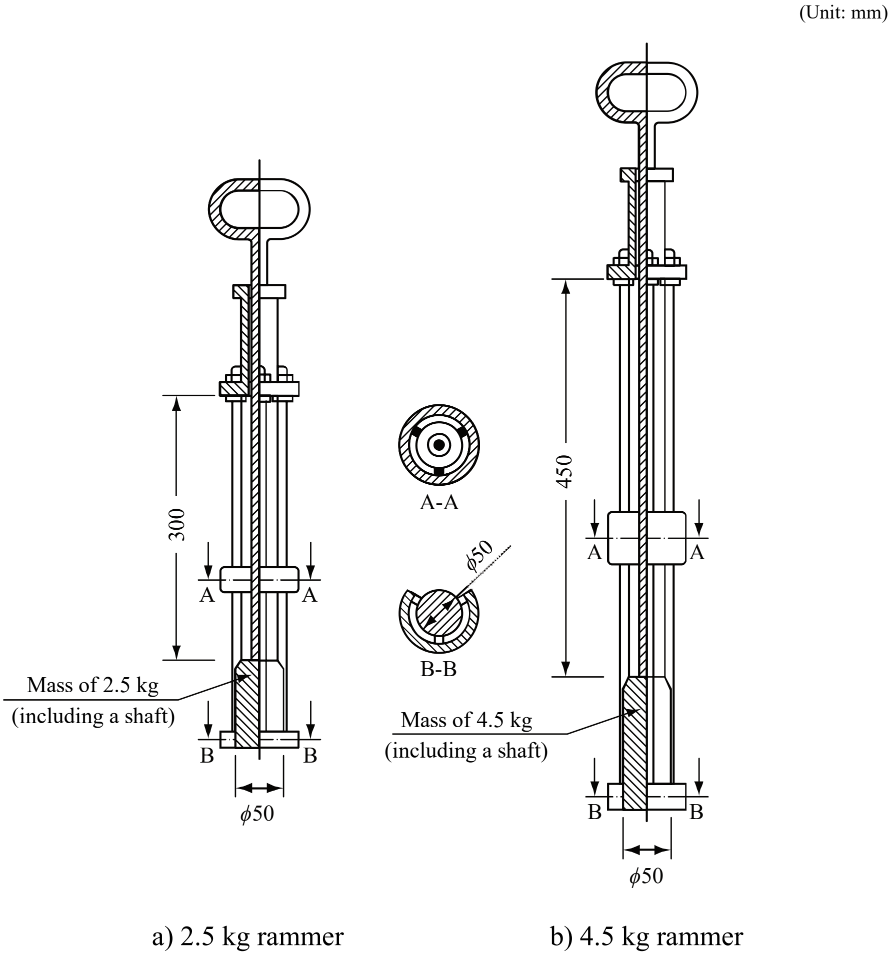


Fig. 3.9.7 Rammer

③ **Test method**

Compaction tests using a rammer are conducted in the following order.

i. Soils shall be put in a mold and subjected to compaction through the prescribed compaction method. In the case of a 15 cm mold, a spacer disk and filter paper shall be set in the mold before putting the specimen inside.

ii. The top surface of the compacted specimen shall be slightly higher than the upper edge of the mold but not more than 10 mm.

iii. After compacting the soil, the collar shall be removed and any surplus soil above the upper edge of the mold shall be gently removed with a blunt blade to expose a flat finished surface.

iv. Soil attached to the outside of the mold and bottom plate shall be thoroughly wiped off before measuring the mass of the entire equipment. Then, the mass of the compacted specimen shall be calculated from that measured mass.

v. The water content of the compacted specimen shall be measured after removing it from the mold with the specimen extrusion device.

Regardless of whether the repeated or non-repeated method is conducted, the procedure described above shall be repeated for 6 to 8 types of specimens having different water contents around the expected optimal water content. When the repeated method is conducted, the specimen used for the measurement of the water content after compaction shall be finely loosened to its original state before compaction, then the loosened specimen shall be mixed with additional soil and a prescribed amount of water to equalize the water content for the next test.

④ **Organization of test results**

The test results of the compaction test using a rammer shall be organized in the following way.

- Wet density of compacted soil ρ_t (g/cm^3)

$$\rho_t = \frac{m_2 - m_1}{V} \tag{3.9.12}$$

where

m_2 : the mass after compaction of specimen and mass of mold and bottom plate (g);

m_1 : the mass of the mold and bottom plate (g); and

V : the volume of the mold (cm^3).

- Dry density of compacted soil ρ_d (g/cm^3)

$$\rho_d = \frac{\rho_t}{1 + \frac{w}{100}} \tag{3.9.13}$$

where

w : the water content (%).

- Maximum dry density ρ_{dmax} (g/cm^3) and optimal water content w_{opt} (%)

  A compaction curve showing the relationship between the dry density and water content shall be created in a manner that plots the measured values on coordinates with the dry density and water content on the vertical and horizontal axes, respectively, and connects the measured values with a smooth curve. The density and water content corresponding to the apex of the curve are the maximum dry density ρ_{dmax} and optimal water content w_{opt}, respectively. **Fig. 3.9.8** shows examples of the compaction curves.

- Dry density ρ_{dsat} (g/cm^3) corresponding to the water content in a zero air void state

  The dry density corresponding to the water content in a zero air void state can be calculated by the following equation. The zero air void curve is obtained by connecting the calculation results plotted on the coordinates of the compaction curve and connecting them with a smooth curve.

$$\rho_{dsat} = \frac{\rho_w}{\frac{\rho_w}{\rho_s} + \frac{w}{100}} \tag{3.9.14}$$

where

ρ_w : the density of the water (g/cm^3); and

ρ_s : the density of the soil particles (g/cm^3).

The compaction curve showing the relationship between the water content and dry density shall additionally show a zero air void curve.

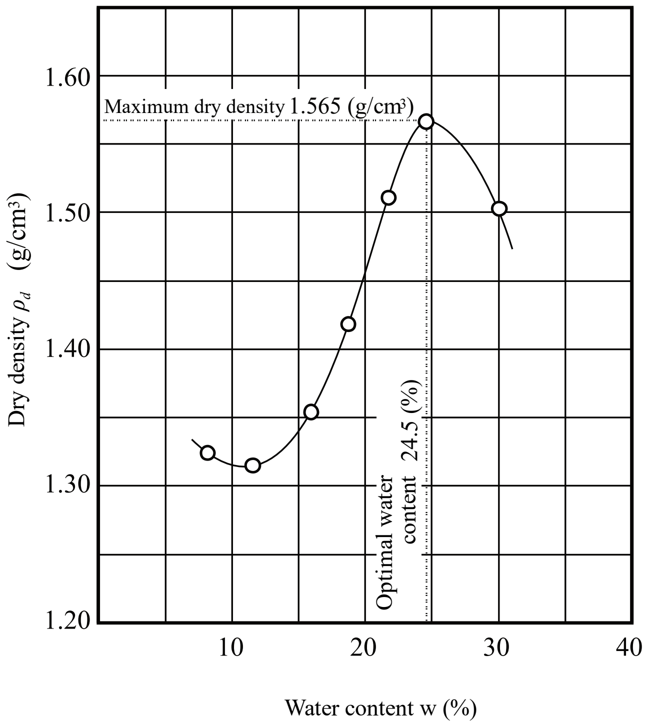


Fig. 3.9.8 Example of a Compaction Curve

(3) Methods for utilizing test results

The test results of compaction tests using a rammer are utilized for understanding the compaction characteristics of the soil in accordance with the site conditions and determining the compaction methods which can maximize the stability of the soil. It is important to determine the standard values for the degrees of compaction and water content at the construction sites because these values can serve as important indexes in the supervision of the compaction work and enhance the compaction work efficiency. Thus, the test results are also utilized for determining the construction standards at the construction sites.

① **Degree of compaction**

How densely soil is compacted can be expressed by the degrees of compaction. The degrees of compaction are the ratios, expressed in percentages, of the in-situ dry density of the subgrade and base course to the maximum dry density obtained through the compaction tests. The general standards of the degrees of compaction are as follows.

- For the landfill sites and earth fill, the degrees of compaction shall be 90% or more of the maximum dry density obtained by Method A, or 85% or more of the maximum dry density obtained by Method D.
- For the subgrade, the degrees of compaction shall be 90 to 95% or more of the maximum dry density obtained by Method A, or 85 to 90% or more of the maximum dry density obtained by Method D.
- For the base course, the degrees of compaction shall be equal to or slightly higher than those of the subgrade.

② **Water content at construction**

The water content at construction shall be expressed in the target ranges to be achieved through actual compaction work employing the compaction methods identical to those used for obtaining the degrees of compaction in compaction tests. In cases that the subgrade and base course have no risk of inundation in the future and require high strength, the water content shall be slightly smaller than the optimal amount. In cases that there is a necessity to ensure stability of an immersed subgrade and base course, the water content shall be equal to or slightly higher than the optimal amount.

There are the following points of caution for compaction tests using a rammer.

- In cases that there is cohesive soil ground with a higher water content at the junction points with existing roads, it is not always possible to achieve compaction with a water content close to the optimal amount. In such cases, the non-repeated compaction method described in above Item (2) shall be conducted with a water content close to the natural and workable ones.
- In many cases, the optimal water content of cohesive soil with a high water content varies depending on the levels of drying treatment. Thus, for this type of cohesive soil, it is preferable to supervise the compaction work not by the degrees of compaction but by the degrees of saturation or air void ratios.

3.9.7 Laboratory CBR Test

(1) Purpose

The laboratory CBR test is conducted to obtain the bearing capacity characteristics of subgrade and base course materials in a manner that uses specimens prepared through compaction of the object soil.

The laboratory CBR test is classified into two types: one using specimens of disturbed soil and the other using specimens of undisturbed soil. For the base course materials, the CBR values shall be obtained through the CBR test by adjusting the specimen close to the optimal water content and modifying it in accordance with the compaction density. Thus, the following section first describes the laboratory CBR test method for the subgrade and base course materials, then describes the method for obtaining modified CBR values for the base course materials.

The laboratory CBR test shall be conducted in accordance with the **Test Methods for the California Bearing Ratio (CBR) of Soils in Laboratory (JIS A 1211)** and the **Survey and Test Manual for Pavement, E 001, Modified CBR Test Method**.[16)]

(2) Laboratory CBR test for subgrade and base course materials

① Purpose

Generally, the soil used for the subgrade and base course is excavated at borrow pits and transported to construction sites; therefore, the soil is in a disturbed state when used for the subgrade and base course at construction sites. The laboratory CBR test using disturbed specimens is, thus, to obtain CBR values as materials for designing the subgrade and base course to be constructed in the same way as described above by testing the specimens representing the conditions of the soil close to the actual conditions at the construction sites. The specimens shall be prepared from soil taken from three locations in each identical layer through auger boring or other methods.

In the case of soil which significantly changes its strength characteristics when disturbed, CBR values obtained through the laboratory CBR test using disturbed specimens are not reliable. A laboratory CBR test using undisturbed specimens is therefore conducted to obtain the CBR values as design materials of the types of soil which are susceptible to disturbances and can be used without being disturbed at sites. The specimens shall be prepared from soil taken from six locations in each identical layer.

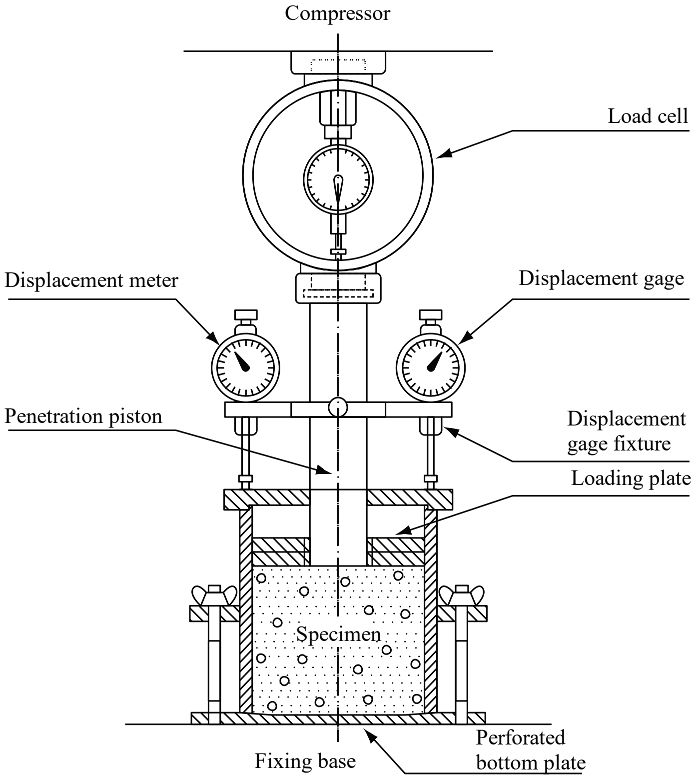
When subjected to increases in water content due to rain or other reasons, the subgrade and base course may show bearing capacity characteristics different from those when tested. Thus, as described in detail below in Item ③ Methods for utilizing test results, penetration test results with specimens immersed in water for four days are normally used as the CBR values. Penetration test results with specimens immersed in water for four days are also used for correcting the results of the in-situ CBR test and the plate loading test. Some types of soil may swell by absorbing moisture and have an adverse effect on the pavement or its surroundings. When using these types of soil, it is necessary to obtain the swelling amount when the water content of the soil is increased.

② Organization of tests and data

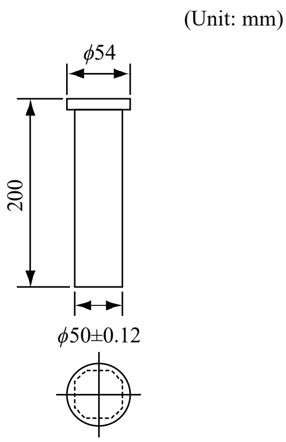
The major difference between tests that use disturbed specimens and those that use undisturbed specimens is the method for preparing the specimens. Furthermore, tests that use disturbed specimens are classified into two types: those that use disturbed soil with an allowable maximum grain size of 19.1 mm and those that use disturbed soil with an allowable maximum grain size of 38.1 mm. The method for preparing the specimens is also the major difference between these two types.

(a) Test equipment

The equipment used in the laboratory CBR test includes a mold, a collar, a perforated bottom plate, a spacer disk, a penetration piston, a loading device, a load cell, a displacement gage, a displacement gage fixture, a perforated plate with a shaft, a loading plate and a specimen extrusion device (as shown in **Fig. 3.9.9**). In addition, when using specimens from undisturbed soil, a cutter needs to be used for preparing the specimens.



a) CBR test equipment



b) Penetration piston

Fig. 3.9.9 Laboratory CBR Test Equipment

(b) Preparation of specimens

1) Case of a specimen of disturbed soil with an allowable maximum grain size of 37.5 mm

A specimen of disturbed soil with an allowable maximum grain size of 37.5 mm can be prepared in the following order.

i. Determination of the optimal water content and maximum dry density

Prior to the preparation of the specimen, its optimal water content and maximum dry density shall be determined through the compaction test specified in **Reference [Part II], Chapter 1, 3.9.6 Compaction Test Using a Rammer**. The compaction method to be used in the test shall follow the requirements for Method E. In addition, the non-repeated method shall be used regardless of whether the dry method or non-dry method is used.

ii. Compaction of specimen

Using the soils left over after the compaction test, the specimen shall be prepared in accordance with the specifications for Method E. After compacting the soil in a mold with a rammer, the mass of the soil and equipment shall be measured after removing any surplus soil above the upper edge of the mold.

2) Case of a specimen of undisturbed soil

A specimen of undisturbed soil can be prepared in the following order.

i. The specimen of undisturbed soil shall be taken in a manner that carefully inserts a mold mounted with a cutter into a place representing the construction site conditions.

ii. When the mold cannot be inserted into the ground, a cylindrical pwith a diameter of about 15 cm shall be carved out from the ground in a manner that excavates the ground around the piece of soil so as not to disturb it, samples the piece of soil after covering it with the mold, and fills the gaps between the piece of soil and the mold with paraffin or other materials.

(c) Test method

In the laboratory CBR test, two types of tests—swelling and penetration tests—shall be conducted.

1) Swelling test

In a swelling test, the specimen shall be immersed in water with a 5 kg weight placed on it through a perforated plate with a shaft, and the swelling amount of the specimen shall be monitored over a course of 96 hours (as shown in **Fig. 3.9.10**).

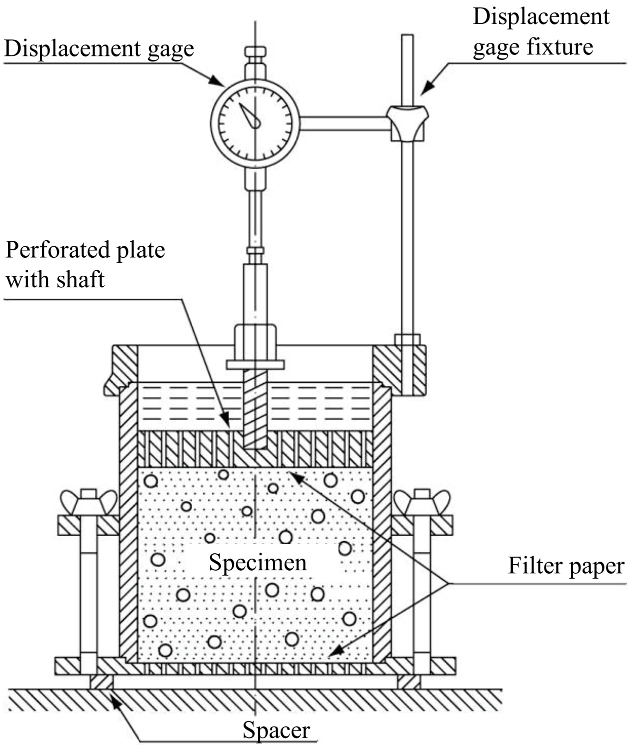


Fig. 3.9.10 Swelling Test

When it is not necessary to take measures for worst scenarios after construction, the swelling test might not be conducted depending on the judgment of the engineers. Furthermore, the swelling test might be conducted without measuring the swelling amounts.

2) Penetration test

The penetration test shall be conducted in the following order.

i. The same weight as that used in the swelling test shall be placed on the specimen.

ii. A loading device shall be set with a small load not more than 49 N in order to bring the penetration piston into contact with the specimen.

iii. The piston shall be penetrated into the specimen at a rate of 1 mm/min by uniformly increasing the load until the penetration amount reaches 12.5 mm, and the load required for achieving 12.5 mm shall be recorded.

iv. The water content of the specimen shall be measured after taken from the mold with the specimen extrusion device.

(d) Organization of test results

For the test results, CBR shall be calculated in the following order.

i. After calculating the load intensity by dividing the applied load by the cross-sectional area of the penetration piston, curves showing relationships between the load intensity and penetration amounts shall be created as shown in **Fig. 3.9.11**. When a curve takes a downward convex shape as is the case with Curve No. 2 in **Fig. 3.9.11**, the curve shall be corrected in a manner that draws a tangent to the curve at the inflection point and makes an intersection between the tangent and horizontal axis as the origin of the axis. Instead of the load intensity, loads can be used to create curves showing the relationships between the loads and penetration amounts.

ii. Using the load intensity corresponding to the penetration amounts of 2.5 and 5.0 mm, which can be read from the created curves that show the relationships between the load intensity and penetration amounts, CBR shall be calculated by the following equation (in the equation, the standard load intensity is the value shown in **Table 3.9.3**).

$$\mathrm{CBR} = \frac{\text{Load intensity} \quad \left(\mathrm{N/mm^2}\right)}{\text{Standard load intensity} \quad \left(\mathrm{N/mm^2}\right)} \times 100\ (\%) \qquad \textbf{(3.9.15)}$$

iii. When the loads are used instead of the load intensity, using the loads corresponding to the penetration amounts of 2.5 and 5.0 mm, which can be read from the created curves showing the relationships between the loads and penetration amounts, CBR shall be calculated by the following equation (in the equation, the standard load is the value shown in **Table 3.9.3**).

$$\mathrm{CBR} = \frac{\text{Load} \quad \left(\mathrm{N}\right)}{\text{Standard load} \quad \left(\mathrm{N}\right)} \times 100\ (\%) \qquad \textbf{(3.9.16)}$$

iv. Generally, the CBR values are those calculated with a penetration amount of 2.5 mm. When CBR calculated with a penetration amount of 5.0 mm are larger than those with 2.5 mm, an additional laboratory CBR test shall be conducted using new specimens. If the additional test results still show larger CBR with 5.0 mm than that with 2.5 mm, that with 5.0 mm is selected as the CBR.

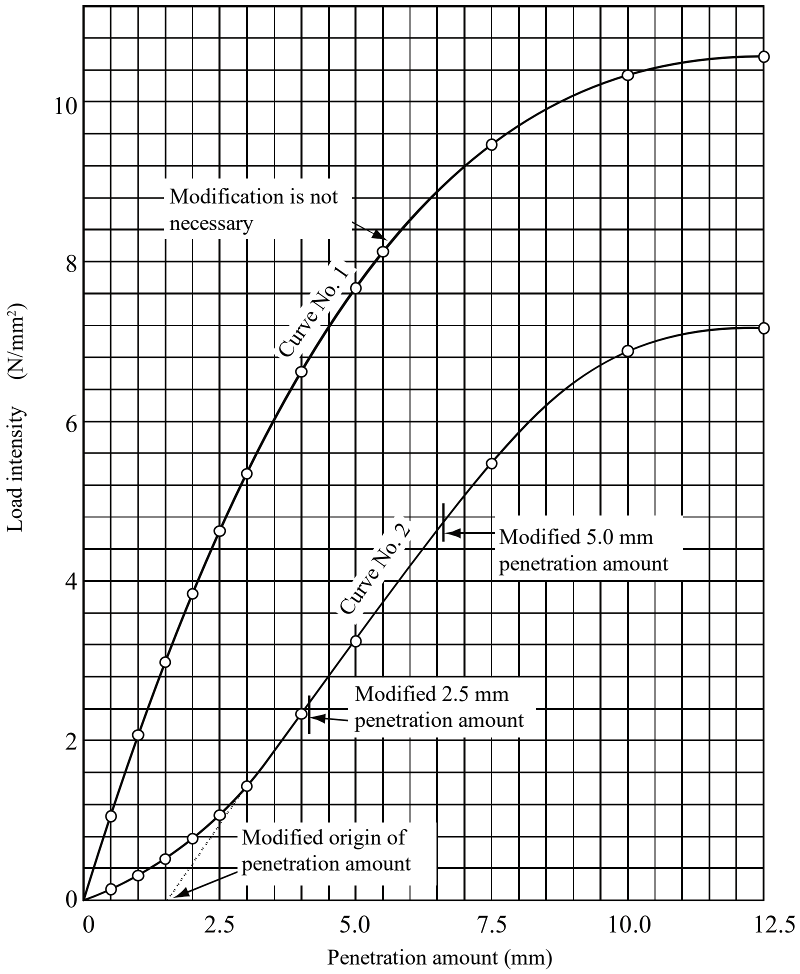


Fig. 3.9.11 Example of Curves Showing the Relationship between the Load intensity and Penetration Amounts in a Laboratory CBR Test

Table 3.9.3 Values of the Standard Load intensity and Standard Loads

Penetration amount (mm)	Standard load intensity (N/mm^2)	Standard load (N)
2.5	6.9	1.34×10^4
5.0	10.3	1.99×10^4
7.5	13.1	2.58×10^4
10.0	15.9	3.12×10^4
12.5	17.9	3.53×10^4

③ **Method for utilizing test results**

The CBR values of the subgrade soil and base course materials obtained through the laboratory CBR test are utilized as design conditions showing the strength of the subgrade and base course when determining the thickness of conventional asphalt pavement. There may be cases of utilizing the CBR test results as the design conditions for pavement used for port roads.

For the pavement of the aprons of quaywalls and shallow draft wharves and the pavement of container yards and port roads, the applicability of the CBR test differs as described below depending on the construction conditions at the sites in terms of the states of the soil and construction methods.

- CBR test for general subgrade soil

  A laboratory CBR test with specimens of disturbed soil (with an allowable maximum soil particle size of 38.1 mm) shall be conducted. In this case, the specimens for the penetration test shall be prepared in a manner that removes aggregate of 40 mm or larger from the object soil, places the object soil with natural water content in three layers in molds with each layer compacted 67 times with a rammer, and immerses the object soil in water for four days.

- CBR tests in cases that the existing soil is used as a subgrade without being disturbed and is known to have CBR values with specimens of disturbed soil significantly lower than those with specimens of undisturbed soil

  A laboratory CBR test with specimens of undisturbed soil shall be conducted. In this case, the specimens for the penetration test shall be prepared in a manner that immerses them in water for four days.

- CBR test for completed subgrade

  The in-situ CBR test shall be conducted in the season when the subgrade soil is considered to be in its wettest state. When the CBR values are obtained when the subgrade soil is not considered to be in its wettest state, the values corrected by the following equation can be used as the CBR values.

$$\text{CBR (modified)} = \text{In-situ CBR} \times \frac{\begin{array}{l}\text{CBR of a specimen of undisturbed soil}\\ \text{(immersed in water for 4 days)}\end{array}}{\begin{array}{l}\text{CBR of a specimen of undisturbed soil}\\ \text{(natural water content)}\end{array}} \qquad \textbf{(3.9.17)}$$

(3) Test for obtaining the modified CBR for the base course materials

① **Purpose**

The test is conducted to obtain the modified CBR values for the base course materials used for asphalt pavement.

The test shall be conducted in accordance with the **Survey and Test Manual for Pavement, E 001, Modified CBR Test Method**.[16)]

② **Organization of tests and data**

(a) Test equipment

The equipment to be used shall be the same as that described in **Reference [Part II], Chapter 1, 3.9.7 (2) Laboratory CBR test for subgrade and base course materials**.

(b) Preparation of specimens

The specimens shall be prepared in the following order.

i. Determination of the optimal water content and maximum dry density

   Prior to the preparation of the specimens, the optimal water content and maximum dry density of the specimens shall be determined through the compaction test specified in **Reference [Part II], Chapter 1, 3.9.6 Compaction Test Using a Rammer**. The compaction method to be used in the test shall follow the requirements for Method E.

ii. Compaction of specimens

   Three groups of specimens shall be prepared, with each group consisting of three specimens with the object soil put in molds and will be subjected to compactions 92, 42 and 17 times, respectively.

(c) Test method

Each specimen shall be used for measuring the dry density and the CBR value after a four-day immersion in water.

(d) Organization of test results

The results of the CBR test shall be organized in the same way as described in **Reference [Part II], Chapter 1, 3.9.7 (2) Laboratory CBR test for subgrade and base course materials**.

③ **Method for utilizing test results**

The modified CBR values shall be obtained in the following order.

i. A curve showing the relationship between the CBR values and dry density obtained from the averages of three specimens for each number of compaction times and a curve showing the relationship between the water content and dry density shall be created with the dry density on the vertical axis, and the water content as well as the CBR values on the horizontal axis, as shown in **Fig. 3.9.12**.

ii. A modified CBR value can be obtained using the two curves in a manner that first identifies the intersection of the horizontal line passing through the dry density corresponding to the prescribed degree of compaction with the curve showing the relationship between CBR and dry density, and then reads CBR corresponding to the point where the vertical line drawn from the intersection intersects with the horizontal axis.

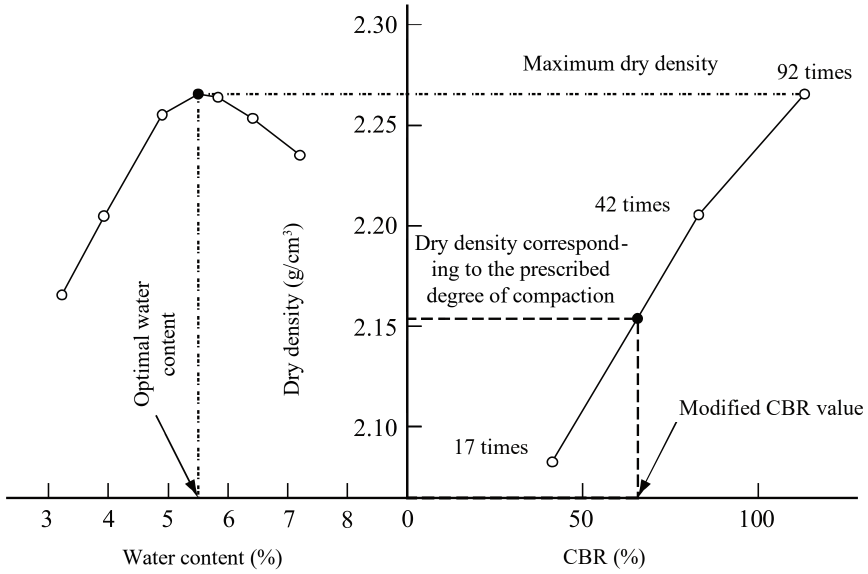


Fig. 3.9.12 Method for Obtaining the Modified CBR Value Corresponding to the Prescribed Degree of Compaction (The number of times in the figure denotes the number of compactions)

3.9.8 Other Tests

Other in-situ tests include the simplified bearing test, frost penetration depth survey and void survey. Other laboratory tests include the unconfined compression test and resilient modulus test.

(1) Simplified bearing test

The simplified bearing test enables the bearing capacity of the constructed subgrade and base course to be easily obtained, and is classified into the simplified penetration test, the test using a simplified bearing capacity meter, and the test using a compact falling weight deflectometer (FWD).

In the simplified penetration test, the bearing capacity is obtained in a manner that applies the load repeatedly to the cone set on the ground by a free dropping weight from a predetermined height, and measures the number of drops required until the penetration depth of the cone in the ground reaches the predetermined depth. The simplified penetration test shall be conducted in accordance with the **Survey and Test Manual for Pavement, S 043-1, Simplified Penetration Test**.[17)]

In the test using a simplified bearing capacity device, the impact value (a value converted from the measured impact acceleration) of a free dropping rammer is measured in a manner that sets the test device with the rammer having a predetermined mass on the ground and allows the rammer to free fall from a predetermined height. The test shall be conducted in accordance with the **Survey and Test Manual for Pavement, S 043-2 T, Test Method Using a Simplified Bearing Capacity Meter**.[18)]

In the test using a compact falling weight deflectometer, a maximum load and maximum deflection at the load center are measured in a manner that a weight having a predetermined mass falls freely from a predetermined height onto the loading plate on the ground. The test shall be conducted in accordance with the **Survey and Test Manual for Pavement, S 043-3 T, Test Method Using a Compact Falling Weight Deflectometer (FWD)**.[19)]

(2) Frost penetration depth survey

In cold regions, the ground freezes, sucking the moisture in the ground toward the frozen surface and creating ice lenses in the soil. The increase in volume due to the freezing causes the ground to swell, which is called the frost heave phenomenon. This phenomenon creates adverse effects on pavement in the form of irregularities and damage to the surfaces. The frost penetration depth survey is to measure the depth of the ground affected by this phenomenon. There are several methods for conducting the surveys including ground temperature distribution measurements through boreholes, visual inspections of the frozen state of the ground, measurements with a methylene blue frost depth meter, and measurements with a thermocouple and resistance thermometer.

(3) Void survey

Earthquakes and sand washing-out may cause the subgrade and base course to develop voids. The void survey is to identify the locations and sizes of the voids without destroying the pavement. The void survey methods include nondestructive investigations using a ground-penetrating radar (vehicle mounted and hand cart types) and borehole cameras.

The void survey with a ground-penetrating radar detects voids using the difference in the specific inductive capacity between the pavement materials and voids. Generally, the locations with suspected voids are identified through a vehicle mounted radar and the suspected voids are inspected in detail through a hand cart radar. The void survey using a borehole camera is to confirm the thicknesses and depths of the voids by imaging the excavated sections with a borehole camera inserted into the ground through holes provided on the pavement surfaces. The void survey with a borehole camera is generally used for verifying the void detection results by the ground-penetrating radar. In addition, the **2014 Specifications for Pavement**[20) can be used as a reference.

There is a method for surveying the presence or absence and the states of interlayer separation using thermal infrared technology for the base course (including surface and base layers) made of asphalt mixtures.[21) The method is used for measuring the temperature of pavement surfaces by taking thermal infrared images using a pavement surface temperature measuring device (thermal infrared camera) which satisfies the predetermined standards. The suspected areas with interlayer separation can be identified as those places having a pavement surface temperature lower than the surroundings (in the case of night-time surveys) or higher than the surroundings (in the case of day-time surveys).

(4) Unconfined compression test

The unconfined compression test is conducted for obtaining the unconfined compression strength of slag or stabilized base course materials using specimens compacted at the sites or in laboratories. Although there is a method for estimating the in-situ shear strength based on the unconfined compression strength measured with undisturbed specimens taken from the original ground and making estimates, such a method is not normally used for the subgrade and base course. The unconfined compression test shall be conducted in accordance with the **Survey and Test Manual for Pavement, E 013, Unconfined Compression Test Method for Stabilized Mixtures**[22) and **E 003, Unconfined Compression Test Method for Steel Slag**.[23)

(5) Resilient modulus test

The resilient modulus test is conducted for setting the material constants of the subgrade and base course required when designing pavement structures. Specifically, in the test, the material constants are obtained through a cyclic indirect tensile test and cyclic triaxial compression test. However, there are still many points about the resilient modulus test which have not been elucidated yet and it is necessary to examine the test conditions and the applicable range of the pavement materials. In addition, the test shall be conducted in accordance with the **Survey and Test Manual for Pavement, E 016, Resilient Modulus Test Method for Subgrade and Base Course Soil**.[24)

3.10 Pile Loading Tests

3.10.1 General

It is rational to conduct pile loading tests and design pile foundations based on the test results. If it is difficult to conduct the pile loading tests when designing the pile foundations for any reason, the pile loading tests shall be conducted during construction to confirm that the pile bearing capacity satisfies the design values.

When conducting pile loading tests, it is necessary to clarify the purposes of the tests and the items to be tested, and to conduct the tests through the appropriate test plans and methods.

3.10.2 Loading Test Purposes

Loading tests are mainly classified into ‘the characteristic investigation test’ and ‘the confirmation test.’ The characteristic investigation test is conducted prior to designing the structures in order to determine the characteristic values of the parameters related to the pile bearing capacity necessary to the design. In contrast, the confirmation test is conducted during construction to confirm that the pile bearing capacity satisfies the design values. The confirmation test is generally conducted when the structures are designed based on the empirical estimate equations and may be conducted to confirm the influences of the ground and structures, taking into consideration their peculiarities.

The supervision of the piling work based on dynamic bearing capacity management equations such as Hiley’s formula (refer to **Reference [Part II], Chapter 1, 3.10.10 Dynamic Bearing Capacity Management Equations**)can be considered to be a type of pile loading test in a broad sense. However, because the accuracy of the dynamic bearing capacity management equations is very low, it is not appropriate to estimate the pile bearing capacity only from the calculation results using the equations. Furthermore, the use purposes of these quations is limited to the supervision of the piling work. For example, the dynamic bearing capacity management equations can be used for confirming the presence or absence of abnormalities in the pile bearing capacity due to the unevenness of the bearing layers through relative changes in the calculation results when using the equations.

3.10.3 Types and Outlines of Loading Tests

(1) Types of loading tests

Depending on the loading directions, pile loading tests are classified into vertical pile loading tests[25)] and horizontal pile loading tests.[26)] In vertical pile loading tests, the piles are subjected to loads in their axial directions. The vertical pile loading tests are further classified into those mainly for testing pile resistance against a pushing force in axial directions and those mainly for testing pile resistance against a pulling force in axial directions. In contrast, in horizontal pile loading tests, the piles are subjected to loads in a direction perpendicular to the axial directions.

The main types of vertical pile load tests for testing pile resistance against a pushing force in axial directions include the pushing test, base loading test, rapid loading test and impact loading test. Among these tests, the pushing and base loading tests apply static loads to the piles. The difference between the pushing and base loading tests is the loading points; in the pushing test, loads are applied to the pile heads, while in the base loading test, loads are applied to the piles through jacks installed around the pile tips. Furthermore, in the base loading test, portions of the piles above and below the jacks are subjected to loads acting in the direction pushing up and pushing down on the piles, respectively. The rapid loading and impact loading tests apply dynamic loads to the piles. The difference between the two tests is the loading time in that the relative loading time T_r (refer to **Reference [Part II], Chapter 1, 3.10.6 Rapid Pile Loading Test**) is longer ($5 < T_r < 500$) in the rapid loading test, and shorter ($T_r < 5$) in the impact loading test. Here, when referring to the existing materials and literature, it is necessary to pay attention to the fact that the pushing and impact loading tests have been called the vertical loading and dynamic loading tests, respectively, in the past.

Among the vertical pile loading tests, those mainly for testing pile resistance against a pulling force in axial directions have been standardized as the pile pulling test. In the pile pulling test, the piles are subjected to static loads applied in the direction they are pulled around the pile heads.

Among the horizontal pile loading tests, only the method that applies static loads to the vicinity of the pile heads has been standardized.

The characteristics of each loading test are summarized in **Table 3.10.1**. The items (2) to (8) below describe the loading test methods and dynamic bearing capacity management equations.

Although sufficient knowledge has not been obtained for the applicability of the loading tests to batter piles, the results of the vertical pile loading tests have often been used in the examination of the resistance against the axial force of batter piles having inclination angles with respect to a vertical direction of up to 20 degrees, which have been widely used in port facilities. In addition, for the resistance of batter piles against forces perpendicular to axial directions, there is a method for correcting the lateral resistance coefficients obtained through the horizontal pile loading tests in accordance with the inclination of the batter piles (refer to **Part III, Chapter 2, 3.4.8 Calculation of the deflection of piles though the PHRI method”**).

Table 3.10.1 Characteristics of the Loading Test Methods

Type of loading test		Loading direction	Loading position	Loading type
Vertical loading test	Pushing test	Vertical pushing	Pile head	Static
	Base loading test	Vertical pushing down/up	Around pile tip	Static
	Rapid loading test	Vertical pushing	Pile head	Dynamic
	Impact loading test	Vertical pushing	Pile head	Dynamic
	Pulling test	Vertical pulling	Vicinity of pile head	Static
Horizontal loading test		Horizontal	Vicinity of pile head	Static

(2) Outline of the pile pushing test

The pushing test is a method that applies static loads to the pile heads and is considered to be the most standard vertical pile loading test method. Examples of the pushing test equipment and a situation involving the pushing test conducted at sea are shown in **Figs. 3.10.1** and **3.10.2**, respectively.

In the pushing test, the resistance against a pushing force in axial directions can be obtained by measuring the loads applied to the pile heads and the displacement at the pile tips. By measuring the axial force with strain indicators attached to the pile bodies, the base resistance and skin friction of the piles when they are subjected to an axial pushing force can be obtained separately. Furthermore, the changes in skin friction in the depth direction can be obtained by measuring the changes in axial force in the piles with strain indicators attached to the pile bodies at different depths.

Depending on the loading methods, the pushing test time ranges from 30 minutes (in the case of a single-cycle method) to four hours or more (in the case of a multi-cycle method). In most cases, the pushing test can be completed in a day. However, because of the necessity of preparing reaction piles and using large-scale loading equipment, it takes a few to 10 days for assembling and dismantling the loading equipment. Thus, among the vertical pile loading tests, the pushing test requires the longest test time and high costs. The pushing test also requires large yards with enough overhead clearance for installing reaction piles and reference beams.

For the details of the pushing test, refer to **Reference [Part II], Chapter 1, 3.10.5 Pile Pushing Test.**

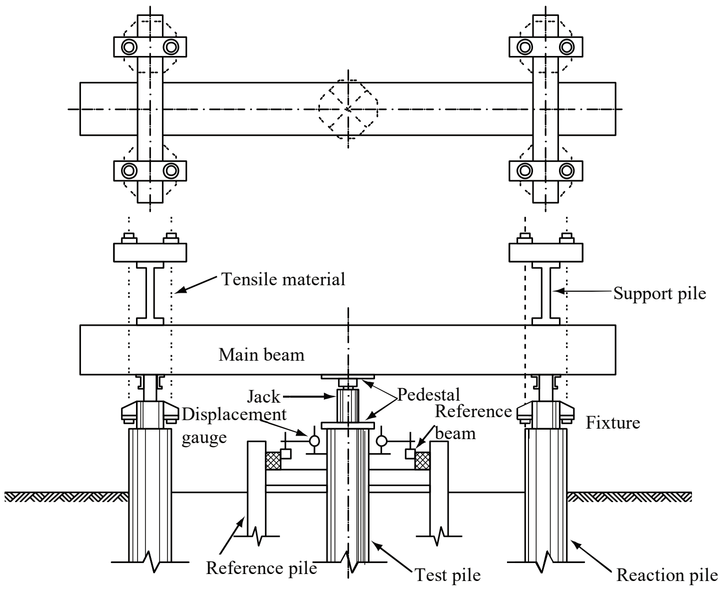


Fig. 3.10.1 Example of the Equipment for the Pile Pushing Test[27)]

Fig. 3.10.2 Example of a Pile Pushing Test at Sea[28)]

(3) Outline of the pile base loading test

The pile base loading test is a method that applies static loads to piles through jacks embedded at the pile tips in a manner that uses the resistance of the piles below the jacks (mainly base resistance) and above the jacks (mainly skin friction) as a reaction force of the static loads. The portions of the piles above and below the jacks are subjected to the actions of the force pushing up and pushing down on the piles, respectively. **Fig. 3.10.3** shows an example of the pile base loading test equipment.

In the base loading test, the base resistance and skin friction of the piles can be obtained by measuring the displacement of the pile heads and the upper faces of the jacks and pile tips. Because it is possible to apply large loads directly to the pile tips, the base loading test allows for the base resistance to be measured with a high accuracy. However, because the base loading test uses the base resistance and skin friction of the piles as a reaction force, as described above, the applicable static loads are capped when either of the reaction forces reaches the ultimate value. Thus, when conducting the base loading test, it is necessary to establish test plans with due consideration given to this fact.

Although the base loading test does not require reaction piles, supplementary reaction devices may be required when the skin friction is smaller than the base resistance, as is the case with short piles. Furthermore, the base loading test takes about the same amount of time as the pushing test, but the time required for the base loading test to assemble and dismantle the loading equipment for the base loading testis about half a day to one day, respectively, which is shorter than for the pushing test.

Recently, there has been an increasing number of cases of applying the base loading test to onshore foundation piles, particularly cast-in-place concrete piles, for the bridges of port roads. However, the base loading test cannot be applied to driven steel pipe piles frequently used for mooring facilities because the jacks cannot be embedded at the pile tips. Thus, the base loading test is excluded from the descriptions in the section below. For the details of the base loading test, refer to the **Methods for Vertical Pile Loading Tests and Commentaries**.[29)]

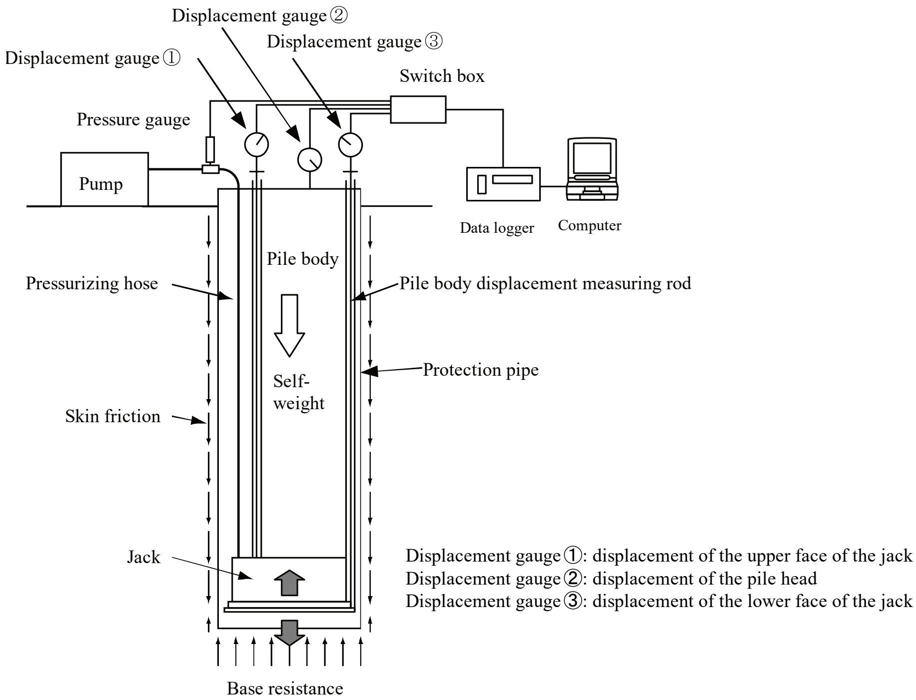


Fig. 3.10.3 Example of the Equipment for the Pile Base Loading Test[29)]

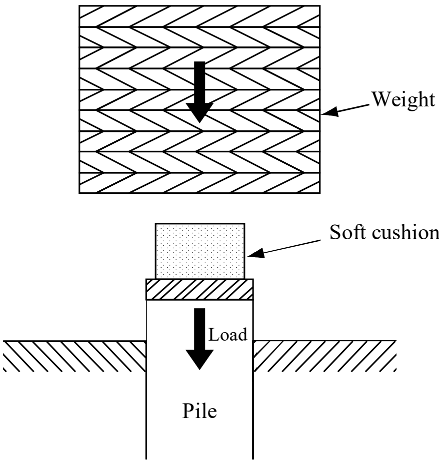
(4) Outline of the rapid pile loading test

The rapid pile loading test is a method that applies dynamic loads to the pile heads with weights dropped on them using special cushions (soft cushion weight fall method). **Fig. 3.10.4** shows an example of the rapid pile loading test equipment.

In the rapid pile loading test, the resistance against a pushing force in axial directions can be obtained. By measuring the axial force with strain indicators attached to the pile bodies, the base resistance and skin friction of the piles when subjected to an axial pushing force can be obtained separately.

The rapid loading test requires a short test time; about half a day to one day depending on the number of cycles. In addition, it takes about half a day to three days, respectively, for assembling and dismantling the loading equipment. Because reaction piles are not required, the rapid loading test is less expensive than the pushing test. However, in the case of the offshore rapid loading test, stages are required to install the test equipment (as shown in **Fig. 3.10.4**).

In addition to the soft cushion weight fall method, the rapid loading test can be conducted through another method called the reaction body inertia force method. For the details of the rapid loading test, refer to **Reference [Part II], Chapter 1, 3.10.6 Rapid Pile Loading Test.**



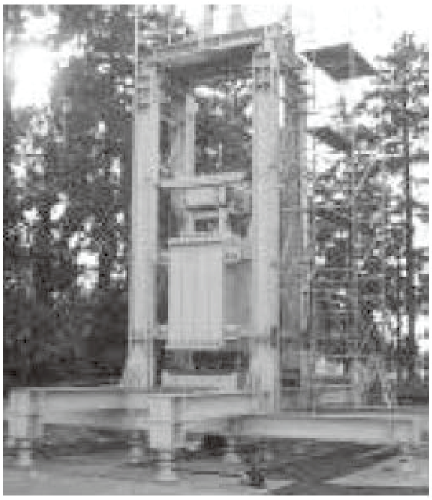


Fig. 3.10.4 Example of the Equipment for the Rapid Pile Loading Test[30)]

(5) Outline of the impact pile loading test

The impact pile loading test is a method that applies dynamic loads to the pile heads. In most cases, hydraulic hammers for driving piles are also used as loading equipment. Thus, the procedures required for the impact test are close to general pile driving work (refer to **Fig. 3.10.5**).

In the impact pile loading test, waveform matching analyses based on the one-dimensional kinematic wave theory need to be conducted using measured data after the impact loading test in order to obtain the pile resistance against a pushing force in axial directions. These analyses allow the base resistance and skin friction to be obtained separately when the piles are subjected to a pushing force.

The impact loading test requires a test time of a few to several dozen minutes and can be completed within half a day when using hammers used for pile driving as the loading equipment. When using weight fall equipment as the loading equipment, it takes about two to five days to complete the impact loading test. When using hammers used for pile driving as the loading equipment, the test can be conducted at extremely low costs. However, it shall be noted that there may be cases where the loading capacity of these hammers is not sufficient for applying the test loads to the piles.

For the details of the impact loading test, refer to **Reference [Part II], Chapter 1, 3.10.7 Impact Pile Loading Test.**

Fig. 3.10.5 Example of an Actual Implementation of the Impact Pile Loading Test[31)]

(6) Pile pulling test

The pile pulling test is a method that applies static pulling loads to the pile heads to investigate the pile resistance against a pulling force in axial directions. **Fig. 3.10.6** shows an example of the pulling test equipment.

In the pulling test, the pile resistance against a pulling force in axial directions can be obtained from the relationship between the pile head displacement and the pulling loads. The strain indicators attached on the pile bodies enable the distribution of skin friction in the depth direction when the piles are subjected to a pulling force to be measured.

Depending on the loading methods, the pulling test takes 30 minutes (in the case of a single-cycle method) to four hours or more (in the case of a multi-cycle method). In most cases, the pulling test can be completed in a day. As is the case in the pile pushing test, the pulling test requires reaction equipment (reaction piles in many cases) and large-scale loading equipment, which take a few to 10 days to be assembled and dismantled. Due to having a smaller base resistance than the pushing test (negative pressure only), the pulling test can be implemented with small planned maximum loads, thereby making the test cost less than the pushing test in many cases.

For the details of the pulling test, refer to **Reference [Part II], Chapter 1, 3.10.8 Pile Pulling Test**.

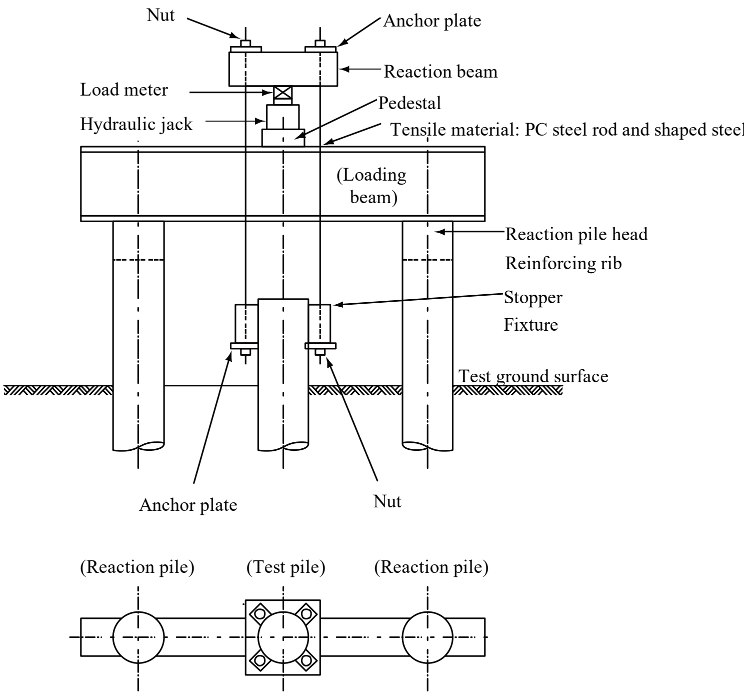


Fig. 3.10.6 Example of the Equipment for the Pile Pulling Test[32)]

(7) Horizontal pile loading test

The horizontal pile loading test is a method that applies static horizontal loads to the piles. **Fig. 3.10.7** shows an example of the horizontal pile loading test equipment. In the horizontal loading test, the behavior characteristics of the piles when subjected to the actions of horizontal loads are investigated by measuring the distribution of the loaded weight, pile head displacement and bending moment on the piles in the depth direction. The behavior of piles subjected to horizontal loads is largely affected by the dimensions of the piles, ground conditions, loading conditions, pile head fixing conditions and various other conditions. Thus, the behavior of the test piles obtained through the loading tests is barely consistent with that of actual piles, and it is difficult to examine the bearing capacity of piles in directions perpendicular to the pile axes only with the horizontal loading test results. In many cases, the moduli of subgrade reaction are obtained through inverse operations based on the horizontal loading test results for use in the pile design.

Depending on the loading methods, the horizontal loading test takes 30 minutes (in the case of a single-cycle method) to four hours or more (in the case of a multi-cycle method). In most cases, the horizontal loading test can be completed in a day. The horizontal loading test requires reaction equipment which takes a few to 10 days to be assembled and dismantled.

For the details of the horizontal loading test, refer to **Reference [Part II], Chapter 1, 3.10.9 Horizontal Pile Loading Test.**

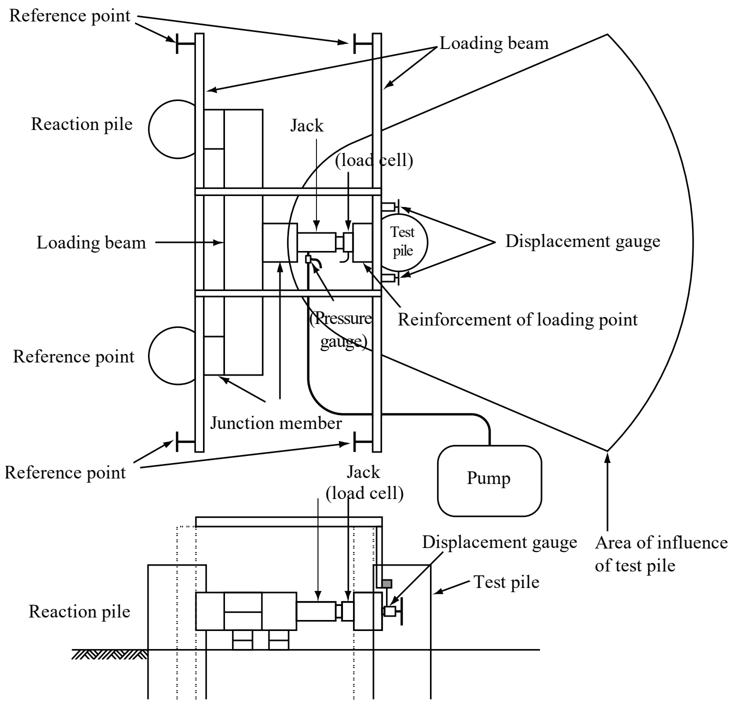


Fig. 3.10.7 Example of the Equipment for the Horizontal Pile Loading Test[33)]

(8) Dynamic bearing capacity management equations

Dynamic bearing capacity management equations are used to calculate the maximum static axial resistance of the piles from the input hammer energy, rebound rates and penetration lengths when driving the piles with hammers. Hiley’s formula is a typical example of the dynamic bearing capacity management equation. Because the accuracy of these quations is very low, they shall only be used for the supervision of piling work.

The data necessary for the dynamic bearing capacity management equations can be obtained along with general pile driving work. Therefore, the equations do not generally require special equipment or additional costs.

For the details of the dynamic bearing capacity management equations, refer to **Reference [Part II], Chapter 1, 3.10.10 Dynamic Bearing Capacity Management Equations.**

3.10.4 Establishment of Test Plans

(1) Setting test items

In regard to the purposes of the pile loading tests, they are largely classified into the characteristic investigation test and confirmation test (refer to **Reference [Part II], Chapter 1, 3.10.2 Loading Test Purposes**). When establishing the test plans, it is necessary to clarify which test is to be conducted.

The items to be investigated in the characteristic investigation test are the various characteristic values concerning the bearing capacity of piles for use in design. The characteristic values concerning the pile bearing capacity for use in design include the pile resistance against a pushing force in axial directions, a pulling force in axial directions and a force in the directions perpendicular to the pile axes. However, these characteristic values can be rarely obtained directly through pile loading tests. Thus, the test items of the characteristic investigation test are generally the characteristic values of the parameters, such as the surface friction per unit contact area between the piles and ground as well as the moduli of subgrade reaction, to be the bases for calculating the characteristic values for use in the design.

In contrast, the confirmation test is conducted during construction to confirm that the values of the pile bearing capacity satisfy the design values. The confirmation test may be conducted to confirm the influences of the ground and structures while taking into consideration their peculiarities. In either case, the items requiring confirmation are the test items of the confirmation test.

(2) Selection of piles for loading tests

The pile loading tests are conducted with either test piles prepared exclusively for the loading tests or the actual piles already constructed. When the loading tests are conducted for the characteristic investigation, the test piles are generally used for pile loading tests because the design of the structures is not finalized and there are no actual piles when the characteristic investigation tests are conducted. In contrast, the piles subjected to loading tests for the confirmation test are generally selected from actual piles while taking into consideration the order and places of the pile driving work and the ground and loading conditions.

The points of caution when preparing test piles for the loading tests are as follows. Here, when using actual piles for the loading tests, there are many cases of preparing actual piles with specifications partially modified for the loading tests in order to prevent the actual piles from being damaged by the concentration of stress on the pile bodies during the tests. In such cases, due consideration shall also be given to the following points of caution.

① **Pile dimensions and lengths embedded in bearing layers**

Due consideration shall be given to the dimensions such as the diameters, overall lengths and wall thicknesses as well as the lengths embedded in the bearing layers because these are the factors that largely affect the pile bearing capacity. Particularly, in the case of steel piles with open tips frequently used for port facilities, the closed area ratios of the pile tips largely vary depending on the pile diameters. Thus, the base resistance of piles having diameters different from the test piles cannot be evaluated with the loading test results of the test piles.

Even in the case of design values which are not considered to be subjected to pile diameters, such as the skin friction per unit contact area between the piles and ground as well as the ground lateral resistance coefficients used in the PHRI method, significant differences in the pile diameters may cause unexpected effects (for example, the changes in the ground areas influenced by the construction and the relative relationships between the gravel diameters of gravel layers and pile diameters).Therefore, it is necessary to fully examine the values of the dimensions and lengths embedded in the bearing layers of the test piles by referring to the ground survey results and estimation equations of the existing pile bearing capacity so as to match these values with those that are expected for the actual piles to the extent possible. In the case of significant differences between the designed values of dimensions and lengths embedded in the bearing layers of actual piles, which are determined based on the loading test results, and the values of the test piles, appropriate measures shall be taken in a manner that conducts either additional loading tests or confirmation tests during construction.

② **Stress generated in test pile bodies**

It is necessary to be careful not to damage the pile bodies during the loading tests when they are conducted for obtaining test items such as the characteristic values subjected to ground properties among the characteristic values for use in design; i.e., the characteristic values of the pile resistance against a pushing force in axial directions or the characteristic values of the base resistance and skin friction to be the bases for calculating those of the pile resistance against a pushing force in axial directions.

For example, the second limit resistance determined by ground failure phenomena (refer to **Reference [Part II], Chapter 1, 3.10.5 Pile Pushing Test**) is generally used as the characteristic value of the pile resistance against a pushing force in axial directions. In the pile design, the design values of the pile resistance against a pushing force in axial directions are determined by applying a safety allowance to the above characteristic values. In examining the stresses in the pile bodies, it is necessary to determine the materials and wall thicknesses so as to ensure that the stresses generated in the pile bodies can be kept within allowable safety ranges even when the piles are subjected to the actions of an axial force corresponding to the design values of the pile resistance against a pushing force in axial directions. It shall be noted that because there is a difference in the design concept between the safety allowance with respect to the resistance against a pushing force in axial directions and the safety allowance with respect to damage due to the stresses generated in the pile bodies, the pile bodies become damaged (before the ground undergoes failure phenomena) when loads corresponding to the characteristic values of the resistance against a pushing force in axial directions are applied to the pile heads.

Particularly, in the case of the rapid loading and impact loading tests, which apply dynamic loads to the piles, it shall be noted that the test loads are larger than in the case of static loading tests (refer to **Reference [Part II], Chapter 1, 3.10.6 Rapid Pile Loading Test** and **Reference [Part II], Chapter 1, 3.10.7 Impact Pile Loading**

Test) because the piles are subjected to the actions of dynamic resistance in addition to the static resistance of the ground.

In order to prevent the pile bodies from being damaged during the loading tests, it is necessary to change the materials of the test piles or increase the wall thicknesses. However, when increasing the wall thicknesses, due consideration shall be given to the influence of the differences in the dimensions on the pile bearing capacity as described in item ① above.

③ **Construction methods**

Pile construction methods largely affect the pile bearing capacity. Thus, it is necessary to construct the test piles through the methods expected to be used for constructing the actual piles. When the construction methods need to be changed, additional loading tests shall be conducted accordingly.

④ **Differences between design and test conditions**

When conducting loading tests with test conditions different from the design conditions (e.g., the existence of soil layers which are planned to be excavated or whose resistance is not considered in the design because the layer shave a possibility of liquefaction or consolidation settlement), the influence of the existence of these types of soil layers shall be examined during the loading tests. To that end, it is necessary to reliably install sensors such as strain indicators at positions on the test piles corresponding to the depths of the boundaries of the soil layers so as to measure the skin friction and bending moment.

(3) Ground survey plans

Pile loading tests are generally conducted at limited locations at construction sites. Thus, when applying the loading test results from limited locations to the design and construction of piles at neighboring locations with different ground conditions, it is important to associate the pile loading test results with the ground survey results. Furthermore, when obtaining the skin friction of each soil layer, it is necessary to install strain indicators at each layer boundary, and, therefore, it is important to determine the installation depths. To that end, ground surveys shall be conducted at areas close to the locations of the pile loading tests. Because the properties of the ground deeper than the pile tips affect the characteristics of the pile bearing capacity, particularly the base resistance of the piles, ground surveys shall be conducted at depths deeper than the pile tips by three to five times the pile diameters.

For designing piles for port facilities, it is necessary to conduct at least the standard penetration test in the case of sandy ground and the specimen sampling and uniaxial compression test in the case of cohesive soil ground. In addition to the above, the ground surveys shall include the examination of the engineering properties of the soil layers through physical and mechanical tests. Specifically, it is necessary to fully investigate and understand the engineering properties of the intermediary and bearing layers, which affect the determination of the pile bearing capacity and construction methods.

It is possible to use the results of previous ground surveys at nearby areas when they are available. However, considering the risk that the pile tips may not reach the bearing layers because of a possible unevenness of the layers, possible differences in ground properties between the locations of the loading tests and ground surveys, and ambiguities in determining the depths to install strain indicators and evaluating the loading test results, ground surveys shall be conducted again at locations close to the test piles as much as possible.

(4) Selection of loading test methods

The pile loading test methods shall be selected so that the loading directions (refer to **Table 3.10.1**) are essentially consistent with the items to be investigated in the loading tests. That is, for the investigation of the pile resistance against a pushing force in axial directions, the methods to be selected shall be the pushing, rapid loading or impact loading test. For the investigation of the pile resistance against a pulling force in axial directions, the method to be selected shall be the pulling test. Similarly, for the investigation of the pile resistance against a force perpendicular to the pile axes, the method to be selected shall be the horizontal loading test. The outlines of each method are as described in **Reference [Part II], Chapter 1, 3.10.3 Types and Outlines of Loading Tests.** Although the base loading test is excluded from the descriptions in this section because it cannot be applied to the steel piles with open tips frequently used for port facilities, it can be considered as one of the applicable loading test methods in the case of testing onshore cast-in-place piles (refer to **Reference [Part II], Chapter 3.10.3 (3) Outline of the pile base loading test**).

Three types of loading test methods—the pushing, rapid loading and impact loading tests—can be applied to the investigation of the pile resistance against a pushing force in axial directions. When selecting the loading test methods, due consideration shall be given to the purposes, items, restrictions (yard areas, etc.), periods and costs of the tests. On such occasions, it is preferable to examine the possibility of combining multiple loading test methods in accordance with the necessary test items. Generally, among the three methods, the pushing test can produce the most accurate test results and apply the largest loads to the piles. In contrast, because the results of the rapid loading test are subjected to the inertia force of the pile bodies, and those of the impact loading test are subjected to wave phenomena in addition to the inertial force of the pile bodies, the accuracy of the test results is slightly inferior to that of the pushing test. Furthermore, when using pile driving hammers as loading equipment in the impact loading test, there are many cases of the hammer capacity being too small to confirm the characteristic values of the pile resistance against a pushing force in axial directions. However, the impact loading test is generally the test that is most advantageous in terms of the test periods and costs, followed by the rapid loading test. Because each test method has advantages and disadvantages, it is preferable to take a wide-range approach to the selection and combination of loading test methods in accordance with the test items, number of test locations and test conditions.

(5) Setting the loading test conditions

The items to be examined as loading test conditions are the maximum loads to be applied to the piles, test equipment, loading methods, measuring items during the loading tests, measuring methods and arrangement of the measuring equipment. Because the methods for setting the loading test conditions vary depending on the loading test methods and test purposes, refer to the details of each loading test method described in **Reference [Part II], Chapter 1, 3.10.5 to 3.10.9**.

(6) Setting the curing periods

Appropriate curing periods shall be set for the test piles from their construction to the implementation of the loading tests. Because the impacts during the construction of the test piles generate excess pore water pressure and loosen the ground, it is necessary to conduct the loading tests after waiting for the loosened ground to be restored. In the **Methods for Vertical Pile Loading Tests and Commentaries,**[34)] the target curing periods are recommended to be five days or more and 14 days or more after the construction of the piles in the case of sandy ground and cohesive ground, respectively. However, considering the curing periods additionally required for cast-in-place concrete piles and problems with the hammer capacity associated with the impact loading test (refer to **Reference [Part II], Chapter 1, 3.10.7 Impact Pile Loading Test**), it is necessary to set the curing periods based on not only the ground conditions but also other conditions.

3.10.5 Pile Pushing Test

(1) Test outline

The pushing test, which is a static test that applies loads to the pile heads through hydraulic jacks, is the most standard loading test among the vertical pile loading tests. When designing pile foundations for port structures and road bridges subjected to dynamic loads, such as earthquakes and waves, these dynamic loads are generally converted into static loads in the design calculation. Thus, the results of the pushing test directly correspond to the characteristic values of the pile resistance against a pushing force in axial directions. However, it shall be noted that the loading time of the pushing test is significantly shorter than the loading time of the actual structures, and, therefore, the influences of long-term ground behavior, such as creep and consolidation settlement of diluvial cohesive soil on the piles, cannot be evaluated.

The pushing test method is specified in the **Technical Standards of the Japanese Geotechnical Society (JGS 1811)**. Conventionally, only a multi-cycle staged loading method had been specified in the standards, but a single-cycle staged loading method and a continuous loading method were added to the standards with the revisions in 2007. It is thought that the staged loading method is suitable for reproducing long-term actions, such as the loading conditions, and the continuous loading method is suitable for reproducing short-term actions, such as earthquakes.[35)] Although there are reports saying that there is no significant difference in the characteristic values between the staged and continuous loading methods,[36)] it is preferable to conduct the pushing test through the multi-cycle staged loading method for consistency with the current design methods, as described above, and the performance records, which are consistent with the test results.

(2) Points of caution when planning loading tests

① Planned maximum loads

In the characteristic investigation test, it is generally necessary to set the planned maximum loads so as to enable the characteristic values of the pile resistance against a pushing force in axial directions to be confirmed. In the design of piles for port facilities, the second limit resistance of the piles (refer to item (3)below) is generally used as the characteristic value of the pile resistance against a pushing force in axial directions. Here, it may be practically difficult to increase the test loads until the second limit resistance can be confirmed in the case of piles supported by stiff layers of soft rock. In this case, it is preferable to examine the possibility of setting the planned maximum loads in a similar way to the confirmation test.

In the confirmation test, the planned maximum loads shall be those that enable the designed values to be confirmed as appropriate even after considering a certain level of safety allowance. However, when conducting the confirmation test with test conditions different from the design conditions (e.g., the existence of soil layers which are planned to be excavated or whose resistance is not considered in the design because the layers have a possibility of liquefaction or consolidation settlement), the planned maximum loads shall be set in consideration of the actions of such layers applied to the test piles in the form of skin friction.

② **Test equipment**

The equipment required in the pushing test includes hydraulic jacks to directly apply loads to the piles, as shown in **Fig. 3.10.8**, loading beams as the reaction force of the loads, and reaction piles to prevent the loading beams from being pulled out. In the case of the onshore pushing test, ground anchors can be used in place of the reaction piles.

The test equipment shall be planned for the safe implementation of the pushing test with the planned maximum loads. That is, the loading beam needs to have sufficient stiffness, and other members shall be planned so as to keep their bending and shear stresses within the elastic ranges. The required number of reaction piles shall be determined by taking into consideration the skin friction so as to prevent the reaction piles from being pulled out when subjected to the planned maximum loads. The reaction piles shall be kept at distances at least three times the diameters of the test piles away from the test piles (1.5 m at minimum), as shown in **Fig. 3.10.9**. Thus, it is necessary to pay attention to the pile intervals when using the actual piles as reaction piles in the case of testing piles for bridge foundations.

③ **Loading and measuring methods**

The loading method shall preferably be the multi-cycle staged loading method.

The basic measuring items include pile head loads, pile head displacement and pile tip displacement. When measuring base resistance R_p separately from skin friction R_f, strain indicators shall be installed on the test pile bodies (at around their pile tips).In addition, installing strain indicators at different depths of the pile bodies corresponding to the boundaries of the soil layers enables the distribution of the axial force in the depth direction and the skin friction in the soil layers with the strain indicators installed at their upper and lower boundaries ($R_{f1\text{-}2}$ in the figure) to be measured, as shown in **Fig. 3.10.10**.

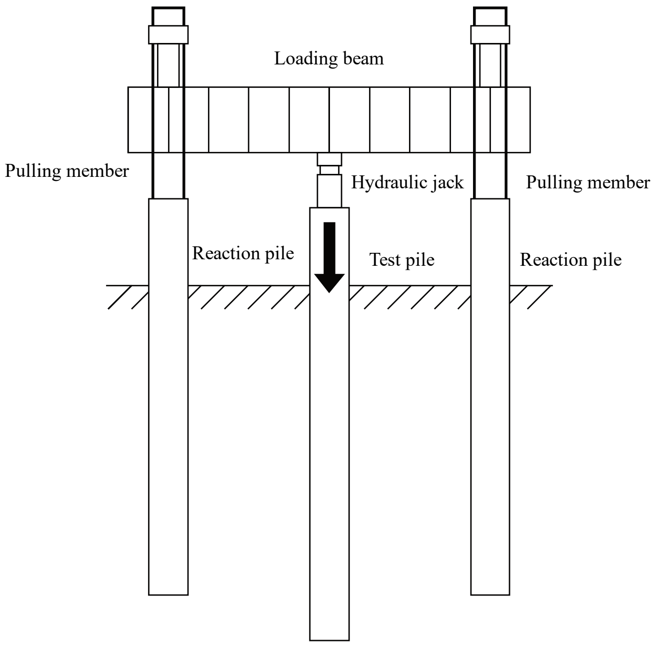


Fig. 3.10.8 Pile Pushing Test Equipment[37)]

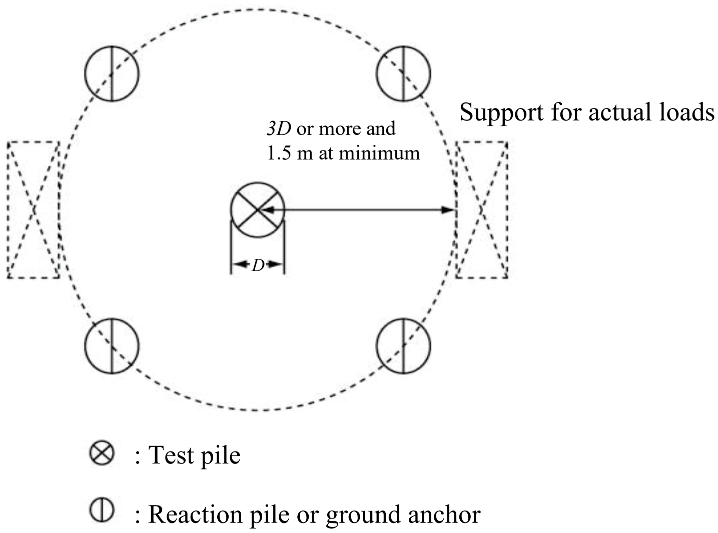


Fig. 3.10.9 Required Interval between the Test and Reaction Piles[38)]

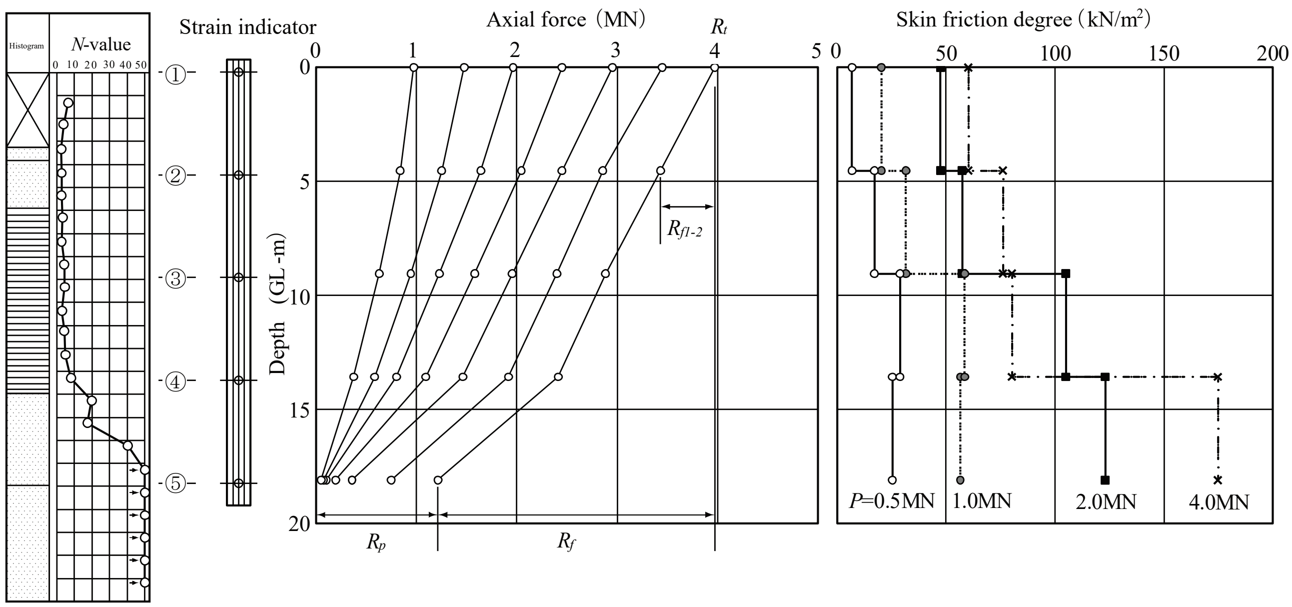


Fig. 3.10.10 Distribution of the Axial Force and Skin Friction Degrees
(Skin Friction per Unit Contact Area) (Revision of reference 39)

(3) Organization of test results

① First limit resistance

The first limit resistance is defined as a load corresponding to a clear break on a log$P$-log$S$ curve representing the relationship between pile head load P and pile head displacement S on double logarithm coordinates, as shown in **Fig. 3.10.11**. The first limit resistance almost corresponds to the values which used to be called yield loads and is considered to represent the upper limits of the elastic ranges where the piles can behave elastically and the ultimate state of skin friction.

② Second limit resistance

The second limit resistance is the maximum pile head load obtained through the loading tests. The second limit resistance almost corresponds to the values which used to be called ultimate loads. It shall be noted that there is a limitation to the second limit resistance in terms of the pile displacement. As shown in **Fig. 3.10.12**, the maximum pile head loads when the pile tip displacement is within a range of 10% or less of the pile tip diameters shall be used to determine the second limit resistance. Although the load-displacement curves of some friction piles in soft ground have clear peaks, those of bearing piles do not in many cases. In such cases, the second limit resistance is determined by the pile tip displacement. There may be cases of using the pile head displacement to determine the second limit resistance when measurements of the pile tip displacement are not available. In these cases, because the pile head displacement is larger than the pile tip displacement by the amounts corresponding to the compression deformation of the pile bodies, the second limit resistance based on the pile head displacement is likely to become smaller than that based on the pile tip displacement.

③ **Base resistance and skin friction**

The measurement results of the strain indicators installed on the pile bodies and the settlement gauges at the pile tips can be used to obtain the distribution of the axial force, as shown in **Fig. 3.10.10**, and the relationships between the base resistance and pile tip displacement, as shown in **Fig. 3.10.12**. The distribution of the axial force can be used to organize the relationships between the skin friction at respective intervals of the strain indicators and the average displacement at the intervals (refer to **Fig. 3.10.13**). Skin friction is often processed into skin friction per unit contact area between the piles and ground.

④ **Determination of characteristic values**

In the case of the characteristic investigation test, the characteristic values for use in the design are determined based on the measurements obtained by organizing the test results.

In the design of piles for port facilities, the second limit resistance is generally used as the characteristic value of the pile resistance against a pushing force in axial directions. When the pile tip displacement does not reach 10% of the pile tip diameters even with the maximum loads in the loading tests, the value 1.2 times the first limit resistance can be used as the characteristic value of the pile resistance against a pushing force in axial directions.[36),40)]Furthermore, when the first limit resistance cannot be obtained from the measurement results of the loading tests, the value 1.2 times the maximum load applied to the piles can be used as the characteristic value of the pile resistance against a pushing force in axial directions.

The base resistance when the second limit resistance is exerted is generally used as the characteristic value of the base resistance. In addition, the maximum skin friction or skin friction per unit contact area obtained through the loading tests are often used as the respective characteristic values. However, depending on the soil layers, there may be cases where the skin friction shows a significant reduction after peaking, along with an increase in displacement, and reaches a residual state with small skin friction when the second limit resistance is exerted. In such cases, the skin friction when the second limit resistance is exerted can be used as the characteristic value of the skin friction.

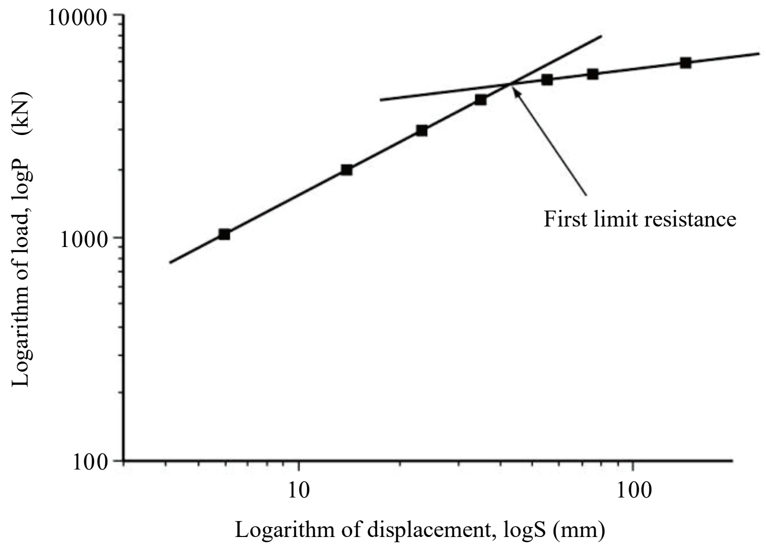


Fig. 3.10.11 Method for Obtaining First Limit Resistance[41)]

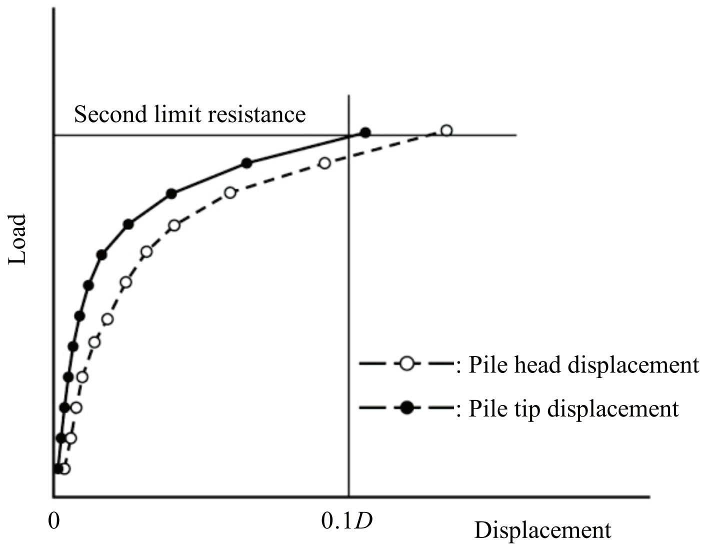


Fig. 3.10.12 Method for Obtaining Second Limit Resistance[42)]

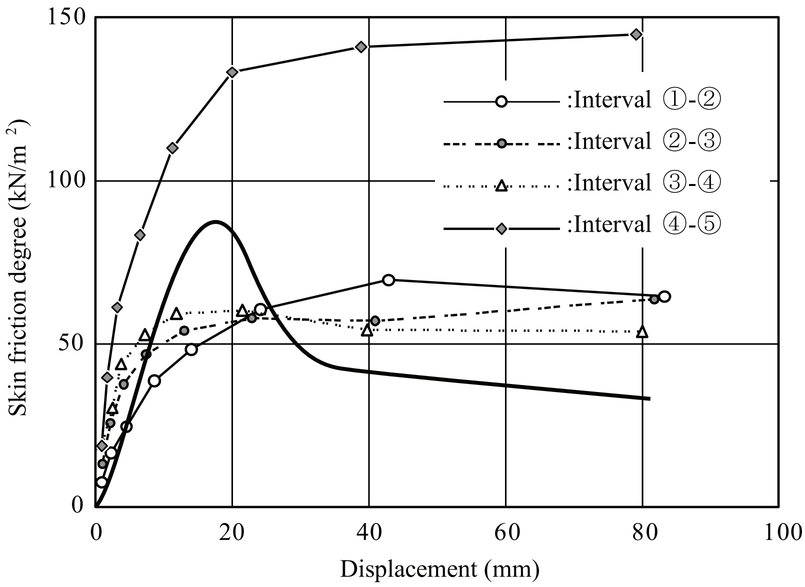


Fig. 3.10.13 Relationship between Skin Friction Degrees at Intervals of Strain Indicators (refer to **Fig. 3.10.10**) and Displacement[43)]

3.10.6 Rapid Pile Loading Test

(1) Test outline

The rapid loading test can be easily applied to offshore loading tests and narrow onshore loading tests because it applies dynamic loads to the pile heads without requiring reaction equipment such as reaction piles. The test method for the rapid loading test is specified in the **Technical Standards of the Japanese Geotechnical Society (JGS 1815)**.

The major difference between the rapid loading test and the impact loading test, which also applies dynamic loads to the pile heads, is the lengths of the loading times. When comparing the relative loading time T_r between the two tests, that of the rapid loading test of $5 \leq T_r < 500$ is longer than that of the impact loading test of $T_r < 5$.The relative loading time is a ratio of the loading time of a load to the time required for waves (divergent waves) to propagate back and forth through a pile body and is expressed by the **equation (3.10.1)**.

$$T_r = \frac{t_L}{2L/C} \tag{3.10.1}$$

where

t_L : the loading time (s);

L : the pile length (m); and

C : the propagation velocity of the divergent wave.

In the rapid loading test, the pile bodies are pressed into the ground in an almost completely compressed state in axial directions. Thus, the wave phenomena generated in the pile bodies due to the dynamic loads can be ignored in a practical sense. However, the influences of the inertia force of the pile bodies cannot be ignored, and the resistance of the ground measured through the loading tests includes dynamic components. Therefore, in order to obtain the resistance against a pushing force in an axial direction equivalent to that of the pushing test, it is necessary to eliminate the influences of the inertia force of the pile bodies and the dynamic components of the ground when organizing the test results.

(2) Points of caution when planning loading tests

① Planned maximum loads and planned loading times

The basic concept of the planned maximum loads for the rapid loading test is the same as that for the pushing test (refer to **Reference [Part II], Chapter 1, 3.10.5 Pile Pushing Test**). In addition to the basic concept, the planned maximum loads for the rapid pile loading test shall be determined in consideration of the inertia force of the pile bodies due to rapid loading. Furthermore, in the case of cohesive ground, the planned maximum loads need to be augmented in consideration of the increase in ground resistance due to the strain rate dependency. Depending on the test conditions, but loads that are about 20 to 30% larger than the target static resistance against a pushing force in axial directions (corresponding to the resistance obtained through the pushing test) are often adopted as the maximum loads in the rapid loading test.

Furthermore, because the loading time needs to be sufficiently long to be able to ignore the wave phenomena generated in the pile bodies, the rapid loading test shall be planned to achieve a relative loading time T_r of 5 or more.

② Test equipment

Initially when the rapid loading test was first introduced in Japan, the loading test using test equipment with a reaction body inertial method was the most commonly used test. In the loading test using the reaction body inertial method, the test piles are subjected to loads with reaction bodies (weights) placed on the pile heads, which are set off with the pressure generated through the combustion of high-pressure gas or chemical reactions (refer to **Fig. 3.10.14**). However, because it is difficult to conduct multi-cycle loading with the reaction body inertial method, the rapid loading test using this method has rarely been used in recent years for the piles of port facilities.

Recently, the mainstream rapid loading test equipment uses test equipment with a soft cushion weight fall method, which applies a striking force of falling weights to the pile heads using cushions (refer to **Fig. 3.10.15**). The method enables the striking force to be converted to the appropriate loads with the loading times required for the rapid loading test by adjusting the characteristics of the cushions. The soft cushion weight fall method enables the loads to be adjusted by changing the weight falling heights and is easily applicable to the multi-cycle loading test.

③ Loading and measuring methods

In order to be consistent with the pushing test, it is preferable to conduct multi-cycle loading. In this case, considering the mechanism of the test equipment, the loading method shall be limited to the continuous loading method.

The basic measuring items include pile head loads, pile head displacement and pile head acceleration. When it is necessary to measure the base resistance separately from the skin friction or confirm the distribution of skin friction in the depth direction, acceleration meters and strain indicators shall be installed at different depths of the test pile bodies.

Because the loading time of the rapid loading testis extremely short, it is necessary to set the sampling intervals (the time intervals to acquire data) at 1 ms or less (at a sampling frequency of 1.0 kHz or more). To that end, dynamic strain meters and laser displacement gauges shall be used for measuring the strain and displacement, respectively.

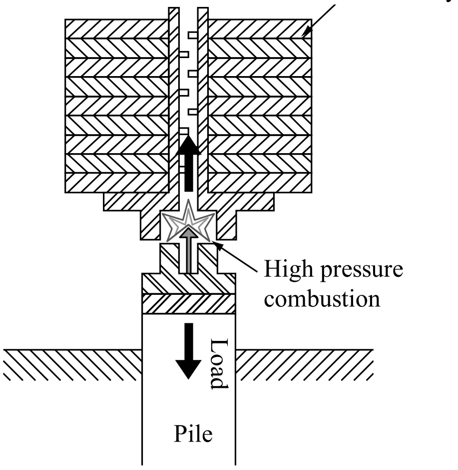


Fig. 3.10.14 Example of the Rapid Loading Test with the Reaction Body Inertial Method[44)]

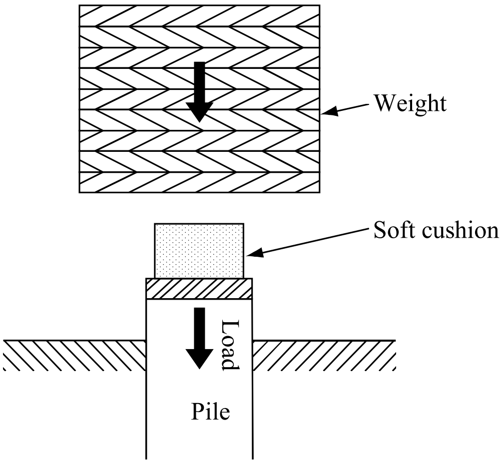


Fig. 3.10.15 Example of the Rapid Loading Test with the Weight Fall Method[30)]

(3) Organization of test results

In the rapid loading test, the influences of the inertia force of the pile bodies and the dynamic resistance of the ground need to be eliminated from the load-displacement curves obtained through the test. As shown in **Fig. 3.10.16**, in the pushing test, the pile head load (static load F_{static}) is balanced with the static load of ground R_w. In contrast, in the rapid loading test, the pile head load (rapid load F_{rapid}) is balanced with the sum of the static load of ground R_w, the inertia resistance of pile R_a and the dynamic resistance component of ground R_v.

Then, when organizing the rapid loading test results, first, ground resistance $R_{soil} = R_w + R_v$ shall be calculated by subtracting the inertia resistance of pile R_a from pile head load F_{rapid}. Next, the relationship between the ground resistance and pile head displacement can be obtained as shown in the graph on the left side of **Fig. 3.10.17**. Here, focusing on the point with the maximum head pile displacement (called an unloading point) in the graph, because the velocity of the pile body at the unloading point is 0, the dynamic resistance component of ground R_v is also estimated to be 0 at the unloading point. Thus, the unloading point represents the relationship between the static resistance component of ground R_w and the pile head displacement.

Accordingly, a static load-displacement curve can be drawn by conducting the multi-cycle rapid loading test and connecting the unloading points measured in the respective cycles (refer to the graph on the right side of **Fig. 3.10.17**). With the static load-displacement curve, the first limit resistance and the second limit resistance can be obtained as with the pile pushing test. Alternatively, there are cases of obtaining the static load-displacement curves by conducting numerical analyses based on the rapid loading test results without using the unloading point method.

In addition, as with the pushing test, the rapid loading test with strain indicators and acceleration meters installed on the pile bodies makes it possible to obtain the distribution of the axial force, measure the base resistance separately from the skin friction and calculate the distribution of the skin friction in the depth direction.

It shall be noted that the static load-displacement curves obtained through the rapid loading test do not completely agree with the load-displacement curves obtained through the pushing test, because the behavior of the piles and ground is simplified to some extent by modeling the piles and ground when organizing the test results through the unloading point method or numerical analyses. Furthermore, it is particularly difficult to accurately estimate the strain rate dependency of the ground resistance. However, as long as the loading tests are properly conducted and the test results are properly organized, the results of the rapid loading test are generally considered to be consistent with those of the pushing test.

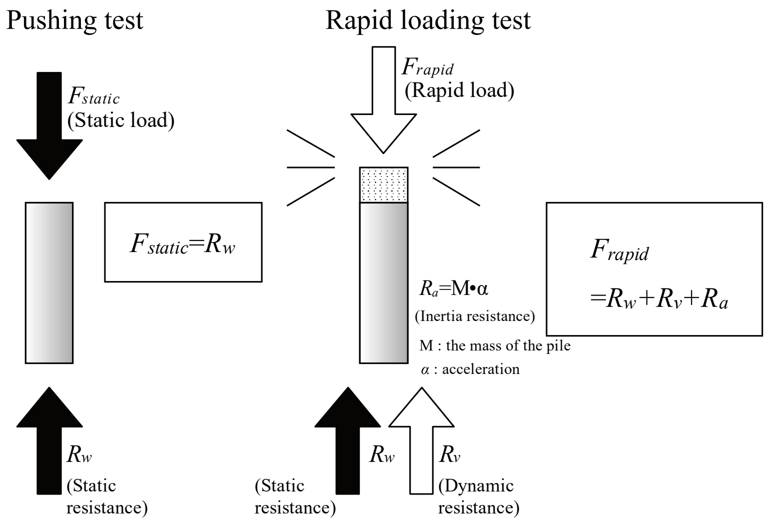


Fig. 3.10.16 Balance of Force in the Pushing and Rapid Loading Tests[45)]

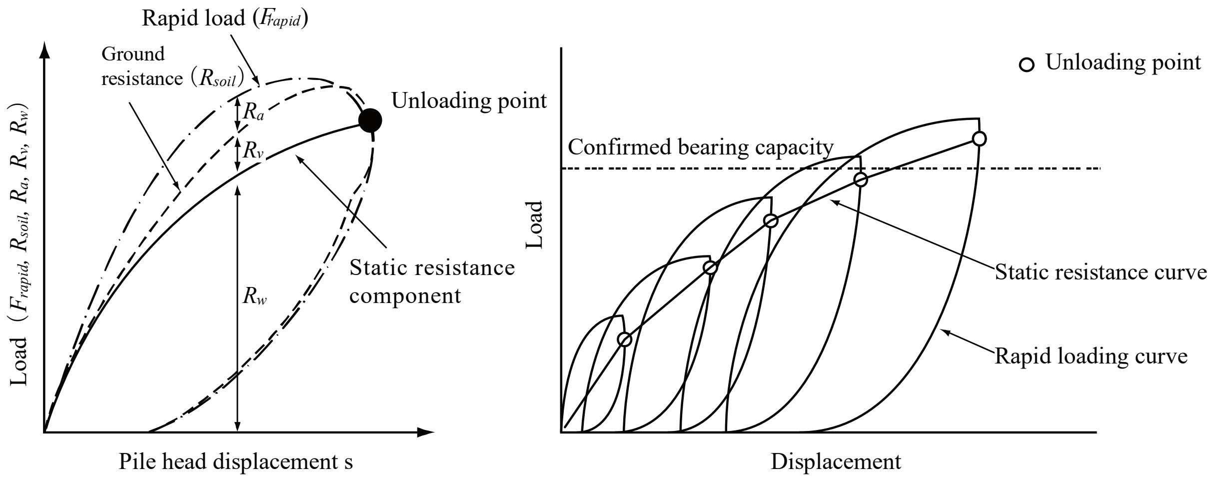


Fig. 3.10.17 Analyses based on the Unloading Point Method[46)]

3.10.7 Impact Pile Loading Test

(1) Test outline

The impact loading test was originally developed as a method for supervising pile driving work and is a method that applies dynamic loads, such as the striking force of hammers, to pile heads in a short amount of time (the relative loading time of T_r<5). Because of the short loading time, the impact loads generate a force called wave phenomena inside the pile bodies and cause them to have different stress states during the impact loading test compared to those during the pushing and rapid loading tests. Furthermore, because the penetration rates of the piles in the impact loading test are larger than those in the rapid loading test, the ratios of the dynamic resistance components to the total ground resistance are larger in the impact loading test. Generally, in the impact loading test, the pile resistance corresponding to the results of the pushing test is obtained in a manner that installs strain indicators and acceleration gauges on the pile heads, measures the strain waveforms and acceleration waveforms generated in the pile bodies when hit by hammers, and analyzes the test results based on the kinematic wave theory. The analysis method generally used in the impact loading test is a simulation analysis method called the waveform matching analysis, which isbased on the one-dimensional kinetic wave theory. The impact loading test method is specified in the **Technical Standards of the Japanese Geotechnical Society (JGS 1816)**.

(2) Points of caution when planning loading tests

① **Test time**

When conducting the impact loading test for driven piles, because of the availability of the impact loading test when the piles are being constructed, unlike other types of loading tests, the test can be conducted in two different ways: a post-construction impact loading test conducted after the piles are built and a post-curing impact loading test conducted once the predetermined curing periods have lapsed after the construction of the piles.

The post-construction impact loading test is often conducted upon completion of the pile driving work, and the post-curing impact loading test is conducted after waiting for the restoration of the strength of the ground disturbed by pile driving. Pile driving generates excess pore water pressure and loosens the ground, but curing driven piles enables the effective stress of the ground to be increased when the excess pore water pressure is dissipated, thereby allowing the ground to recover from being disturbed. The phenomenon causing the increase in static resistance is called set-up, and the ratio of the static resistance after the completion of pile driving to that after curing is called a set-up ratio (refer to **Fig. 3.10.18**). Thus, once the set-up ratios are known, conducting only the post-construction impact loading test may allow the static resistance after curing to be estimated for the piles around those which have been tested.

In the **Methods for Vertical Pile Loading Tests and Commentaries**,[34)] the target curing periods are recommended to be five days or more and 14 days or more after the construction of piles in the case of sandy ground and cohesive ground, respectively. However, curing periods that take too much time may make the appropriate loading tests difficult because of an insufficient hammer capacity with respect to the restored ground resistance. To that end, there are many cases of limiting the curing periods to a few days to one week, as well as cases of conducting the post-curing impact loading test multiple times by changing the curing periods.

When conducting the post-curing impact loading test using actual piles, it is preferable to preliminarily examine the allowances in the overall pile lengths and the lengths of heavy-duty corrosion protection areas in cases where the elevations of the pile heads after the pile driving work may be lower than the designed elevations.

② **Planned maximum impact energy**

In the impact loading test, first, the planned maximum loads (the static resistance of ground) are set, as is the case in the pushing test and rapid loading test, then the planned maximum impact energy is planned in accordance with the planned maximum loads. Because the resistance of the ground cannot be appropriately evaluated if the amounts of displacement of the piles when they are hit are too small, it is necessary to allow the piles to be sufficiently displaced with the appropriate impact energy. Thus, the planned maximum impact energy needs to be set to ensure a sufficient displacement of the pile bodies. **Fig. 3.10.19** shows a procedure from the setting of the static resistance of the ground to the selection of the hammers. The hammers shall be selected with due consideration to the transmittable energy, transmission efficiency and characteristics of the construction machines (refer to **Figs. 3.10.20** and **3.10.21**).

There have been many cases where the maximum impact energy required for the loading tests of large diameter steel pipe piles used for port facilities becomes significantly large, causing difficulties in procuring adequate hammers or problems with possible damage to the pile bodies due to excessive stresses in the pile bodies being subjected to significantly large impact energy. In such cases, it is necessary to review the set values of the planned maximum loads or the items to be investigated in the loading tests. For example, there are cases of keeping the impact energy at manageable levels by limiting the items to be investigated in the loading tests to the base resistance and constructing test piles in a manner that prevents them from being subjected to a large skin friction force. In other cases, the impact energy can be kept at manageable levels by applying the interpretation of the test results, focusing on the fact that the skin friction is more affected by the set-up than the base resistance, and that the measurements of the base resistance through the post-construction impact loading test and those of skin friction through the post-curing impact loading test can be combined for the evaluation of the pile resistance against a pushing force in axial directions.

③ **Test equipment**

Fig. 3.10.22 shows an example of the equipment used for the impact loading test. The impact loading test can be conducted at lower costs by substituting the weight fall equipment with hammers used for constructing the actual piles. However, it is necessary to be attentive to possibly insufficient hammer energy when using hammers for the post-curing impact loading test. There may be cases where hammers that are efficient for constructing the actual piles become insufficient for the post-curing impact loading test because of a possible increase in the resistance of the ground due to the set-up. Thus, there are cases of preparing high-standard hammers just for the post-curing impact loading test, in addition to the hammers used for the actual construction. When using hydraulic hammers with a ram weight of 10 tons, which are widely used in port construction, the measurable static resistance in the impact loading test is around 6,000 to 7,000 kN. Thus, when the resistance of the ground is expected to be larger than the capacity of the hammers used for construction, weight fall equipment with sufficient impact energy shall be mobilized to the sites.

In addition, in the post-curing impact loading test, it is necessary to ensure the bearing capacity of the pile materials because the piles are subjected to a larger impact force in the test than in the actual construction. When the bearing capacity of the pile materials is not sufficient, measures shall be taken to change the materials or increase the wall thickness.

④ **Measuring method**

The pile head loads and pile head acceleration shall be measured with two sets of a strain indicator and an acceleration meter installed at axially symmetric positions at least 1.5 times the length of the pile diameters away from the pile heads. In the impact loading test, it is necessary to use measuring equipment (with sampling intervals of 0.1 ms or less) capable of accurately measuring input signals of up to at least 10 kHz in order to obtain accurate data on the stress waves propagating through the pile bodies at high speeds.

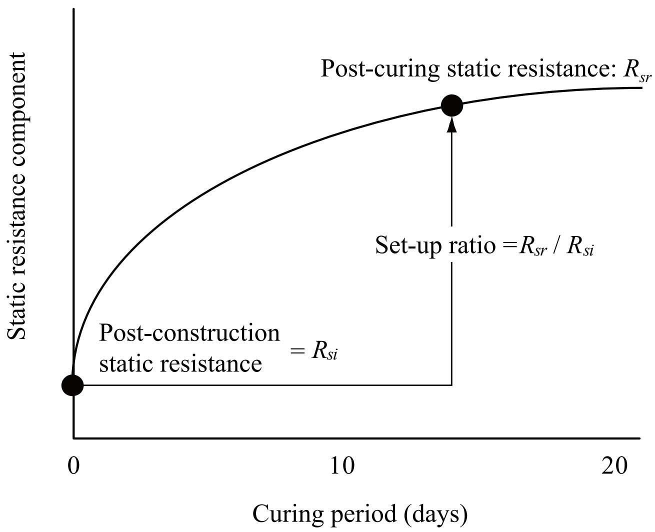


Fig. 3.10.18 Concept of a Set-up Ratio[47)]

Setting of static resistance components that satisfy the test purposes

↓

Estimation of the transmission energy corresponding to the set static resistance components

↓

Calculation of nominal energy taking into consideration energy transmission efficiency

↓

Selection of a hammer based on the nominal energy

Fig. 3.10.19 Hammer Selection Procedure[48)]

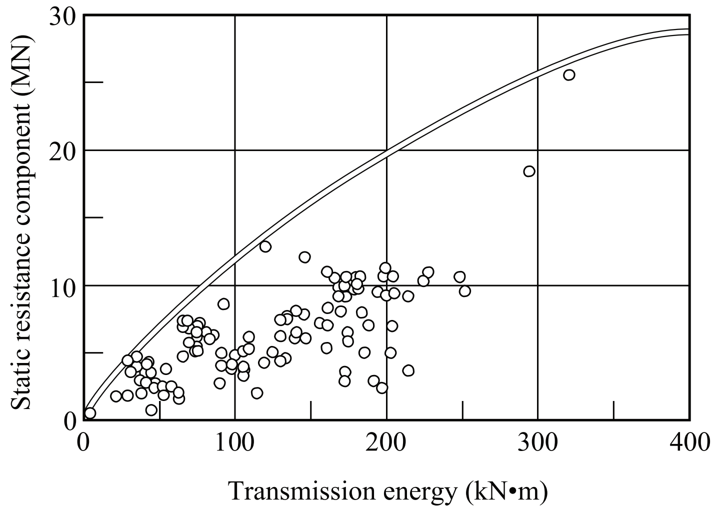


Fig. 3.10.20 Relationship between Transmission Energy and Static Resistance Components[48)]

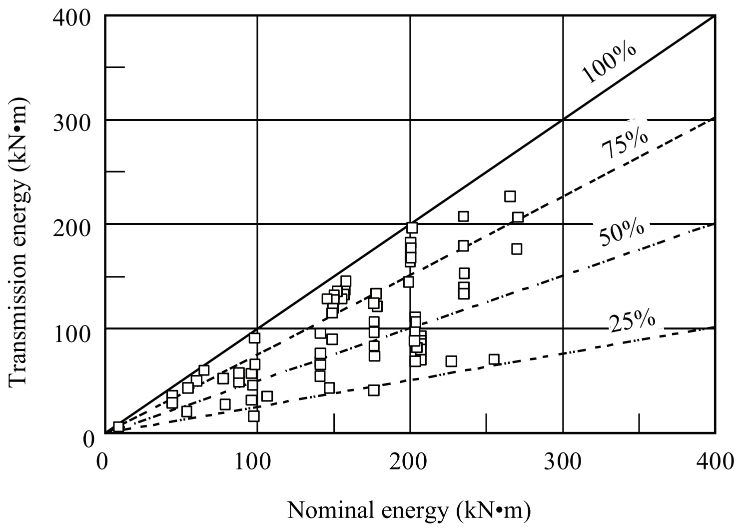


Fig. 3.10.21 Relationship between Nominal Energy and Transmission Energy[48)]

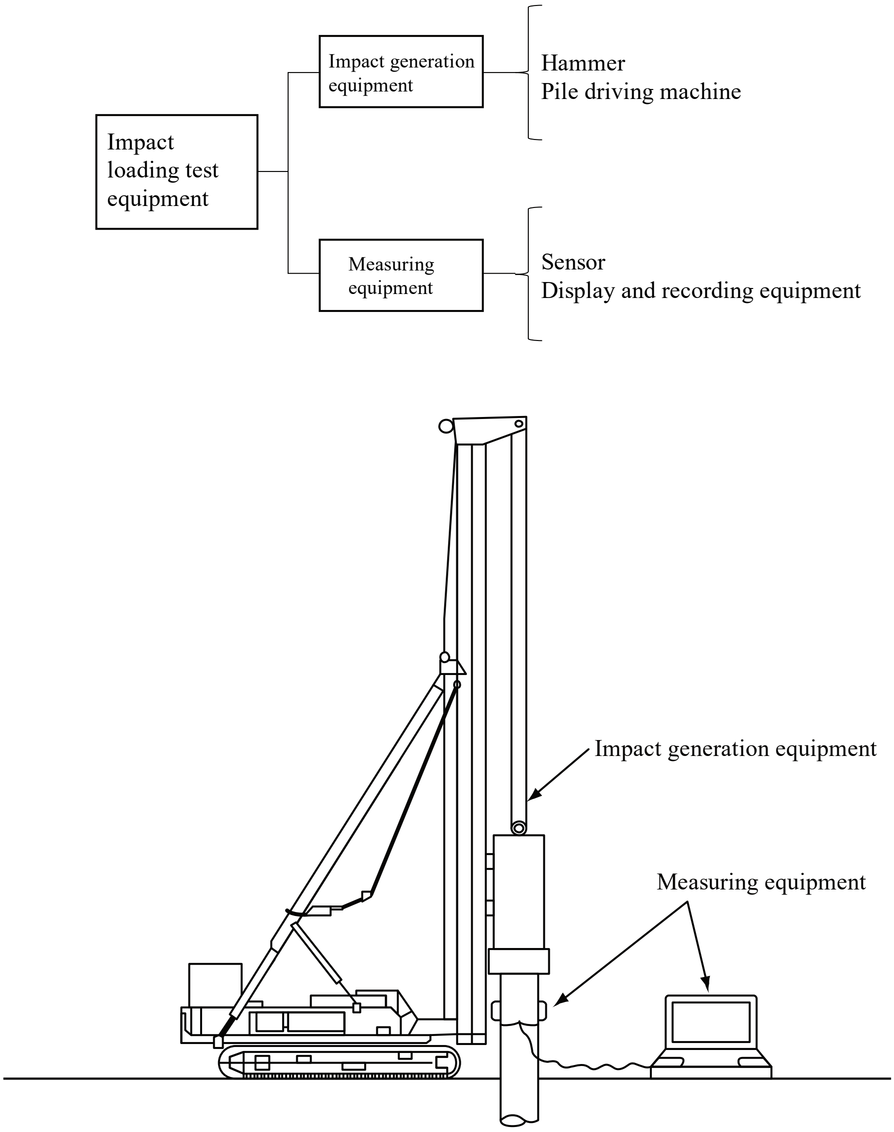


Fig. 3.10.22 Example of the Configuration of Impact Loading Test Equipment[49)]

(3) Organization of test results

In the impact loading test, the pile resistance is analytically estimated from the test results. First, the strain measured at the pile heads is used for obtaining the axial force, and the acceleration measured at the pile heads is used for obtaining the velocities and displacement of the pile bodies. Next, the input waves and reflected waves in axial directions at the pile heads are obtained from the axial force and velocities based on the one-dimensional kinetic wave theory (refer to **Fig. 3.10.23**). The input waves and reflected waves in axial directions represent the hammer impact loads and the resistance states of the ground, respectively. The static resistance characteristics of the piles can be obtained through several analyses of these values.

The analysis methods include the CASE method and the waveform matching analysis method. The CASE method enables the resistance of the ground to be measured during the impact loading test in real time, but the accuracy of the measurement results is low. The waveform matching analysis method is used to simulate the loading test by modeling the piles and surrounding ground and identify the static resistance components of the ground through inverse analyses, enabling the friction force of each soil layer to be accurately obtained. For the details of these analysis methods, refer to the **Methods for Vertical Pile Loading Tests and Commentaries**.[50)]

In the impact loading test, the individual values of pile resistance against a pushing force in axial directions, base resistance and skin friction are generally reported without showing the relationships between the resistance and displacement. Additional numerical analyses enable curves representing the relationship between the static loads and displacement at the pile heads to be estimated from the waveform matching analysis results, but such numerical analyses have been rarely practiced in the pile loading tests for port facilities. Thus, in many cases, the first and second limit resistance are not available in the impact loading test. Furthermore, because of problems with a possible shortage of impact energy, as described in item (2) above, it is generally difficult to conduct the impact loading test as the characteristic investigation test. However, considering that the impact loading test results (even though they are not the second limit resistance) can be one of the indexes of the pile resistance against a pushing force in axial directions, and the relatively simple test method enables the test to be conducted multiple times, there have been trials to estimate the pile resistance against a pushing force in axial directions through statistical processing of a large number of impact loading test results.[51)] In addition, there have been many cases of conducting the impact loading tests as confirmation tests during construction, and there have been proposals of methods for improving the accuracy of construction work supervision using the impact loading test results.[52)]

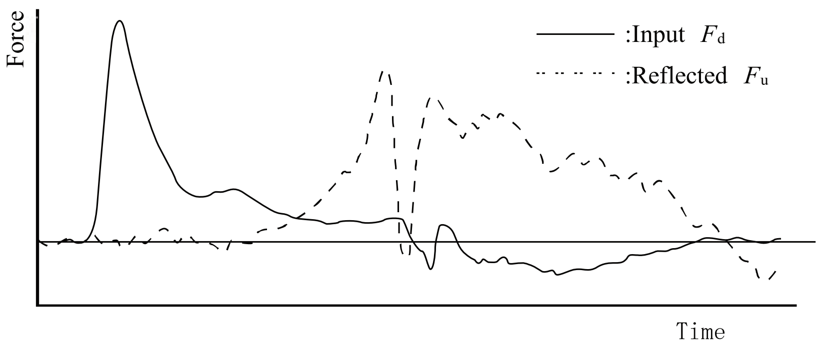


Fig. 3.10.23 Example of Curves Representing the Relationships between Input Waves and Time, and between Reflected Waves and Time[53)]

3.10.8 Pile Pulling Test

(1) Test outline

The pile pulling test applies static pulling loads to the pile heads. The equipment used in the pulling test includes a force application device, a reaction device and a measuring device. Reaction piles are generally used as the reaction bodies in the test.

The pulling test method is specified in the **Technical Standards of the Japanese Geotechnical Society (JGS 1813)**. The multi-cycle staged loading method used to be the only method specified in the standards, but the one-cycle staged loading and the continuous loading methods have been additionally specified in the standards since the revision in 2007.The pulling force applied to the piles is generally caused by earthquakes, winds and wave loads, which act on the piles in relatively short periods. In the past, the test force duration was 15 minutes, but since the revision, it has been changed to 30 minutes, as is the case in the pushing test, while taking into consideration the availability of the continuous loading method. The staged loading method and continuous loading method are considered to be suitable for reproducing the loading conditions of long-term and short-term actions (such as earthquakes), respectively.

(2) Points of caution when planning loading tests

① Planned maximum loads

The same basic concept of the planned maximum loads as the pushing test can be applied to the pulling test (refer to **Reference [Part II], Chapter 1, 3.10.5 Pile Pushing Test**); provided, however, that the planned maximum loads shall be set while taking into consideration the self-weight of the piles and the resistance against the pulling force at the pile tips. Although the effective self-weight with buoyancy subtracted from the total weight is generally used in the pile design, it is recommended to use the total weight in the pulling test. For the resistance against the pulling force at the pile tips, it is preferable to use a suction pressure corresponding to the atmospheric pressure at the pile tips. Furthermore, considering that the loads required for the pulling test are smaller than those of the pushing test, it is preferable to conduct the pulling test until the loads reach the second limit resistance (refer to item (3) below).

② **Test equipment**

Fig. 3.10.24 shows an example of the equipment used in the pulling test. As shown in the figure, a test pile is subjected to a pulling force that is applied through a reaction beam pushed up by a jack. Reaction piles are generally used to support the reaction of the pulling force (reaction pile method), and there are cases of using reaction plates, as shown in **Fig. 3.10.25** (reaction plate method). In the pulling test, the reaction force acts in the downward direction, which is the opposite direction as in the pushing test. Thus, it is necessary to fully examine any problems with the bearing capacity or deformations in the ground.

The equipment for the pulling test shall be planned so as to enable the planned maximum loads to be safely applied to the piles. It is also necessary to prepare loading beams with sufficient stiffness and to plan other members so that their bending and shear stresses are kept within elastic ranges. Furthermore, the reaction piles and plates shall be kept at a distance at least three times the diameters of the test piles away from the test piles (1.5 m at minimum), as shown in **Fig. 3.10.26**. Thus, it is necessary to pay attention to the pile intervals when using actual piles as reaction piles in the case of testing piles for bridge foundations.

In the pulling test, it is very difficult to apply pulling loads to the centers of the test piles because of many factors, including the inclinations of the test piles, the gradients of the pile head faces and the positions of the force application jacks. Thus, there are cases where the pile bodies are subjected to a bending stress in addition to a tensile stress at the pile heads, with tensile test loads applied to the pile heads as eccentric loads. Thus, it is necessary to reinforce the pile head sections so as to prevent the pile bodies from yielding or being damaged when subjected to the actions of the planned maximum loads. The methods for reinforcing the steel pipe piles include increasing the wall thicknesses and providing reinforcing ribs inside the piles. Examples of the methods for reinforcing steel pipe piles are shown in **Fig. 3.10.27**.

③ **Loading and measuring methods**

The pulling force acting on the piles is generated either by loads acting on the piles for relatively short periods, such as earthquakes, winds and waves, or horizontal loads acting constantly on the piles, as is the case with sheet pile walls with pile anchorages. Thus, the loading methods shall be determined in accordance with the design conditions of the piles.

The basic items to be measured are the pile head loads and pile head displacement. When measuring the skin friction of each soil layer, strain indicators shall be installed on the pile bodies. Concrete piles such as cast-in-place concrete piles often have cracks when subjected to an axial tensile force at areas close to where the axial strain indicators are installed. In such cases, it is advisable to increase the number of cross sections to measure the axial strain in areas with possible risks of having cracks so as to ensure the successful measurement of the axial strain with the indicators left undamaged by cracks. It is also preferable to measure displacement at the tips and intermediary sections of the test piles as needed just in case the cracks prevent the measurement of strain from which the amounts of displacement are estimated.

In addition, in the pulling test, there may be cases where the maximum resistance against a pulling force is exerted when the test piles are still in the early stages of developing displacement, or the test piles are pulled out in extremely short periods. Thus, it is necessary to give due consideration to such incidences when establishing the measurement plans.

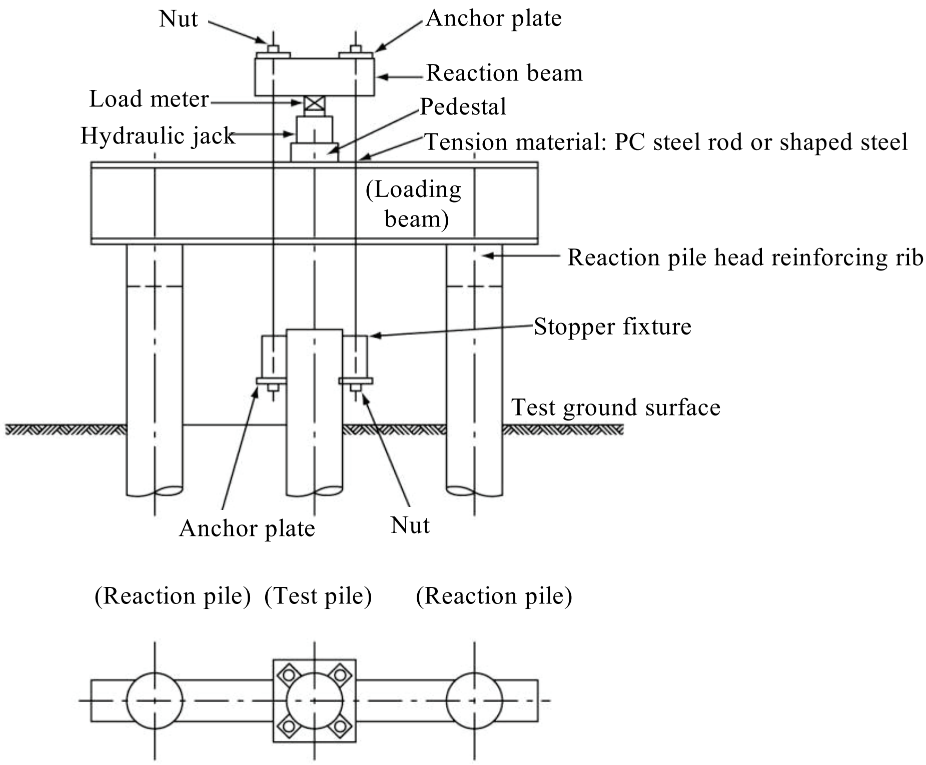


Fig. 3.10.24 Equipment for the Pile Pulling Test[54)]

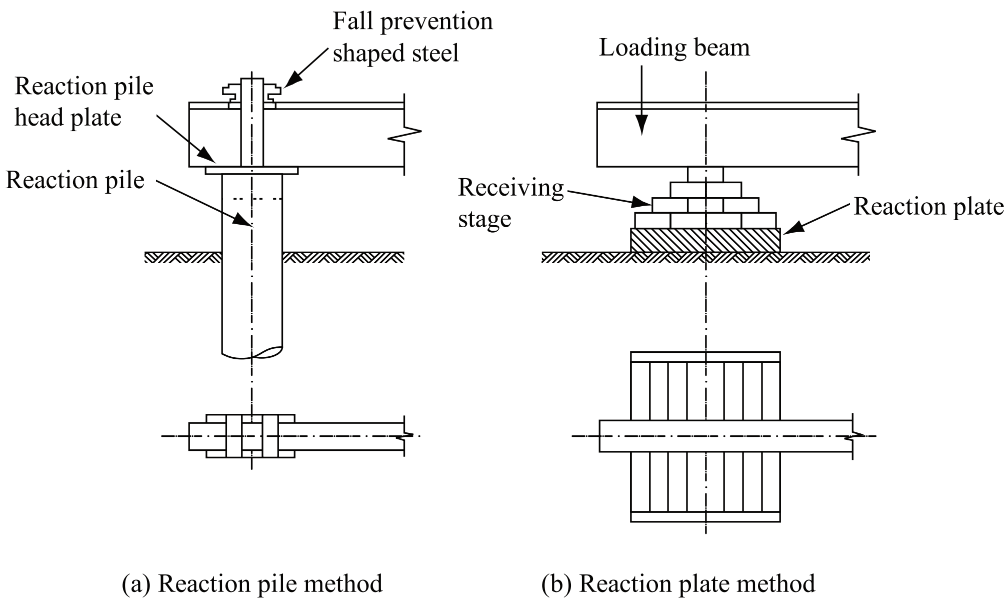
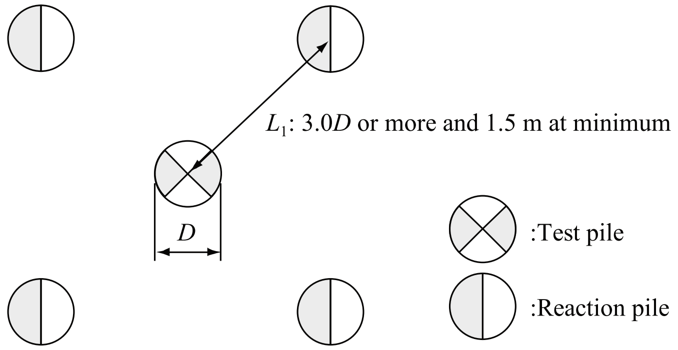
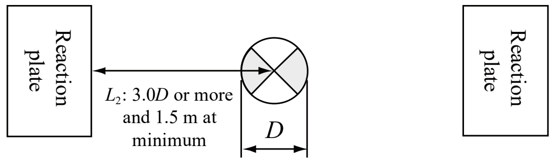


Fig. 3.10.25 Reaction Pile and Reaction Plate Methods[55)]



(a) Reaction pile method



(b) Reaction plate method

Where
L_1: the central clearance between the test pile and reaction pile; and
L_2: the central clearance between the test pile and reaction plate.

Fig. 3.10.26 Required Intervals between the Test Pile and Reaction Piles or Plates[56)]

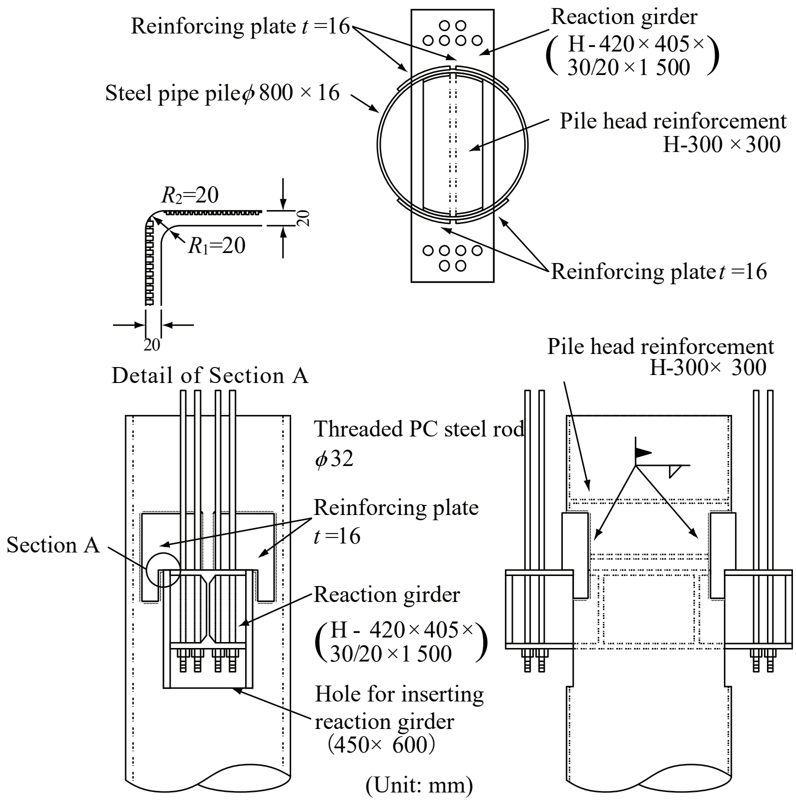


Fig. 3.10.27 Example of Reinforcement of the Pile Head Section of the Test Pile (for Steel Pipe Piles)[57)]

(3) Organization of test results

① **First limit resistance**

The first limit resistance is defined as a load corresponding to a clear break on a log$P$-log$S$ curve representing the relationship between pile head loads P and pile head displacement S on double logarithm coordinates, as shown in **Fig. 3.10.28**. The first limit resistance corresponds to the load when the shear stress, generated on the outer periphery of the pile or in the ground around the pile when the pile is pulled, starts to yield throughout almost the entire length of the pile.

② **Second limit resistance**

The second limit resistance is defined as the maximum pile head load obtained through the pulling test (refer to **Fig. 3.10.29**) and corresponds to the load when the resistance against a pulling force on the outer periphery of the piles is put into an extreme state. When the loading test results do not produce curves with clear peaks, like the one shown in **Fig. 3.10.29**, the loads with a pile tip displacement equivalent to 10% of the pile tip diameters can be set as the second limit resistance.

③ **Skin friction**

The distribution charts of the axial force, as shown in **Fig. 3.10.30**, can be obtained when measuring the axial strain with strain indicators installed on the pile bodies. In the case of general piles, the base resistance can be barely expected, as shown in **Fig. 3.10.30**. However, a resistance against the pulling force at the pile tips can be substantially increased in cases of piles with enlarged bottoms and piles with blades. The skin friction per unit contact area between the piles and ground can be calculated in a manner that divides the skin friction obtained from the distribution charts of the axial force by the outer peripheral areas of the piles.

When obtaining the distribution charts of the axial force for concrete piles, it is necessary to give due consideration to the nonlinearity of Yang's modulus of pile materials. Particular attention is required when the strain amounts are abruptly increased in certain loading stages because of the possibility that the concrete piles might be affected by cracks. Although this is not the case for steel piles, for steel piles with changing cross-sectional areas in the depth direction, or steel piles with strain indicator protection members, the distribution charts of the axial force shall be established with due consideration to these influences for the changing cross-sectional areas or protection members.

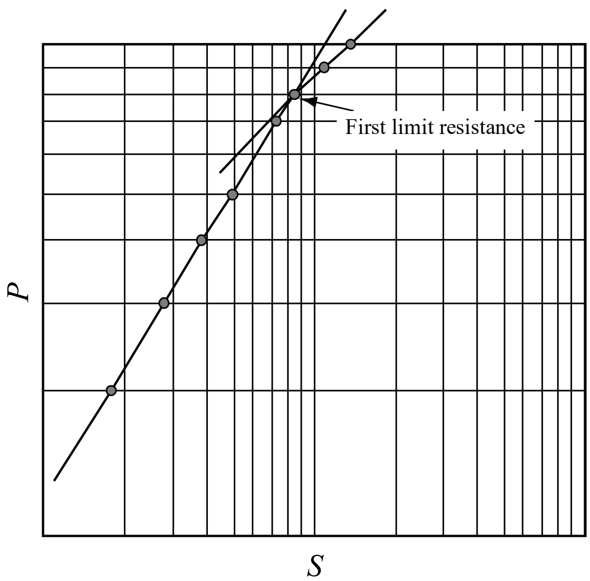


Fig. 3.10.28 Method for Obtaining the First Limit Resistance[58)]

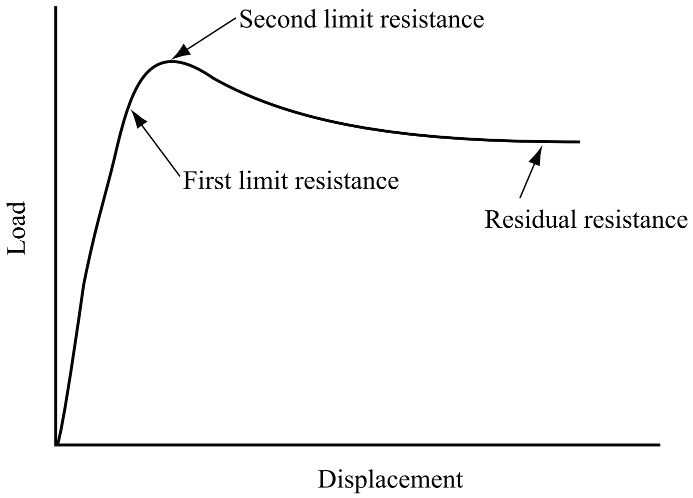


Fig. 3.10.29 Schematic Diagram of the Pulling Test Results[58)]

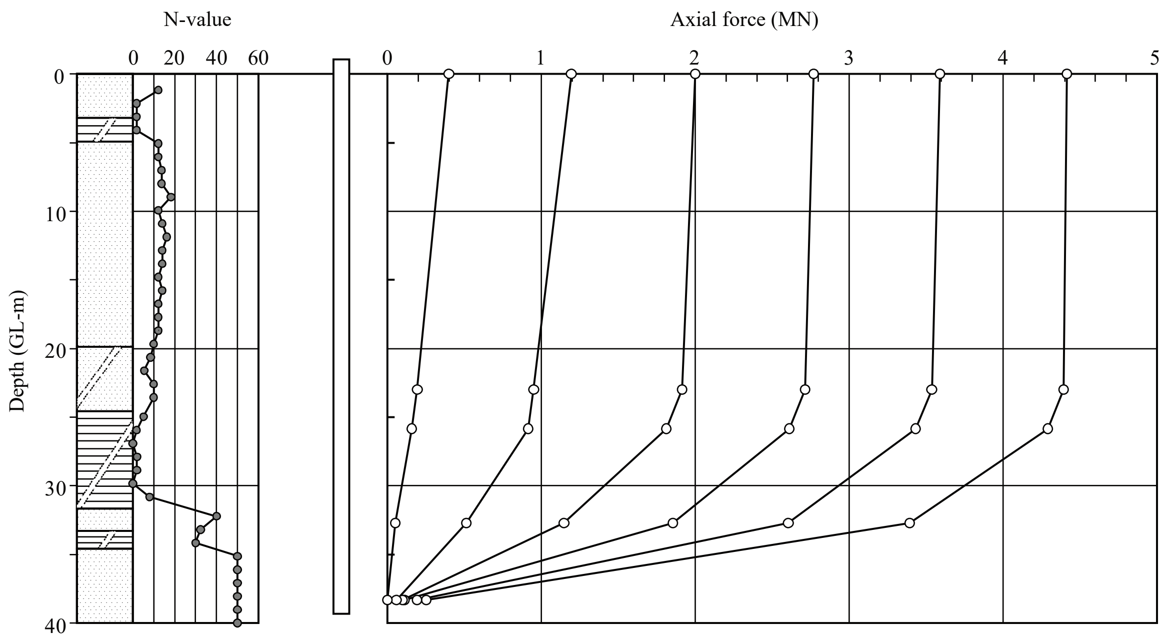


Fig. 3.10.30 Distribution of Axial Force through the Pulling Test[58)]

3.10.9 Horizontal Pile Loading Test

(1) Test outline

The horizontal pile loading test is a test method that statically applies horizontal loads to the vicinity of the pile heads using hydraulic jacks. The purpose of the horizontal loading test is to evaluate the pile resistance against a force perpendicular to axial directions. The behavior of piles subjected to a force perpendicular to axial directions is largely affected by the stiffness and widths of the piles, loading heights, pile head fixing conditions, ground conditions, axial loads and the characteristics of the loads perpendicular to axial directions. It is very difficult to conduct the horizontal loading test under conditions corresponding to the diversified conditions that are expected for the actual piles. Furthermore, it is practically impossible to directly examine the pile resistance against a force perpendicular to axial directions from the horizontal loading test results. Thus, in many cases, the horizontal loading test is conducted while focusing on obtaining the information about the ground resistance (mainly the moduli of subgrade reaction) while estimating the behavior of the actual piles through analytical methods (refer to **Part III, Chapter 2, 3.4.8 Calculation of the deflection of piles though the PHRI method**).

The horizontal loading test method is specified in the **Technical Standards of the Japanese Geotechnical Society (JGS 1831)**. Conventionally, only a multi-cycle staged loading method had been specified in the standards, but a single-cycle staged loading method and a continuous loading method were added to the standards with the revisions in 2010. There is a report showing no significant difference in the coefficients of the lateral subgrade reaction between the continuous horizontal loading method and the staged horizontal loading method under the conditions that the piles are installed in identical ground and subjected to loads at identical loading rates.[59)]

The continuous horizontal loading test is advantageous in that it can reduce test time. In contrast, the staged horizontal loading test is advantageous in that it enables the test data to be consistent with conventional data. When conducting the horizontal loading test, it is necessary to set appropriate loading patterns and the number of cycles, while taking into consideration the purposes, schedules and costs of the test (refer to **Table 3.10.2**).

Table 3.10.2 Types of Horizontal Loading Methods and Their Applicability[60)]

Loading method			Applicability
Loading pattern	Direction	Single direction	For a test primarily intended to confirm the coefficients of lateral subgrade reaction
			For a test where piles are subjected to unidirectional loads
		Reciprocal	For a test intended to examine the influences of reciprocal horizontal loads on piles
	Cycle	Single-cycle	For a test primarily intended to confirm the coefficients of lateral subgrade reaction
		Multi-cycle	For a test intended to obtain a wide variety of information, including the loads when the ground or pile bodies start to undergo plasticization
Loading method		Continuous	For a test primarily intended to confirm the coefficients of lateral subgrade reaction
		Staged	For a test when a possible deformation of piles is harmful to the safety of the test equipment
			For a test primarily intended to obtain data consistent with conventional data

(2) Points of caution when planning loading tests

① Planned maximum loads and planned maximum displacement

The planned maximum loads and displacement shall be appropriately set in accordance with the items to be investigated. When the horizontal loading test is conducted for directly measuring the pile resistance against a force perpendicular to axial directions, with the test piles subjected to horizontal loads under conditions identical to actual piles until the test piles undergo bending failure or the pile head displacement reaches the predetermined values, this test shall be planned so as to enable the horizontal loads to be applied to the piles until they undergo bending failure or have sufficiently large pile head displacement. However, this type of test has rarely been conducted because of the reason described in item (1) above. In many cases, the horizontal loading test is generally conducted with the acquisition of the information about ground reactions, such as the moduli of subgrade reaction, as the primary items to be investigated. Thus, in the horizontal loading test, the planned maximum loads are set in a manner that allows the test piles to undergo sufficient deflection to the depths where the ground reaction is expected to act on the actual piles.

② Test equipment

The horizontal loading test is conducted in a manner that either directly pushes the test pile with a hydraulic jack, as shown in **Fig. 3.10.31**, or pulls the test pile through a PC steel rod, as shown in **Fig. 3.10.32**. In either case, reaction piles are required.

It has been known that the deformation of the ground surrounding test piles which undergo deformation spreads across a wide area, particularly at the front of the test piles, based on the loading directions. Thus, the **Methods for Horizontal Pile Loading Tests and Commentaries**[61)] recommends conducting the horizontal loading test with no structures, earth fill, foundation piles, reaction piles or reference points located within the areas shown in **Fig. 3.10.33**. When conducting the horizontal loading test with the ground excavated, it shall be noted that the areas indicated in **Fig. 3.10.33** shall be leveled in principle.

③ **Loading and measuring methods**

It is necessary to select loading methods suitable for the test purposes with reference to **Table 3.10.2**. When focusing on obtaining data consistent with conventional data, the multi-cycle loading method is preferable.

The basic items to be investigated include the pile head loads, pile head displacement and strain in the pile bodies. The strain in the pile bodies is measured mainly to examine the distribution of the bending moment and the displacement of the pile bodies in depth directions. Thus, strain indicators are installed in multiple stages in the depth direction, and at the same depth, a pair of strain indicators are installed at axially symmetrical positions on the loading axis.

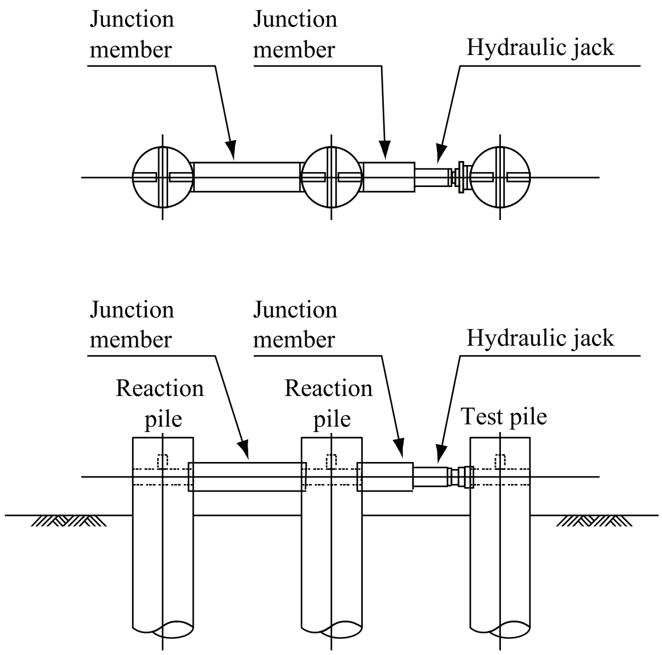


Fig. 3.10.31 Example of Loading Equipment When Pushing a Test Pile[62)]

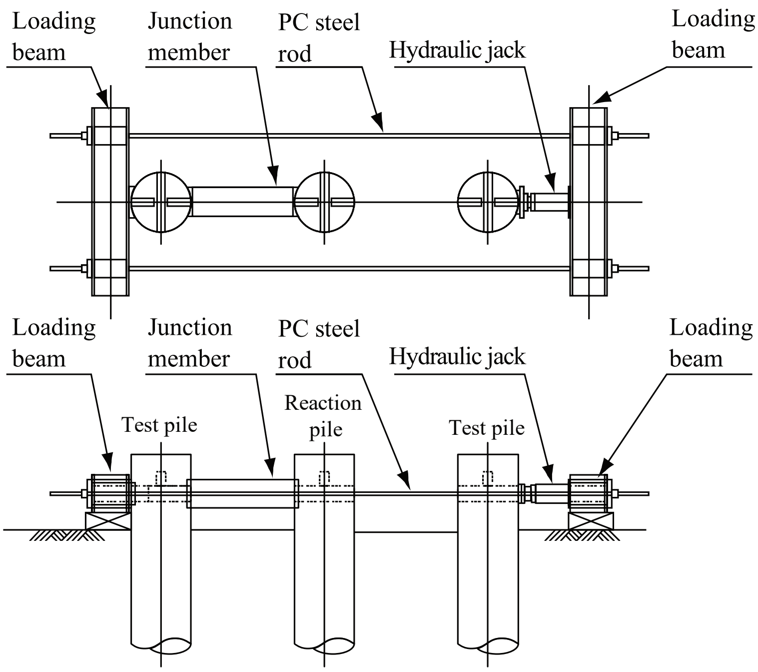


Fig. 3.10.32 Example of Loading Equipment When Pulling a Test Pile[62)]

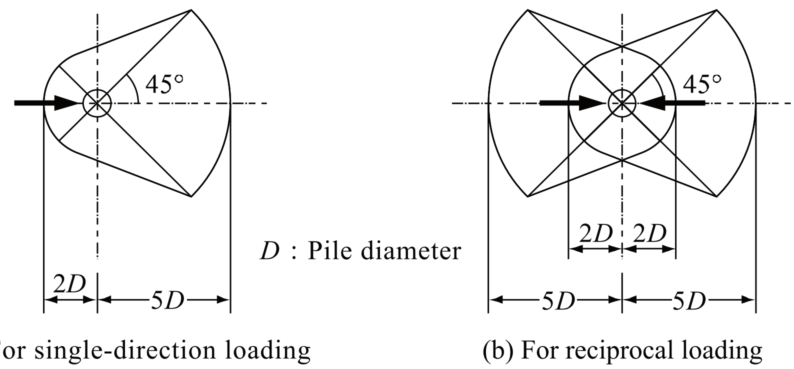


Fig. 3.10.33 Influence Range of a Test Pile[63)]

(3) Organization of test results

① **Characteristic values for direct evaluation of the resistance against a force perpendicular to axial directions**

When the horizontal loading test is conducted for directly measuring the pile resistance against a force perpendicular to axial directions, with the test piles subjected to horizontal loads under conditions identical to those of actual piles, the loads where the test piles undergo bending failure or pile head displacement reach the predetermined values can be the characteristic values for the pile resistance against a force perpendicular to axial directions.

② **Characteristic values for the deflection calculation of piles through the PHRI method**

The characteristic values of the ground lateral resistance coefficients can be obtained from the horizontal loading test results through an inverse analysis method specifically by calculating the deflection of piles using the PHRI method in accordance with the test conditions of the horizontal loading test (refer to **Part III, Chapter 2, 3.4.8 Calculation of the deflection of piles though the PHRI method**).

Fig. 3.10.34 shows the relationship, obtained through a horizontal loading test, between the loads and displacement at a pile head (loading point) plotted on double logarithm coordinates together with the calculation results of the relationship between the loads and displacement through the PHRI method, with the lateral resistance coefficients k_s of Type S ground changed from 500 kN/m$^{3.5}$ to 5,000 kN/m$^{3.5}$. According to **Fig. 3.10.34**, it can be determined that the horizontal test result agrees well with the calculation results of the PHRI method with the ground lateral resistance coefficients set at 1,300 kN/m$^{3.5}$, regardless of the magnitude of the loads, and that the value can be the characteristic values of the lateral resistance to be used in the design. In this way, the characteristic values of the ground lateral resistance to be used in the design and verification of the pile resistance against a force perpendicular to axial directions using the PHRI method can be determined from the horizontal loading test results.

③ **Characteristic values of the coefficients of lateral subgrade reaction used in Chang's method for the calculation deflection of the piles**

The characteristic values of the coefficients of lateral subgrade reaction can be obtained from the horizontal loading test results through an inverse analysis method specifically by calculating the deflection of piles using Chang's method in accordance with the test conditions of the horizontal loading test (refer to **Part III, Chapter 2, 3.4.7 Calculation of the deflection of piles though Chang's method**).

Fig. 3.10.35 shows the relationship, obtained through a horizontal loading test, between the loads and displacement at the pile head, together with the calculation results of the relationship between the loads and displacement through Chang's method, with the coefficients of the lateral subgrade resistance k_{CH} changed from 1,500 kN/m^3 to 10,000 kN/m^3. As can be seen in **Fig. 3.10.35**, the horizontal loading test results show a curved relationship between the loads and displacement at the pile head. On the other hand, the calculation results of Chang's method, with a constant coefficient of lateral subgrade reaction, show linear relationships between loads and displacement. This is because Chang's method does not consider the nonlinearity of ground. Generally, the characteristic values of the coefficients of lateral subgrade reaction to be used in the design and verification of the pile resistance against a force perpendicular to axial directions using Chang's method can be practically determined from **Fig. 3.10.35**, with the loads or displacement of the pile heads set to those expected for the actual piles.

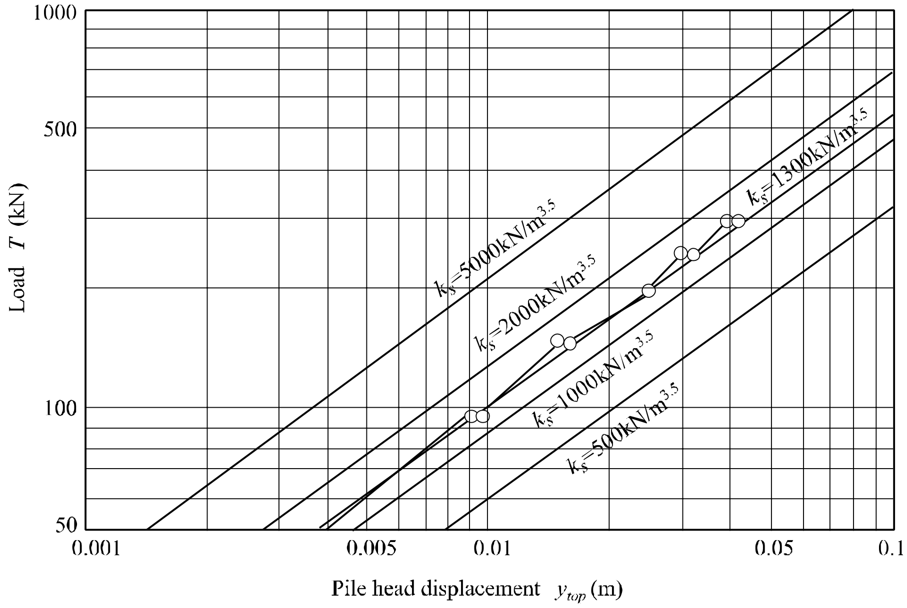


Fig. 3.10.34 Example of Determining the Ground Lateral Resistance Coefficient from the Horizontal Loading Test Results[64)]

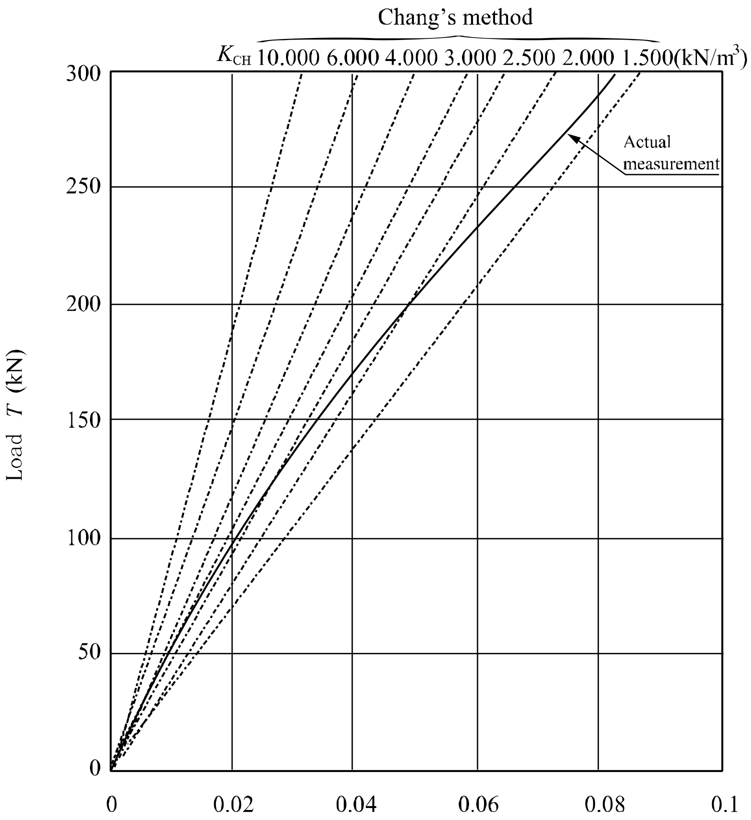


Fig. 3.10.35 Example of Determining the Coefficient of Lateral Subgrade Reaction from the Horizontal Loading Test Results

3.10.10 Dynamic Bearing Capacity Management Equations

(1) Outline of dynamic bearing capacity management equations

Dynamic bearing capacity management equations are used to obtain the maximum static axial force of piles through dynamic penetration resistance. In the past, dynamic bearing capacity management equations were considered as a type of loading test in a broad sense, but that is no longer the case. The dynamic bearing capacity management equations have very low accuracy for estimating the pile resistance against a pushing force in axial directions. It has been known that the estimation results of the dynamic bearing capacity management equations widely deviate from the loading test results, as can be seen in **Fig. 3.10.36**. Thus, the pile resistance against a pushing force in axial directions shall not be estimated based on the calculation results of the dynamic bearing capacity management equations.

The dynamic bearing capacity management equations can be used only for the purpose of supervising piling work in a manner that obtains the relative differences in axial resistance of each pile constructed in large quantities at identical construction sites with almost identical ground properties, or determines the completion of the driving operation of each pile.

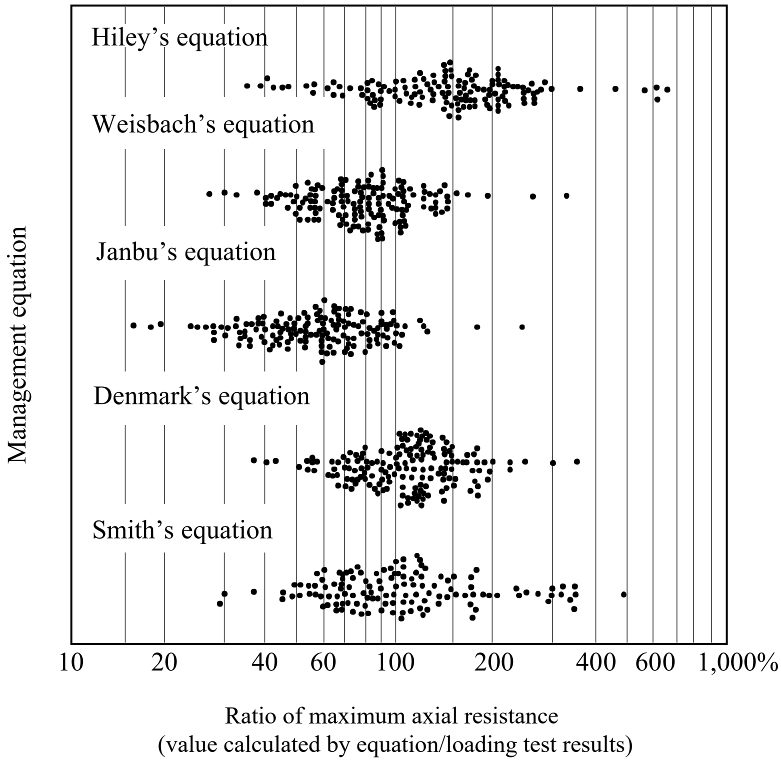


Fig. 3.10.36 Comparison of the Dynamic Bearing Capacity Management Equations and Loading Test Results (Modification to the Material by Sawaguchi[65)])

(2) Hiley's equation

Hiley's equation has been widely used as the dynamic bearing capacity management equation in the construction of port facilities and is derived from the assumption that the impact energy applied to piles through hammers is equivalent to the work done by the piles when they are penetrated into the ground. For steel piles, a simplified **equation (3.10.2)** based on certain assumptions is called Hiley's equation and utilized.[66)]

$$R_d = \frac{e_f \cdot F}{S + \frac{K}{2}} \qquad \textbf{(3.10.2)}$$

where

R_d : the maximum dynamic axial resistance of the pile (kN);

S : the penetration length (m);

K : the rebound rate (m);

F : the impact energy (kN·m); and

e_f : the hammer efficiency (generally 0.5).

The penetration length S is the residual settlement of the pile head by a single blow of a hammer, and the rebound rate K is a value obtained by subtracting the penetration length from the maximum settlement of the pile head during the blow. The values used for the penetration lengths and rebound rates are generally the averages of the last five to 10 blows and the last 10 to 20 blows before the completion of the respective driving work in the case of using drop hammers and types other than drop hammers, respectively. Here, when supervising pile driving work, it is preferable not to focus only on the calculation results from Hiley's equation, but to evaluate the changing trends of the penetration lengths and the rebound rates with respect to the pile tip depths, and the relationships between these factors and the hammer impact energy.

There has been a proposal of a method[67)] for correcting Hiley's equation using loading test results, but it shall be noted that the method has been proposed not for the improvement of the estimation accuracy of Hiley's equation, but for the correction of the calculation results only.

3.11 Field Observation

3.11.1 General

(1) Definition of a field observation

When constructing earth fill and structures on relatively soft ground, there are cases of phenomena in which the earth fill and structures settle or the ground moves sideways. In contrast, when excavating soft ground, there are cases of phenomena in which excavation faces swell or excavated bottoms rise upward.

A field observation is defined as a time-series measurement of the displacement and stresses of the structure bodies to be constructed, as well as their foundation ground, surrounding ground and neighboring structures, during and after construction, to achieve the purposes described in Item (2) below.

(2) Purposes of the field observation

When building structures on ground or excavating ground, it is necessary to ensure the safety of the structures, even for temporary construction, and to minimize the effects of the construction on neighboring structures and facilities. It is also necessary to observe the settlement and displacement of the structures under construction and compare the measurements with the designed values before construction so as to review the design of the structures and improve safety. The measurements of settlement and displacement of the structures under construction are necessary for making accurate predictions of future settlement and displacement after the construction and utilizing the prediction results to facilitate maintenance of the structures. Furthermore, there has been an increasing necessity to confirm the safety of the structures which have undergone deformations due to aging or damage caused by disasters such as earthquakes.

The purposes of the field observation are as follows:

① Confirmation of structure safety during construction, including temporary work;

② Understanding of the influence of ongoing excavation or ground improvement work on neighboring structures;

③ Verification of the settlement and displacement of structures estimated before construction, and feedback on the verification results for the design and construction;

④ Prediction of future settlement and displacement after the completion of the structures and utilization of the prediction results for the maintenance program of the structures;

⑤ Confirmation of the safety of structures undergoing deformation; and

⑥ Prevention of secondary disasters due to progressive destruction of structures damaged by earthquakes.

The progressive destruction of structures damaged by earthquakes includes the sand washing-out of quaywalls, which causes the ground surfaces behind them to cave in.

(3) Methods for the field observation

A field observation is a time-series measurement of the displacement and stresses of the structure bodies to be constructed, as well as their foundation ground, surrounding ground and neighboring structures, during and after construction. The field observation can be divided into the measurement of displacement, such as the settlement of the structure bodies, settlement as well as lateral flow of the ground and the measurement of earth pressure to the structures and stresses of the structures. **Table 3.11.1** summarizes the major observation items and general observation equipment and methods.

Table 3.11.1 Major Items and General Equipment and Methods for Field Observations

Observation item		Observation equipment and method
Structure	Displacement	Clinometer (multistage and insertion types) and survey
	Load and stress	Load meter, earth pressure gauge, hydraulic gauge and strain indicator
Ground	Displacement	Settlement plate, settlement gauge (stratified and hydraulic types) and clinometer
	Stress	Earth pressure gauge and piezometer
Other		Hydraulic gauge and vibration gauge

(4) Plans for the field observation

Examples of coastal construction projects requiring field observations are as follows.

For projects that require the implementation of large-scale reclamation in a short period of time, as was the case for the construction of Kansai International Airport and the D-Runway at Tokyo International Airport, field observations are conducted to stabilize the structures and accurately predict future settlement, along with the construction of revetments and the development of reclamation land.[68] Field observations are particularly necessary when implementing the design and construction if there is an expectation for the enhancement of consolidation effects by surcharge loads in order to manage the stability and settlement of the structures during construction.

Field observations are also conducted as needed when constructing onshore approach sections and ventilation towers for undersea tunnels by excavating land areas on a large scale.

There are many cases of conducting a field observation for the construction of new types of quaywalls and revetments to acquire basic information to confirm the stability of the structures and establish the design methods. For example, for the construction of an earthquake-resistant quaywall (-9 m) at the East Port District in Kushiro Port, a field observation was conducted to confirm the stability of a freestanding earth retaining wall made of stabilized bodies constructed through ground improvement along the shoreline.[69] In addition, for the construction of an underwater strut-type steel quaywall, also at the East Port District in Kushiro Port, a field observation was conducted for acquiring basis information to confirm stability and establish the design method of the structure.[70] Over the course of this field observation, the measurement of the mechanical behavior of the entire system of the steel structure and the ground, including the backfill sections, was conducted for a portion of the quaywall. For the construction of a piled pier for container cargoes to which a jacket structure was applied at Oi Wharf in Tokyo Port, a plan for an earthquake field observation was established to observe the behavior of the structure during an earthquake.[71]

Furthermore, there have been cases of conducting field observations to monitor the displacement and deformation of piles when rubbles are dropped between the steel pipes in a manner that makes rubble mound beneath the piled piers.

For the above situations, it is difficult to uniformly define the scope of the field observations because they have been conducted in a variety of ways in terms of the locations, purposes, methods, frequency and assessment methods of the observation results. Nonetheless, field observations shall be conducted while focusing on the general points of caution listed below:

- Early confirmation of the presence or absence of any problems with neighboring structers or facilities;
- Consensus formation on supervising and management plans among the project owners, designers and contractors before starting construction;

- Implementation of appropriate management of displacement and stresses with predetermined management values depending on the observation purposes;
- Preliminary deliberation of the measures to be taken when values exceeding the management values are observed;
- Preliminary deliberation of the methods to feed back the deviation of observation values from those that were predicted to the design and construction;
- Establishment of observation plans which can be flexibly modified in accordance with the observation results; and
- Provision of precautionary measures for measuring equipment prone to damage during construction; for example, preliminary installation of the auxiliary measuring equipment.

In **Reference (Part II), Chapter 1, 3.11.2 Examples of Field Observations** below, two successful cases are introduced to explain how the field observations were planned and effectively conducted.

3.11.2 Examples of Field Observations

(1) Field observation for an undersea road tunnel project

① Outline of the undersea road tunnel project

This road was planned to be a new port road crossing underneath D Bay. The construction of the phase 1 zone, with a total length of 1,181 m (including an immersed tunnel section of 557 m, made up of seven immersed tunnel elements), a W District side block of 323 m (made up of earth retaining wall and onshore tunnel sections), and a T District side block of 266 m (made up of earth retaining wall and onshore tunnel sections) began construction in 2000 and was completed in 2011.

The undersea tunnel is used exclusively for automobiles and has the following unique characteristics compared to other tunnels: 1) all the immersed tunnel elements have curved structures; 2) vertical shafts (ventilation towers) are not required because of the short length of the tunnel sections; 3) the application of a rigid joint structure using stretchable waterproof rubber for all joints of immersed tunnel elements; and 4) the development and use of filling concrete with intermediary fluidity tailored to make the immersed tunnel elements.[72)]

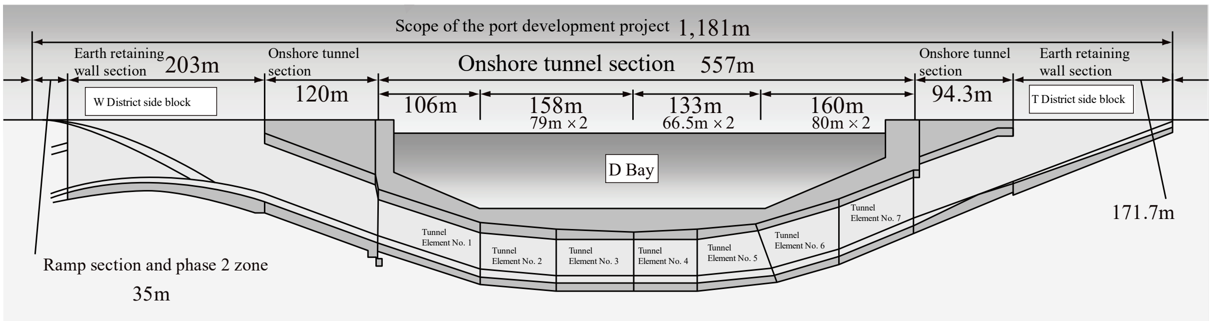


Fig. 3.11.1 Schematic Drawing of the Undersea Road Tunnel

② Structure of the road tunnel

(a) Immersed tunnel section

Each immersed tunnel element has a full-sandwich hybrid structure constructed by first fabricating a steel shell as an element body and then placing filling concrete in the internal cavity. Because the horizontal alignment of the road tunnel has a curved section in the water area of D Bay, the immersed tunnel elements have curved planar shapes. Furthermore, the longitudinal gradients of the respective elements also differ. Thus, all seven of the immersed tunnel elements have different structural shapes.[73)] Using Tunnel Element No. 1 as an example, the element has a width of about 27.9 m, a height of 8.4 m, a length of 106 m and a weight of about 24,000 tons. The cross-sectional view of the immersed tunnel section is shown in **Fig. 3.11.2**.

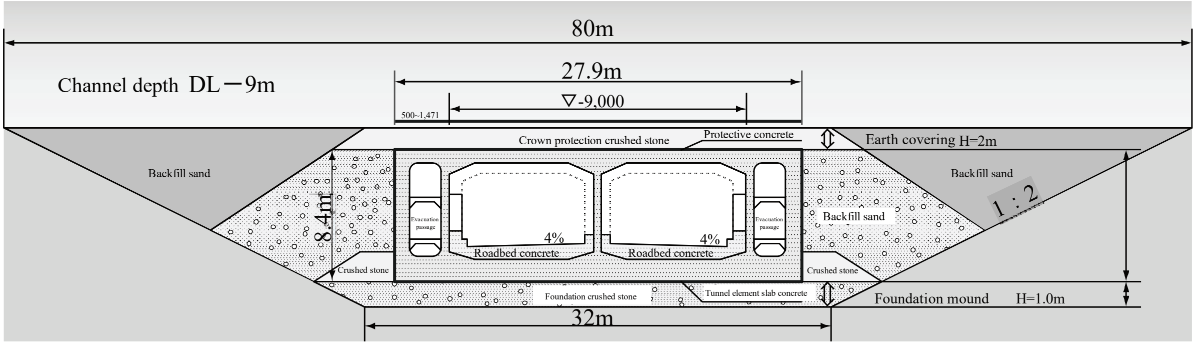


Fig. 3.11.2 Cross-Sectional View of the Immersed Tunnel Section

(b) Onshore tunnel and earth retaining wall sections

The onshore tunnel sections for both the W and T District side blocks have the structure of box culverts constructed through the open-cut method. The earth retaining wall section for the W District side block comprises four pneumatic caissons, a U-shaped retaining wall constructed using the open-cut method and a ramp used as the approach to the phase 1 zone. In addition, the earth retaining wall section for the T District side block comprises a U-shaped retaining wall constructed using the open-cut method. Aerial photographs of the onshore portions of the W and T District side blocks are shown in **Figs. 3.11.3** and **3.11.4**, respectively.

For the construction of the onshore tunnel sections using the open-cut method, field observations were conducted for the excavation and structural work.

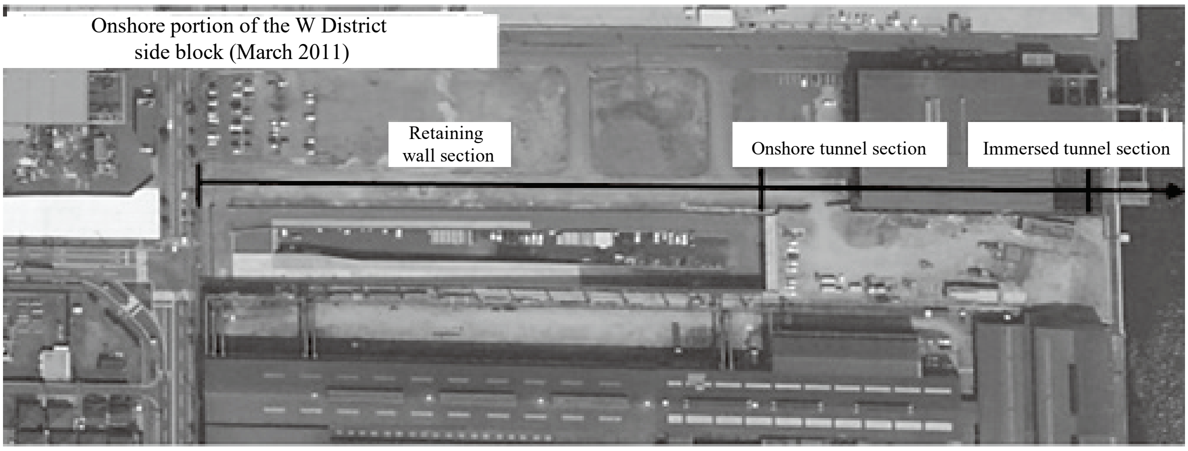


Fig. 3.11.3 Aerial Photograph of the Onshore Portion of the W District Side Block

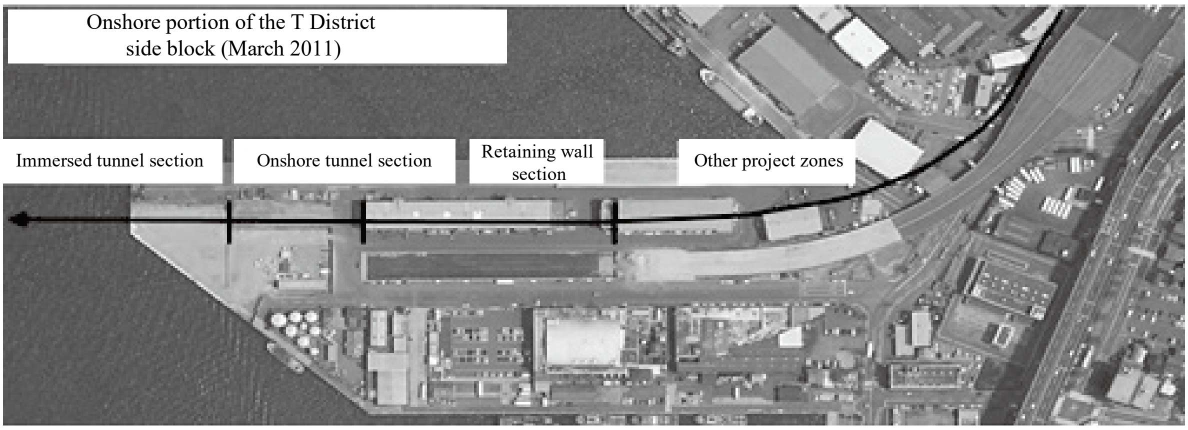


Fig. 3.11.4 Aerial Photograph of the Onshore Portion of the T District Side Block

③ **Field observation for onshore tunnel sections**

The construction sites of the onshore tunnel sections are reclaimed land facing D Bay with factories that have precision machines and overhead traveling cranes located close to the construction site at the W District side block, and gas conduit pipes and tanks located close to the construction site in the T District side block. Thus, the challenge for construction of the onshore tunnel sections was to make sure that the construction activities did not create any obstacles for the operation of these factories and facilities, as well as ensure the safety of the excavation sections during open-cut work.

The purposes of the field observation for the onshore tunnel sections were to: 1) confirm the safety during the excavation and structure work, including temporary work; and 2) identify the effects from the excavation and earth retaining wall construction on the neighboring factories that have precision machines.

In particular, the effects from the construction of the onshore tunnel sections on neighboring facilities were examined by collecting, organizing and analyzing the data on the behavior of the earth retaining walls and the surrounding ground during the construction of the earth retaining walls and other tunnel-related facilities based on the “System for the supervision of the construction of the W District-side onshore tunnel through field observation” established in 2001. When constructing the pneumatic caissons, the caissons were lowered to the designated positions while measuring in real time the positions of the structure bodies, the actions put on them and the behavior of the surrounding facilities through supervision using information technology.[74)]

(a) Field observation items

Table 3.11.2 summarizes the targets, items, equipment and purposes of the field observation for the onshore tunnel sections.

Table 3.11.2 List of Field Observation Items

Target	Item	Equipment	Purpose
Earth retaining wall	Deformation of steel pipe column earth retaining wall	Multistage clinometer	1. Observation of the displacement distribution of a column earth retaining wall; comparison of the measured, design and management reference values; and determination of the safety of the earth retaining wall 2. Observation of the settlement of the surrounding ground to acquire data to confirm the effects on neighboring structures
		Insertion type clinometer	1. Observation of the displacement and bending moment distribution of a column earth retaining wall; comparison of the measured, design and management reference values; and determination of the safety of the earth retaining wall 2. Acquisition of input data for prediction analyses (next stage prediction)

Target	Item	Equipment	Purpose
	Stress of steel pipe column earth retaining wall	Strain gauge (vertical direction)	1. Observation of the bending stress distribution of steel pipes on a column earth retaining wall; comparison of the measured, design and management reference values; and determination of the safety of the earth retaining wall 2. Acquisition of data to understand the behavior of the earth retaining wall by comparing the prediction analysis results with measured values and confirming the differences between the measured values and analysis results
	Earth pressure acting on steel pile column earth retaining wall (excavation side and back side)	Earth pressure gauge	1. Observation of the lateral pressure distribution acting on a column earth retaining wall; comparison of the designed and measured values of the external force; and determination of the safety of the retaining wall at the respective excavation stages 2. Acquisition of data to understand the behavior of the retaining wall by comparing the prediction analysis results with measured values and confirming the differences between the measured values and analysis results
	Water pressure acting on steel pile column earth retaining wall (excavation side and back side)	Piezometer	1. Observation of the water pressure distribution acting on a column earth retaining wall; verification of the designed water pressure distribution; and acquisition of the changes in the values calculated by (measured lateral pressure) – (measured water pressure) = (effective earth pressure)
Timbering	Axial force on short strut	Short strut strain indicator	1. Observation of the axial force of short struts due to excavation; comparison of the measured, design and management reference values; and determination of the safety of the short struts
	Axial force on ground anchor	Load meter	1. Observation of the behavior of the axial force of ground anchors due to excavation; comparison of the measured, design and management reference values; and determination of the safety of the ground anchors
Excavated ground	Rebound rate of excavated ground	Stratified settlement gauge	1. Observation of the rebound rates of the ground due to excavation; predictions for swelling and boiling; and deliberation of preventive countermeasures against swelling and boiling
Surrounding ground	Horizontal displacement of ground behind earth retaining wall	Insertion type clinometer	1. Observation of the influences of the displacement of an earth retaining wall due to excavation on the surrounding ground by depth 2. Preliminary identification of the influences on the surrounding structures, together with deliberation of data measured with equipment installed on the surrounding structures; and estimation of the influences on these structures
	Settlement of ground behind earth retaining wall	Ground surface settlement gauge	1. Observation of the influences of the displacement of an earth retaining wall due to excavation on the surrounding ground through the vertical displacement of the ground surface 2. Preliminary identification of the influences on the surrounding structures, together with deliberation of data measured with equipment installed on the surrounding structures; and estimation of the influences on these structures
	Groundwater level	Groundwater level gauge	1. Observation of the fluctuations in ground water levels due to excavation; and confirmation of the cut-off performance of the steel pipe column earth retaining wall
Existing structures	Settlement and swelling of existing structures	Water leveling type settlement gauge	1. Observation of the vertical displacement of the existing structures due to excavation; comparison between the measured and management reference values; and determination of the safety of the existing structures 2. Prompt implementation of countermeasures when the measured values exceed the management reference values

Target	Item	Equipment	Purpose
	Inclination of existing structures	Installation type clinometer	1. Observation of inclinations of the existing structures; comparison between the measured and management reference values; and determination of the safety of the existing structures 2. Prompt implementation of countermeasures when the measured values exceed the management reference values

(b) Locations and amounts of observation equipment

The equipment used for field observations is classified into equipment for observing the earth retaining timbering and equipment for observing the surrounding ground and structures. **Tables 3.11.3** to **3.11.5** summarize the observation items, equipment used for field observations, and locations and amounts of equipment. One of the characteristics of the field observations in undersea road tunnel construction is the use of a variety of equipment to observe the effects of the construction on neighboring factories.

Fig. 3.11.5 shows the layout of the onshore portion of the W District side block with the locations of the observation equipment. The cross-sectional view of the onshore tunnel section in the W District side block is shown in **Fig. 3.11.6** (① Cross Section No. 66 + 04 in **Table 3.11.3**).

Table 3.11.3 Locations and Amounts of Observation Equipment (Used for the Field Observation of Earth Retaining Timbering at the Onshore Tunnel Section in the W District Side Block)

Observation item/equipment			Field observation location in the onshore tunnel section								Total
Observation item	Object cross section		Seaside	① Cross Section No. 66 + 04		② Cross Section No. 67 + 18		③ Cross Section No. 69 + 18		④ Cross Section No. 72 + 00	
	Location of earth retaining wall		End side	Company A side	Company B side	Company A side	Company B side	Company A side	Company B side	Company B side	
	Construction method of earth retaining wall		Steel pipe sheet pile method						ONS method		
Deformation of earth retaining wall	Multistage clinometer		15 units	15 units				14 units			44 units
	Insertion type clinometer			1 piece 30.5 m, manual	1 piece 30.5 m, automatic		1 piece 28.2 m, automatic	1 piece 28.2 m, manual	1 piece 26.7 m, automatic	1 piece 28.2 m, automatic	6 pieces 172.3 m
Deformation stress of earth retaining wall	Strain gauge		30 units	30 units	30 units		28 units	28 units	26 units	28 units	200 units
External force on earth retaining wall	Earth pressure gauge	Excavated side		4 units	4 units						8 units
		Back side		4 units	4 units					3 units	11 units
	Water pressure gauge	Excavated side		4 units	4 units	1 unit	1 unit	1 unit	1 unit		12 units
		Back side		4 units	4 units	1 unit	3 units	3 units	3 units	3 units	21 units
Axial force on short strut	Strain indicator (with thermometer)				16 units		10 units		10 units	12 units	48 units
Tension on anchor	Load meter		27 units								27 units
Rebound rate of excavated ground	Stratified settlement gauge						1 location (2 units: 28.2 m)		1 location (2 units: 28.2 m)		2 locations (4 units: 56.4 m)
Horizontal displacement of ground behind earth retaining wall	Insertion type clinometer		2 pieces 30.5 m, automatic		1 piece 22.5 m, automatic		5 pieces 28.2 m, manual				8 pieces 224.5 m
Settlement of ground behind earth retaining wall	Ground surface settlement gauge				1 unit, manual		5 units, manual				6 units
Groundwater level of ground behind earth retaining wall	Groundwater level gauge				1 unit 22 m		1 unit 9 m	1 unit 24 m	1 unit 7 m		4 units 62 m
Amount of equipment	Automatic equipment		72 units + 2 pieces	61 units	63 units + 2 pieces	2 units	45 units + 1 piece	47 units	43 units + 1 piece	46 units + 1 piece	379 units + 7 pieces
	Manual equipment			1 piece	1 unit		5 units + 5 pieces	1 piece			6 units + 7 pieces

Table 3.11.4 Locations and Amounts of Observation Equipment (Used for the Field Observation of the Surrounding Ground and Structures at the Onshore Tunnel Section in the W District Side Block)

Observation item	Location of equipment	Company B								Subtotal
		NC plasma cutter	NC primer cleaner	Acid water facility (tank)	NC drill press	Portal crane foundation (2.8 tons)	Overhead travelling crane foundation	Building foundation	Passage near No. 67 + 18	
Settlement of structure	Water leveling type settlement gauge	5 units	6 units		4 units					15 units
	Reference tank	1 unit	1 unit		1 unit					3 units
Inclination of structure	Clinometer (two directions)	4 points (2 units)	4 points (2 units)	4 points (2 units)	4 points (2 units)			8 points (4 units)		24 points (12 units)
	Clinometer (one direction)					7 units	6 units			13 units
Horizontal displacement of ground	Insertion type clinometer								5 pieces 139.5 m	5 pieces
Amount of equipment	Automatic equipment	9 units	10 units	4 units	8 units	7 units	6 units	8 units		52 units
	Manual equipment								5 pieces	5 pieces

Observation item	Location of equipment	Company A	Subtotal	Total
		Overhead travelling crane foundation		
Settlement of structure	Water leveling type settlement gauge			15 units
	Reference tank			3 units
Inclination of structure	Clinometer (two directions)			24 points (12 units)
	Clinometer (one direction)	6 units	6 units	19 units
Horizontal displacement of ground	Insertion type clinometer			5 pieces
Amount of equipment	Automatic equipment	6 units	6 units	58 units
	Manual equipment			5 pieces

Table 3.11.5 Locations and Amounts of Observation Equipment (Used for the Field Observation of the Earth Retaining Wall Section in the W District Side Block)

Observation item	Equipment name	Model	Capacity	Conversion method (detection principle)	Section 1						Section 2			
					U1		U2		U3		U4		U5	
					Cable length (equipment – scanner)	Quantity	Cable length (equipment – scanner)	Quantity	Cable length (equipment – scanner)	Quantity	Cable length (equipment – scanner)	Quantity	Cable length (equipment – scanner)	Quantity
ONS pile deformation	Automatic insertion type clinometer	BKA-1000A	±10°	Servo type	205 m (control cable)	1 (existing)								
ONS pile stress	Strain gauge	KFG-5-350	50000 μ	Strain gauge		28 (existing)								
Earth pressure	Earth pressure gauge	GTI-E210	500 KPa 1 MPa	Strain gauge		3 (existing)								
Pore water pressure	Piezometer	BP-B	200 KPa 500 KPa	Strain gauge		3 (existing)								
Strain on short strut	Strain indicator	BS-8FT	1000 μ	Strain gauge		12 (existing)								
SMW Core material deformation	Automatic insertion type clinometer	BKA-1000A	±10°	Servo type			155 m (control cable)	1			120 m (control cable)	1		
SMW Core material deformation	Insertion type clinometer	BK-5G	±5°	Strain gauge						3				3
Deformation of overhead crane foundation	Clinometer (one direction)	BKK-A-1	±1°	Strain gauge		2 (existing)	25 m	3 (1 existing unit)	105 m	3	60 m	3	147 m	4
Deformation of CO_2 tank foundation	Clinometer (two directions)	BKK-A-1-D	±1°	Strain gauge									248 m	2
Fireproof water tank	Clinometer (two directions)	BKK-A-1-D	±1°	Strain gauge									220 m	2

Observation item	Equipment name	Model	Capacity	Conversion method (detection principle)	Section 1						Section 2			
					U1		U2		U3		U4		U5	
					Cable length (equipment – scanner)	Quantity	Cable length (equipment – scanner)	Quantity	Cable length (equipment – scanner)	Quantity	Cable length (equipment – scanner)	Quantity	Cable length (equipment – scanner)	Quantity
Groundwater level	Water level gauge	BWL-20MET	20 m	Strain gauge			5 m	1			5 m	1		
Rebound rate of excavated ground	Rebound meter	SVL	±2m	Magnetic type						1 (2 stages)				1 (2 stages)
Observation equipment	Scanner	USB-70A-30				1 (existing)				1				1
Observation equipment	Data logger	JCAM-60A-AC								1				1
Observation equipment	Personal computer	Including display								1				1
Total						49	30 m	5	105 m	Automatic: 3 Manual: 3	65 m	Automatic: 5	615 m	Automatic: 8 Manual: 3
Scanner cable length (scanner – equipment room)					205 m		155 m				120 m			

* Automatic insertion type clinometers were relocated to the optimal observation locations in accordance with the construction stages.

* Insertion type clinometers (manual) were used when conducting arbitrary observations at the locations with automatic insertion type clinometer guide pipes.

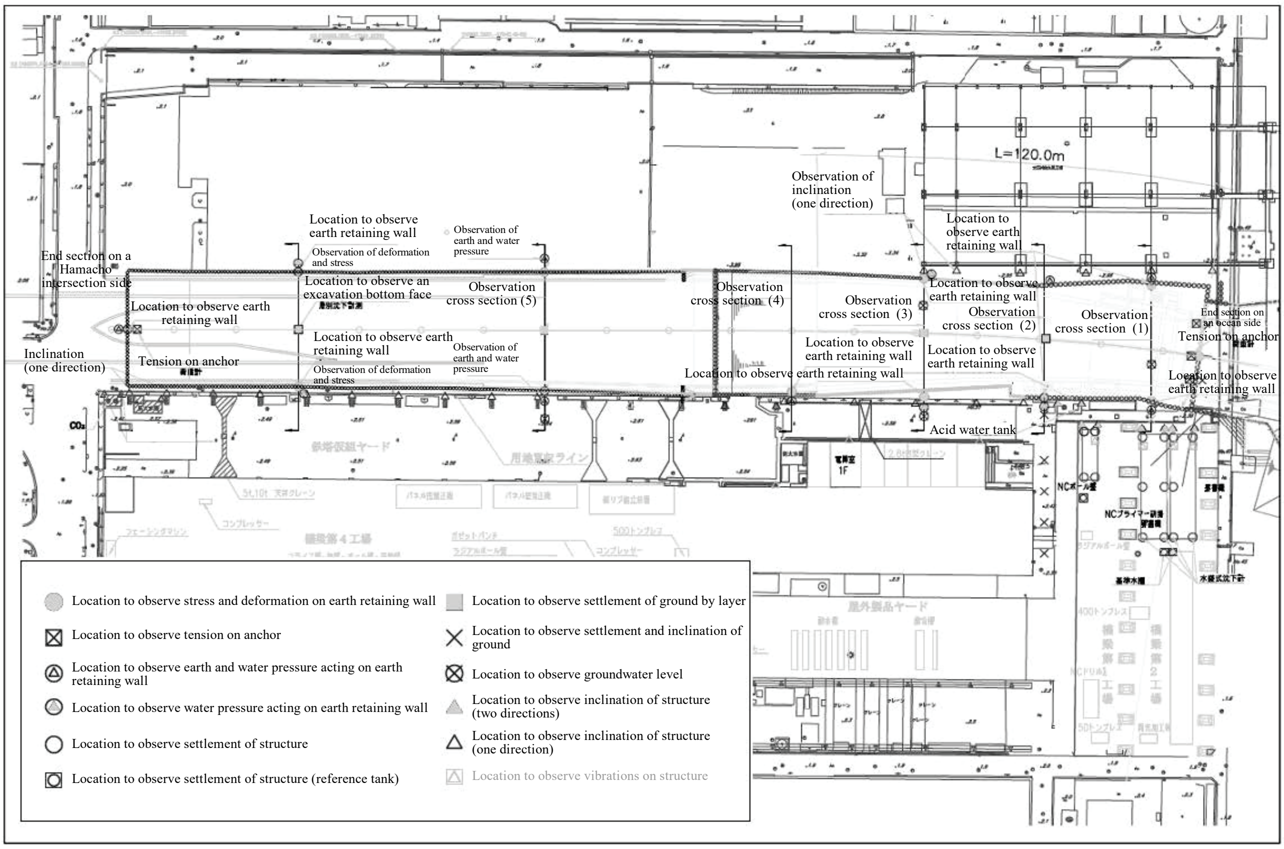


Fig. 3.11.5 Layout of Observation Equipment (Onshore Tunnel and Earth Retaining Wall Sections in the W District Side Block)

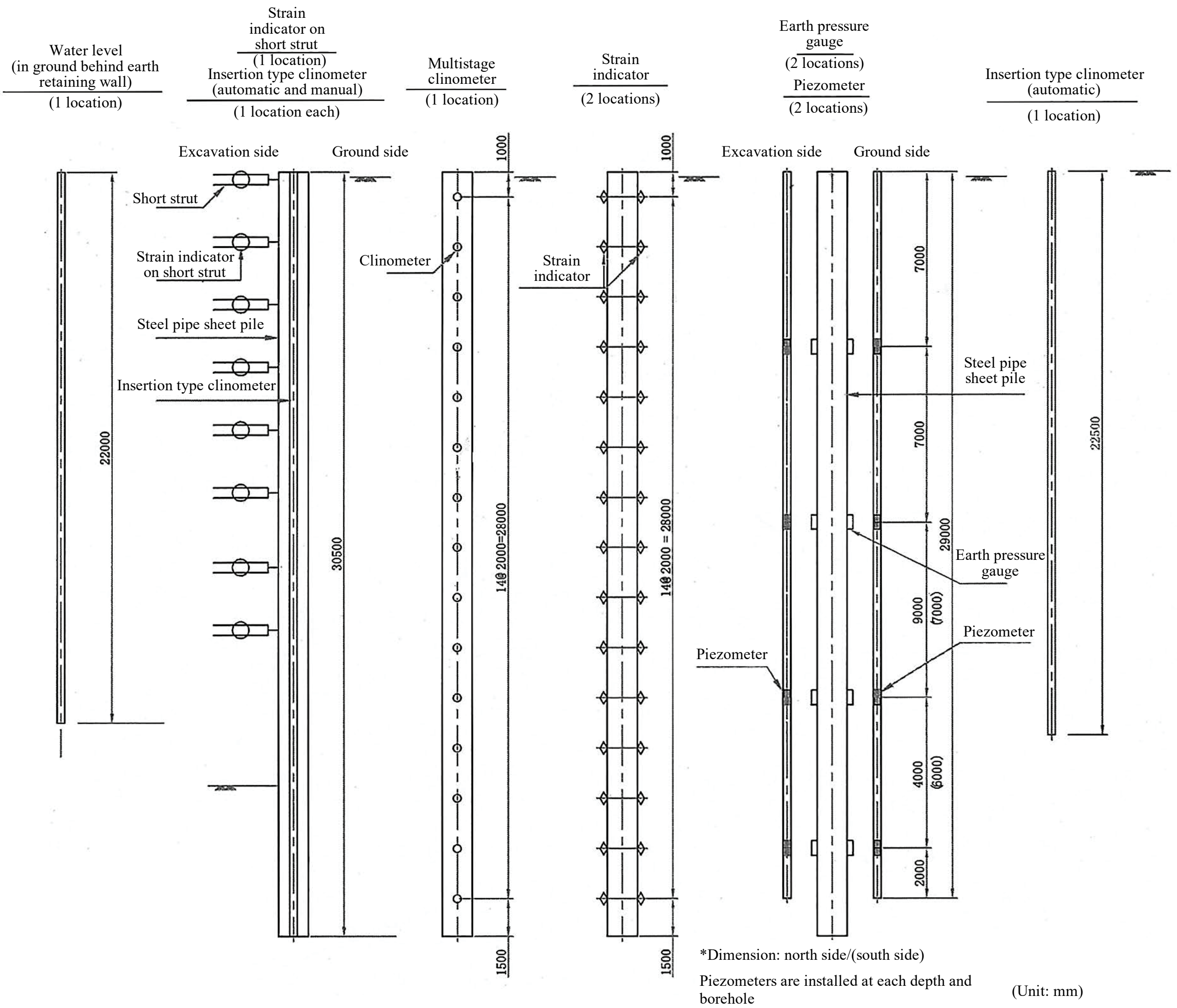


Fig. 3.11.6 Cross-Sectional View of Observation Equipment Locations (Onshore Tunnel Section in the W District Side Block: ①Cross Section No. 66 + 04 in **Table 3.11.3**)

(c) Management reference values

For the onshore tunnel section in the W District side block, where there are possible influences from the construction on the neighboring factories, the tunnel construction was implemented while conducting a field observation with the equipment installed at the factories of two neighboring companies, Companies A and B, so that the daily measurements of the equipment did not exceed the observation management reference values.

Because the concerns during the construction of the earth retaining wall were mainly in regard to the effects on the precision machines and overhead travelling crane at Company B’s factory, daily field observations were conducted for the following facilities with the management reference values (design, primary management and secondary management values) set appropriately.

The observation items for which the management reference values were set included the settlement of the earth floor at the factory, where the precision machines are installed, and the inclination and level differences of the rails for portal and overhead travelling cranes. These management reference values were set through mutual consultation among the orderer, the contractor of the tunnel section and Company B. **Tables 3.11.6** to **3.11.9** show examples of the management reference values set for the respective facilities and equipment.

In the T District side block, there are gas conduit pipes and tanks near the construction site. Thus, the field observation was conducted with management reference values set for the displacement of these structures.

Table 3.11.6 Example of the Management Reference Values for the Onshore Tunnel Section in the W District Side Block (Related to the Earth Retaining Wall)

Observation cross section	Location	Observation item	Type of equipment	Allowable value (management limit value)	Designed value	Primary reference value	Secondary reference value
① No. 66 + 0.40	South side	Deformation of earth retaining wall	Automatic insertion type clinometer	50 mm	24.14 mm	19 mm	24 mm
		Stress on earth retaining wall	Strain gauge	210 N/mm^2	169 N/mm^2	135 N/mm^2	169 N/mm^2
		Moment of earth retaining wall		3422.35 KNm	2754.18 KNm	2203.34 KNm	2754.18 KNm
		Stress on first short strut	Short strut strain indicator	210 N/mm^2	67.0 N/mm^2	54 N/mm^2	67 N/mm^2
		Stress on second short strut	Short strut strain indicator	210 N/mm^2	68.0 N/mm^2	54 N/mm^2	68 N/mm^2
		Stress on third short strut	Short strut strain indicator	210 N/mm^2	131.0 N/mm^2	105 N/mm^2	131 N/mm^2
		Stress on fourth short strut	Short strut strain indicator	210 N/mm^2	85.0 N/mm^2	68 N/mm^2	85 N/mm^2
		Stress on fifth short strut	Short strut strain indicator	210 N/mm^2	69.0 N/mm^2	55 N/mm^2	69 N/mm^2
		Stress on sixth short strut	Short strut strain indicator	210 N/mm^2	66.0 N/mm^2	53 N/mm^2	66 N/mm^2
		Stress on seventh short strut	Short strut strain indicator	210 N/mm^2	59.0 N/mm^2	47 N/mm^2	59 N/mm^2
	North side	Deformation of earth retaining wall	Multistage clinometer	50 mm	32.34 mm	26 mm	32 mm
			Manual insertion type clinometer	50 mm	32.34 mm	26 mm	32 mm
		Stress on earth retaining wall	Strain gauge	210 N/mm^2	169 N/mm^2	135 N/mm^2	169 N/mm^2
		Moment on earth retaining wall		3107.44 KNm	2500.75 KNm	2000.60 KNm	2500.75 KNm

- Primary reference value → 80% of designed value
- Secondary reference value → 100% of designed value
- Management limit value → Allowable value

Table 3.11.7 Example of Management Reference Values for the Earth Retaining Wall Section in the W District Side Block (Related to the Earth Retaining Wall)

Observation cross section	Observation item	Type of equipment	Allowable value (management limit value)	Designed value	Primary reference value	Secondary reference value
① U1-U2	Deformation of earth retaining wall	Multistage clinometer	50 mm	49.9 mm	40 mm	50 mm
	Stress on earth retaining wall	Strain gauge	210 N/mm^2	125 N/mm^2	100 N/mm^2	125 N/mm^2
	Moment on earth retaining wall H440		1162 KNm	693.5 KNm	554.8 KNm	693.5 KNm
	Stress on first angle brace	Angle brace strain indicator	210 N/mm^2	4.7 N/mm^2	4 N/mm^2	4.7 N/mm^2
	Stress on second angle brace	Angle brace strain indicator	210 N/mm^2	11.7 N/mm^2	9 N/mm^2	11.7 N/mm^2
	Stress on third angle brace	Angle brace strain indicator	210 N/mm^2	14.0 N/mm^2	11 N/mm^2	14.0 N/mm^2
	Stress on fourth angle brace	Angle brace strain indicator	210 N/mm^2	19.4 N/mm^2	18 N/mm^2	19.4 N/mm^2
	Stress on fifth angle brace	Angle brace strain indicator	210 N/mm^2	31.3 N/mm^2	25 N/mm^2	31.3 N/mm^2
	Stress on sixth angle brace	Angle brace strain indicator	210 N/mm^2	16.7 N/mm^2	13 N/mm^2	16.7 N/mm^2
	Stress on seventh angle brace	Angle brace strain indicator	210 N/mm^2	11.0 N/mm^2	9 N/mm^2	11.0 N/mm^2
	Stress on eighth angle brace	Angle brace strain indicator	210 N/mm^2	5.3 N/mm^2	4 N/mm^2	5.3 N/mm^2
	Stress on ninth angle brace	Angle brace strain indicator	210 N/mm^2	6.1 N/mm^2	5 N/mm^2	6.1 N/mm^2
	Stress on tenth angle brace	Angle brace strain indicator	210 N/mm^2	4.3 N/mm^2	3 N/mm^2	4.3 N/mm^2
	Stress on eleventh angle brace	Angle brace strain indicator	210 N/mm^2	15.4 N/mm^2	12 N/mm^2	15.4 N/mm^2
	Stress on twelfth angle brace	Angle brace strain indicator	210 N/mm^2	1.3 N/mm^2	1 N/mm^2	1.3 N/mm^2

Table 3.11.8 Management Reference Values for Facilities around the Onshore Tunnel Section in the W District Side Block

Item	Water leveling type settlement gauge (settlement of floor)			Clinometer			Level gauge (rails)		
	Primary reference value	Secondary reference value	Management limit value	Primary reference value	Secondary reference value	Management limit value	Primary reference value	Secondary reference value	Management limit value
1. Company B precision machines									
① NC drill press	1.8 mm	2.4 mm	3.0 mm	0.07°	0.10°	0.17°	1.6 mm	2.0 mm	3.0 mm
② NC primer cleaner	1.8 mm	2.4 mm	3.0 mm	0.07°	0.10°	0.17°	1.6 mm	2.0 mm	3.0 mm
③ NC plasma cutter	1.8 mm	2.4 mm	3.0 mm	0.07°	0.10°	0.17°	1.6 mm	2.0 mm	3.0 mm
2.0 mm + 0.8 mm (0.8 mm: temperature error) ○ Difference in rail settlement ±2.0 mm or less ○ Span length ±1.0 mm or less ○ Rail straightness ±1.0 mm or less/20 m				(Displacement of 2.0 mm at h = 1.0 m) (Manufacturer's guaranteed value)			(Displacement of 3.0 mm at h = 1.0 m) (Company B's management reference value)		
2. Portal crane (2.8 tons)	–	–	–	0.06°	0.08°	0.10°	6.0 mm	8.0 mm	10.0 mm
○ Difference in rail settlement ±23.0 mm (1/500 of span) ○ Span length ±10.0 mm ○ Meandering NA							(Displacement of 23 mm at h = 11.5 m) (Company B's management reference value)		
3. Overhead travelling crane (5.0 tons)	–	–	–	0.029°	0.038°	0.048°	3.0 mm	4.0 mm	5.0 mm
○ Difference in rail settlement ±49.5 mm (1/500 of span) ○ Span length ±10.0 mm (allowable value) ±20.0 mm (management reference value) ○ Meandering 2.5 mm/10 m							(Displacement of 10 mm at h = 12 m) (Company B's management reference value)		
4. Company A building	–	–	–	0.06°	0.08°	0.10°	6.0 mm	8.0 mm	10.0 mm
Although no management reference values were designated by Company A, the same values as with Company B's portal crane (2.8 tons) were set because there was a crane at Company A's factory.									

Table 3.11.9 Management Reference Values Related to the Facilities around the Earth Retaining Wall in the W District Side Block

No	Item	Company B management reference		Field observation (during edging wall, earth retaining wall and caisson installation work)					
		Management item	Management reference value	Observation item	Observation method and equipment	Management classification	Management limit value	Counter-measures to be taken in case of abnormalities	Remarks
1.	2.8-ton portal crane (with foundation repaired)	Difference in rail settlement	±23 mm (1/500 of span)	Cumulative inclination (foundation inclination angle)	Clinometer	Primary management	0.07°	Review of construction procedure and method	Preconstruction field inspection Confirmation of observation data on onshore tunnel section
						Secondary management	0.10° (displacement of 23 mm at h = 11.5 m)	Suspension of construction work and investigation of causes	
				Cumulative inclination (rail settlement)	Observation of levels	Primary management	5.0 mm	Review of construction procedure and method	
						Secondary management	10.0 mm	Suspension of construction work and investigation of causes	
		Span length	±10 mm						
		Meandering	NA						

No	Item	Company B management reference		Field observation (during edging wall, earth retaining wall and caisson installation work)					
		Management item	Management reference value	Observation item	Observation method and equipment	Management classification	Management limit value	Counter-measures to be taken in case of abnormalities	Remarks
2.	Overhead travelling cranes 5 tons and 10 tons (with 3 supports repaired and 12 supports unrepaired)	Difference in rail settlement	±49.5 mm (1/500 of span)	Cumulative settlement (settlement of foundation)	Observation of levels	Primary management	3.0 mm	Review of construction procedure and method	
						Secondary management	5.0 mm	Suspension of construction work and investigation of causes	
		Span length	±10 mm (allowable value) ±20 mm (management value)	Cumulative inclination (foundation inclination angle)	Clinometer	Primary management	0.033°	Review of construction procedure and method	
						Secondary management	0.048° (displacement of 10 mm at h = 12 m)	Suspension of construction work and investigation of causes	
		Meandering	2.5 mm/10 m	Relative horizontal displacement of foundation	Transit	Primary management	0.8 mm/10 m	Review of construction procedure and method	
						Secondary management	2.5 mm/10 m	Suspension of construction work and investigation of causes	
3.	CO_2 facility	Unknown		Cumulative settlement	Level	Primary management	20 mm	Review of construction procedure and method	The criteria to suspend the construction work were based on the immediate settlement limit values of buildings with RC concrete structures (rigid frame and wall structures) specified in the Design Standards for Building Foundation Structures and Commentaries. The immediate settlement limit values correspond to the limit states which cause destructive cracks.
						Secondary management	30 mm	Suspension of construction work and investigation of causes	
				Relative settlement	Level	Primary management	5 mm (0.021°)	Review of construction procedure and method	
						Secondary management	8 mm (0.028°)	Suspension of construction work and investigation of causes	
4.	Fireproof water tank	Unknown		Cumulative settlement	Level	Primary management	20 mm	Review of construction procedure and method	
						Secondary management	30 mm	Suspension of construction work and investigation of causes	
				Relative settlement	Level	Primary management	5 mm (0.021°)	Review of construction procedure and method	
						Secondary management	8 mm (0.028°)	Suspension of construction work and investigation of causes	

(d) Observation equipment

As shown in **Tables 3.11.3** to **3.11.5**, the equipment used in the field observation for the construction of the onshore tunnel section, the excavation work for the earth retaining wall and the building of structures was the equipment used in general excavation work, such as several types of clinometers, strain gauges, strain indicators, earth pressure gauges and water level gauges.

In contrast, the equipment used for the field observation of the influences of the onshore tunnel construction on the surrounding factories was as follows.

1) Equipment for the field observation of the surrounding ground

i. Manual insertion type clinometer

Manual insertion type clinometers made by Company C (BK-5G) were installed at depths from 20.5 m to 29.5 m with 50-cm intervals.

ii. Stratified and ground surface settlement gauges

Settlement gauges made by Company C (BJB-C-100S) were used to automatically acquire and organize data once a day. **Fig. 3.11.7** illustrates the installation state of a ground surface settlement gauge.

iii. Groundwater level gauge

Groundwater level gauges made by Company C (BWL-10MET) were used to automatically acquire and organize data once a day.

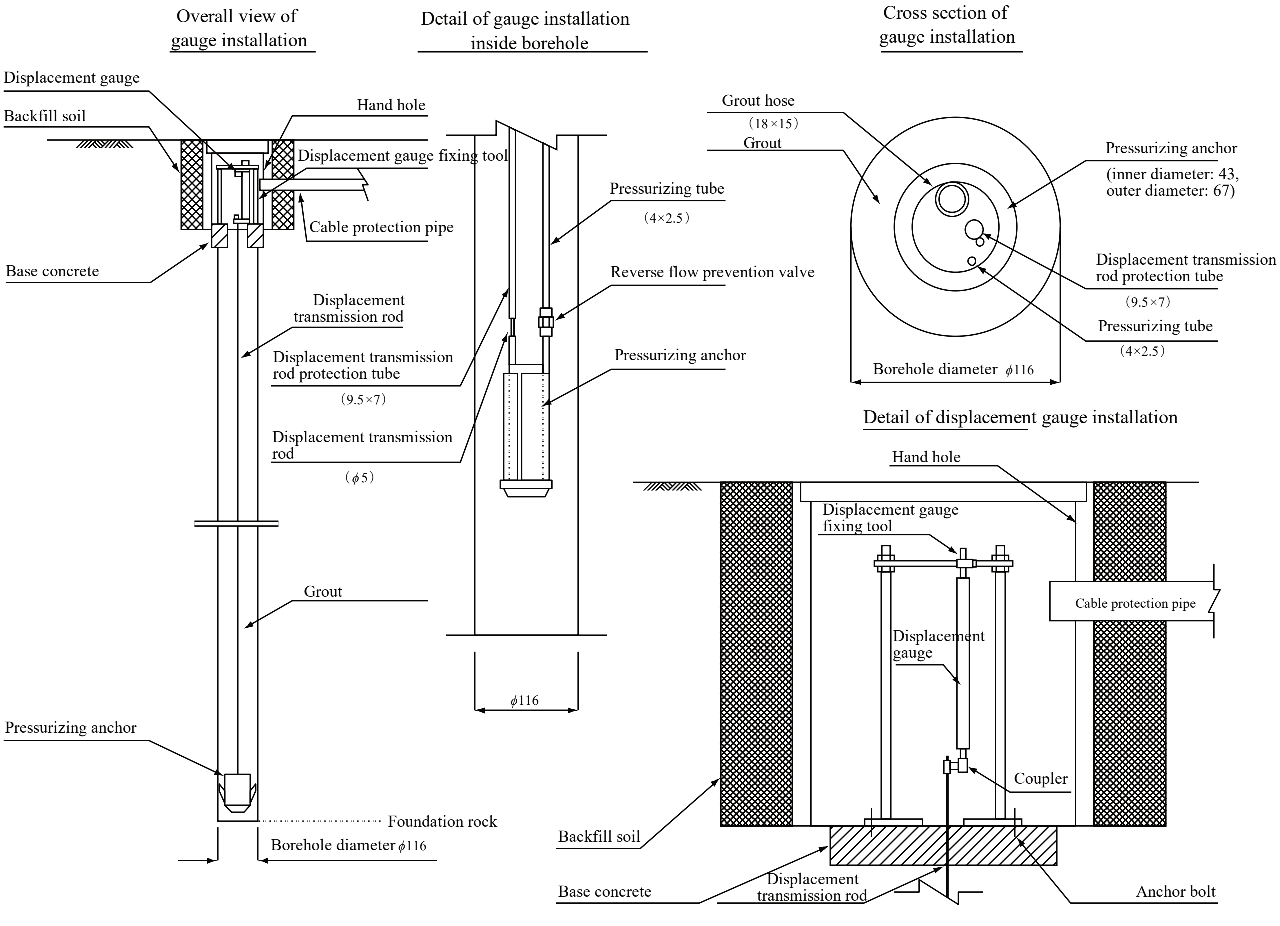


Fig. 3.11.7 Installation State of Ground Surface Settlement Gauge

2) Equipment for the field observation of existing structures

i. Water leveling type settlement gauge

Water leveling type settlement gauges made by Company C (FT-20C) were used to automatically acquire and organize data once a day. **Fig. 3.11.8** illustrates the installation state of a water leveling type settlement gauge.

ii. Installation type clinometer

Biaxial and uniaxial installation type clinometers made by Company C (BKK-A-1-D and BKK-A-1) were used to automatically acquire and organize data once a day. **Fig. 3.11.9** illustrates the installation state of an installation type clinometer.

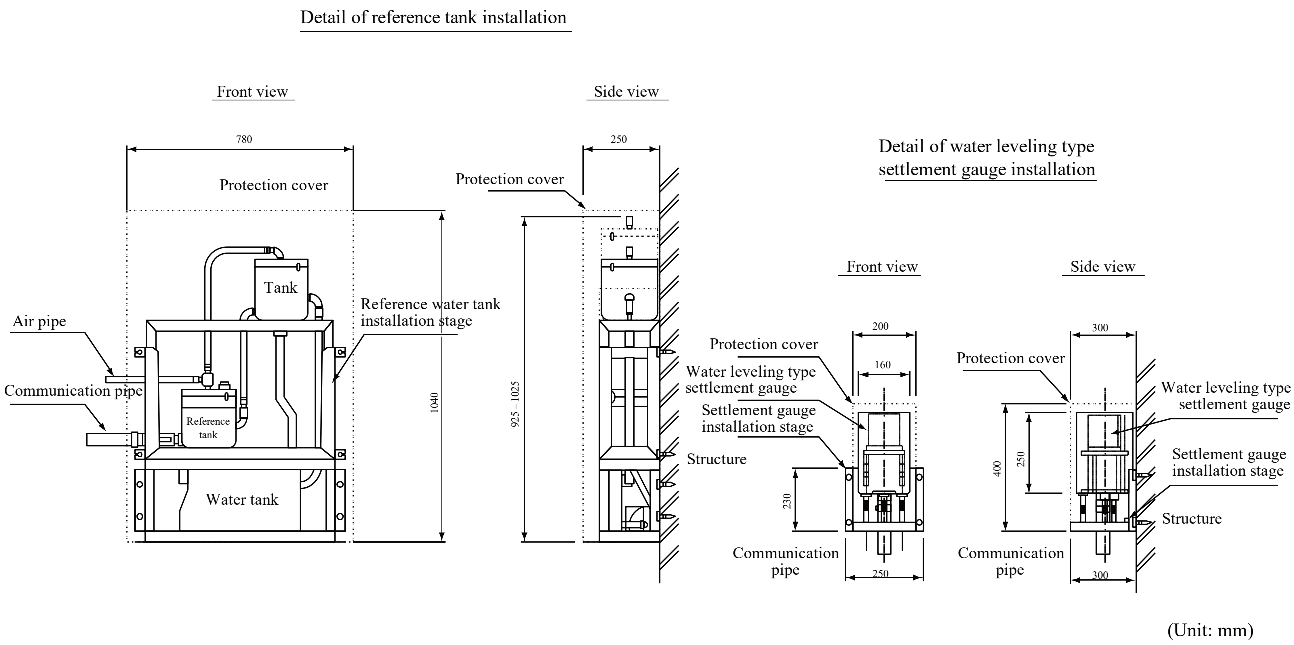


Fig. 3.11.8 Installation State of Water Leveling Type Settlement Gauge

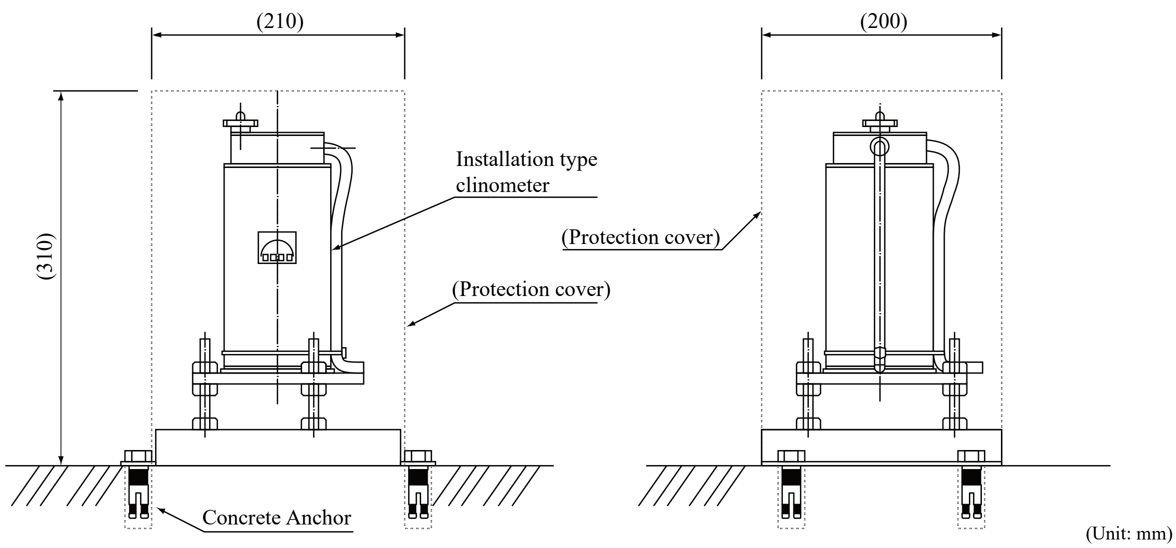


Fig. 3.11.9 Installation State of Installation Type Clinometer

(e) Issues and points of caution when conducting the field observation

The construction of the onshore tunnel section in the undersea road tunnel project was implemented with a thorough system for the supervision of the construction through field observations established for the purpose of ensuring not only the safe implementation of the tunnel and earth retaining wall construction, but also careful implementation to prevent the construction work from causing adverse effects on the precision machines in the factories neighboring the construction sites by setting management reference values. As a result, the construction of the onshore tunnel section was safely implemented with the adverse effects on the surrounding facilities minimized.

Through this experience of the construction of the onshore tunnel section, the following points of caution were identified for improving the performance of the field observation for the neighboring construction.

1) Preliminary identification of all precision machines in neighboring factories to prevent the construction from causing adverse effects on the business activities of these factories
2) Preliminary surveys of the conditions of all precision machines in neighboring factories in terms of their positions (distances from the construction sites) and the types of foundations
3) Preliminary surveys of the allowable displacement and current displacement of all precision machines in neighboring factories
4) Establishment of field observation plans based on the preliminary identification and survey results, and selection of the types and capacities of the observation equipment
5) Establishment of multiple countermeasures to cope with any displacement that may exceed the management reference values (both for the construction work and the precision machines in neighboring factories)

In any event, it is important to take proactive steps with a correct understanding of the issues associated with the neighboring construction.

(2) Field observation for a large-scale reclamation project on soft ground

① Outline of the large-scale reclamation project

This project was started in March 2007 under an agreement between the government and the contractor, a private joint venture company, for the design-built and maintenance for 30 years after construction. Following the completion of the reclamation in August 2010, the project has been in the service phase since October 2010. The reclamation work was completed in three and half years, which is significantly shorter than for other similar projects.

The major scope of the project was to develop a large-scale reclamation area on soft ground and the portion of the area over an estuary was a piled pier structure to allow river water to pass through. The reclamation area has a width of about 500 m and reclamation heights from A.P. +15.0 m up to A.P. +17.1 m (from a T.P of about +16.0 m to about +13.9 m), which is significantly higher than neighboring reclamation areas having reclamation heights of A.P. +4.0 m to A.P. +7.3 m (a T.P. of about +2.9 m to about +6.2 m).

Because the reclamation area was developed on a soft seabed, the project was implemented by improving the portion of seabed below the revetments through the low replacement rate sand compaction method (with 30% improvement) and the remaining portion of seabed below the reclamation area through the sand drain method.

In order to achieve in a short period of time the reclamation of high earth fills (about 35 m reclamation height from the seabed elevation of A.P. -15 m to A.P. -20 m to the highest reclamation elevation of A.P. +17.1 m) on seabed with a thick accumulation of soft alluvial clay, the challenges faced by the project were the acceleration of consolidation settlement and the stabilization of the improved ground (with a reclamation load of up to 600 kN/m^2). Furthermore, because of a necessity to restrict the lateral flow of the soft ground at the junction between the reclamation and piled pier sections, a steel pipe sheet pile cellular structure (having a water depth of 18 m and a revetment crown height of 14 m above seawater) was adopted as the revetment (earth pressure resisting structure). The revetment structure at the reclamation section is shown in **Figs. 3.11.10** and the structure at the junction between the reclamation and piled pier sections is shown in **Figs. 3.11.11**. Because the reclaimed area needs to be placed in service while allowing the area to have settlement and displacement, it was necessary to prevent the functions of the facilities on the reclamation area from being affected by the displacement. Thus, the project also faced challenges when attempting to accurately predict the amount of displacement and its effects on the structures, and establish a future maintenance plan through the field observation during the construction. In order to cope with these challenges, the project was implemented through the following field observation and construction using information technology.

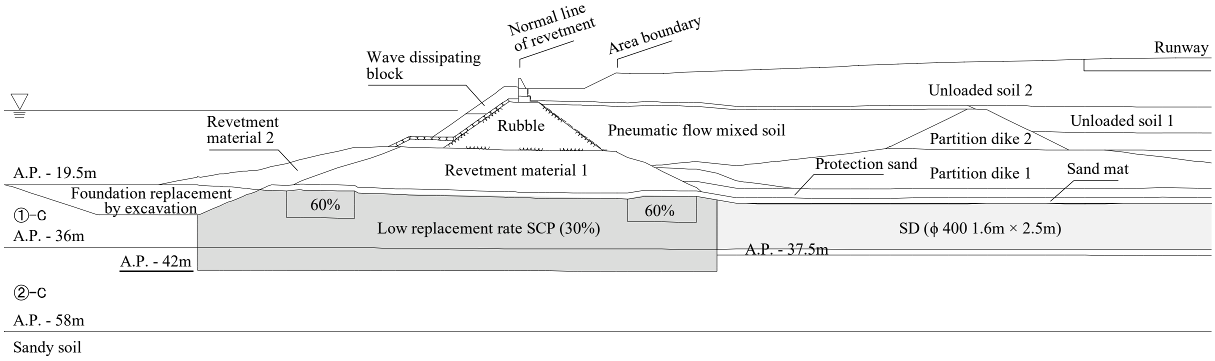


Fig. 3.11.10 Structure of Reclamation Revetment

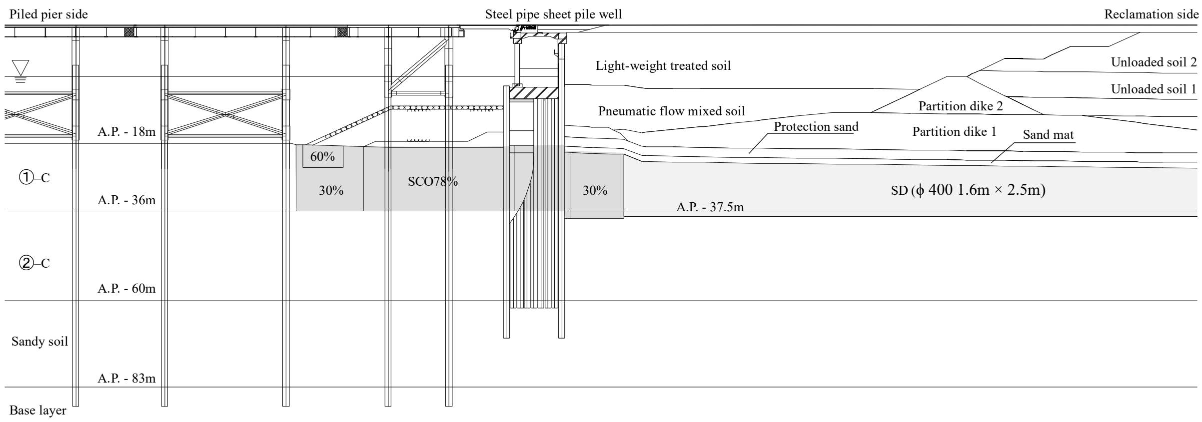


Fig. 3.11.11 Structure of Junction Section between the Reclamation and Piled Pier Sections

② **Ground structure**

In related construction implemented prior to this project, the ground of the project area had been classified into Layer A (alluvium), a soft layer accumulated to A.P. -35 m, and Layer D (diluvium), which was deeper than A.P. -35 m.[75)] In this project, however, the ground is classified into Type ① to Type ⑤ layer groups[76)] through a combination of geology and soil engineering. The list of the soil properties of the layer group classifications and the stratum structure of the ground around the center of the reclamation area are shown in **Table 3.11.10** and **Fig. 3.11.12**, respectively.

The Type ①-C-1 and Type ①-C-2 layers are geologically new layers located in a depth range from A.P. -18 m to A.P. -36 m in **Fig. 3.11.11**. These layers collectively have a thickness of about 15 m and are characterized as soft layers with large consolidation settlement and lateral flow. The Type ①-H layer partially covers the Type ①-C-1 and Type ①-C-2 layers in the project area close to the existing reclaimed area. The Type ①-H layer is a cover layer of the existing reclaimed area and is characterized as an inhomogeneous sandy soil layer.

The Type ② layer group is located in a depth range from A.P. -35 m to A.P. -60 m and is characterized as a diluvial and an inhomogeneous cohesive soil layer with intervening thin lenticular sand layers. The Type ① and Type ② layer groups are subjected to consolidation settlement. Because the Type ② layer group is characterized as a layer of cohesive soil mixed with sand which does not show a clear consolidation yield point, it was very difficult to predict the consolidation settlement of this layer group.

The layer groups of the Type ③ and deeper are characterized as cohesive soil, sand and gravel layer groups having *SPT-N values* larger than 30, and are, therefore, considered to cause no large deformations from the engineering judgment. Thus, the Type ① and Type ② layer groups are the main subjects of the field observation in regard to ground deformation.

Table 3.11.10 List of Soil Property Classifications

Large classification	Small classification		Soil type	Property
①	① -H	Cover soil	Mainly cohesive soil with intervening sand	* Superficial cover layer mainly of cohesive soil with intervening sand
	① -C- 1 to ① -C- 2	Ac 1 (A.P. - 20 to 30 m)	Mainly cohesive soil	* High water content, plasticity index and void ratio (Wn > 100, Ip > 60, e > 3) * Classification according to consolidation characteristics with 5-m intervals * Unit weight γ_t of about 13.5 kN/m^3
	① -C- 3	Ac 1 (A.P. - 30 m to)		* Unit weight γ_t of about 15.0 kN/m^3
②	② -C- 1 to ② -C- 4	Ac 2 to Dc 1 layer	Mainly cohesive soil with intervening sand in the lower half of the Type ② layer	* Cohesive soil with coarser and lower plasticity than the Type ① layer * Classification according to consolidation characteristics with 5-m intervals * Unit weight γ_t of about 18.0 kN/m^3, which is relatively large for cohesive soil
	② -S	Sandy layer as a part of Ac 2 to Dc 1 layer		* Thin, alternating layers of sandy soil located in the lower half of the Type ② layer
③	③ -C- 1 to ③ -C- 3 ③ -C- 2 low C_C	Cohesive soil section of alternating layers	Alternating layers of cohesive and sandy soil as well as gravel	* Unit weight γ_t of about 18.0 kN/m^3, which is relatively large for cohesive soil * Type ③-C-1 layer positioned above the high C_C layer and Type ③-C-3 layer positioned below the high C_C layer * Low C_C layer and high Cc layer at same depth
	③ -S	Sandy soil section of alternating layers		* Continuous distribution of sandy soil with *SPT-N value* of 50 or more
	③ -C- 2 high C_C	High water content cohesive soil		* Unit weight γ_t of about 14.5 kN/m^3, which is relatively small for cohesive soil * Soil with a large concentration of diatom, high plasticity, partially high water content and high void ratio * Appearance at around A.P. -70 m
	③ -G	Sand gravel section of alternating layers		* Intervening buried terrace gravel (btg) layer in the Type ③ layer * Gravel diameter over 100 mm
④	④ -C	Layer that could function as an engineering foundation (cohesive soil section)	Mainly sandy and gravel soil with intervening cohesive soil	* Cohesive soil with a relatively low plasticity index Ip with a range from 10 to 40
	④ -S	Layer that could function as an engineering foundation (sandy soil section)		*Mainly sandy soil **SPT-N value* of 50 or more in most areas with some low *SPT-N value* areas
	④ -G	Layer that could function as an engineering foundation (gravel section)		*Gravel diameter of about 80 mm **SPT-N value* of 50 or more
⑤	⑤	Engineering foundation layer	Mainly sandy soil	*Mostly sandy soil layers with some thin cohesive soil layers *Continuous layers with *SPT-N value* of 50 or more

γ_t: Unit weight

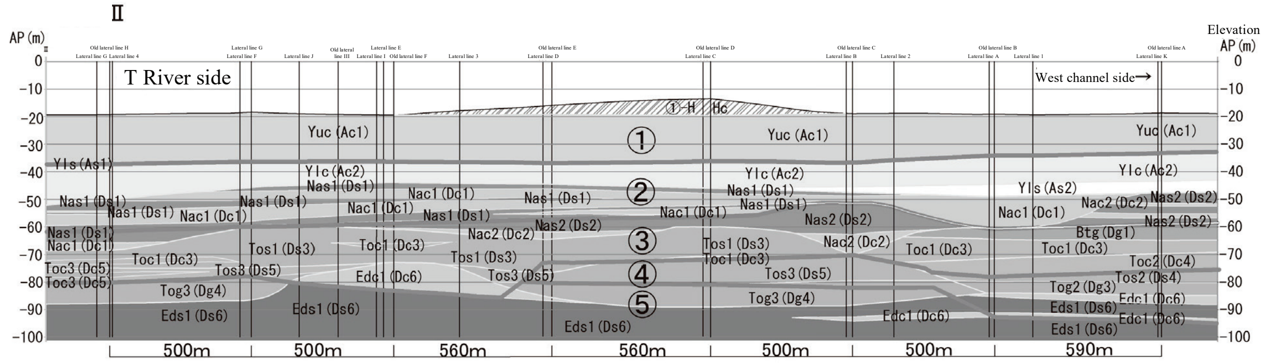


Fig. 3.11.12 Stratum Structure

③ Field observation items and equipment

Field observations and construction using information technology were implemented for the following purposes:

- Control of settlement and stability during construction;
- Setting of the final reclamation crown height based on the settlement predictions;
- Establishment of a maintenance program after the commencement of service; and
- Measurement of the lateral flow at the junction between the reclamation and pile pier sections.

As the consolidation settlement had not been completed when the reclamation area was put into service, it was predicted that the reclamation area would undergo an additional consolidation settlement of 0.7 to 1 m. Thus, it was necessary to set the constants of ground according to the data history on loading and settlement during reclamation and to decide extra filling for the crown level of the reclaimed land while taking into consideration future settlement. Furthermore, because the contract included the maintenance of the reclaimed area for 30 years after the commencement of service, field observations and construction using information technology have played an important role to acquire the data necessary for establishing the maintenance program. In addition, the lateral flow behind the junction between the reclamation and piled pier sections has been observed and managed.

The items which are subjected to the field observations and construction using information technology are:

1) Management of the settlement of the reclaimed area;
2) Management of the stability of the revetment;
3) Management of history record of the changes in reclamation layer thicknesses and predictions for the long-term settlement;
4) Management of the lateral flow of the junction between the reclamation and piled pier sections; and
5) Management of the compression of earth fill.

Table 3.11.11 shows a list of the field observation items and equipment. The layout of the field observation equipment and a typical equipment cross section arrangement are shown in **Figs. 3.11.13** and **3.11.14**, respectively. Intensive observation zones are installed at intervals of about 500 m throughout the reclamation section, which has a length of 2,020 m, with simplified observation zones to supplement the intensive zones at intervals of about 250 m. In addition, hydraulic type settlement gauges are installed at intervals of about 250 m in the central portion of the reclamation area. **Fig. 3.11.15** shows the procedure for selecting the observation zones. In addition, a ground survey using the radioisotope cone penetration test (hereinafter referred to as the "RI-CPT"[77]) has been conducted at each construction step to confirm the development of the ground strength to a level that allows the reclamation work to progress to the next step.

Table 3.11.11 List of Field Observation Items and Equipment

Field observation item	Description	Equipment
Ground settlement	• Observation and verification of the progress of consolidation during construction • Setting of the earth fill crown heights according to the amount of settlement • Setting of the consolidation constants of ground	C settlement plate[A)] CB settlement plate[B)] Hydraulic type settlement gauge Stratified settlement gauge Pore water pressure gauge
Revetment stability	• Confirmation of the increases in ground strength and judgement of the suitability of construction with additional earth fill reclamation through stability analysis • Examination of revetment stability using a stability management chart • Continuous observation of the behavior of earth fill ground using real time GPS	RI-CPT Clinometer Observation of displacement using GPS
History record changes in reclamation layer thickness and long-term settlement predictions	• Management of reclamation layer thicknesses • Management of the soil unloading procedure and observation of slip failure at soil unloading locations • Utilization of the observation data for predictions of long-term settlement and analysis of the increases in ground strength	GPS Bathymetric survey HASP Earth pressure gauge
Lateral flow at the junction between the reclamation and piled pier sections	• Observation and analysis of the displacement of the steel pipe sheet pile cellular • Management of the displacement of the ground behind the junction	CB settlement plate Hydraulic type settlement gauge Clinometer Pore water pressure gauge Stratified settlement gauge
Compression of earth fill	• Observation of the compression of earth fill layers during and after the completion of earth fill	Cross-arm type settlement gauge

Note) A): Temporary settlement plate (refer to Item ④ (a))
B): Settlement plate which doubles as guide pipes for check boring (refer to Item ④ (c))

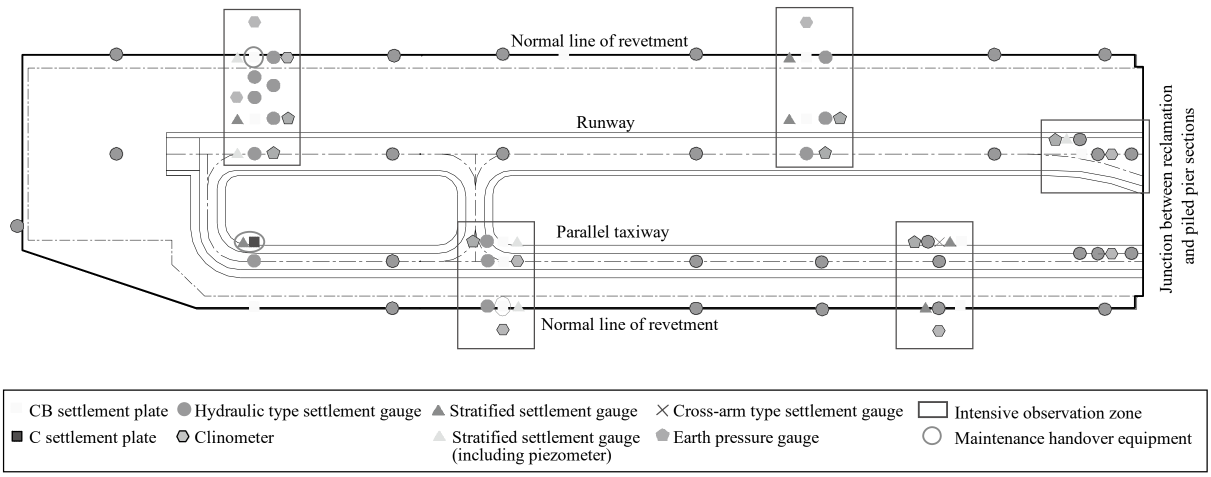


Fig. 3.11.13 Layout of Field Observation Equipment

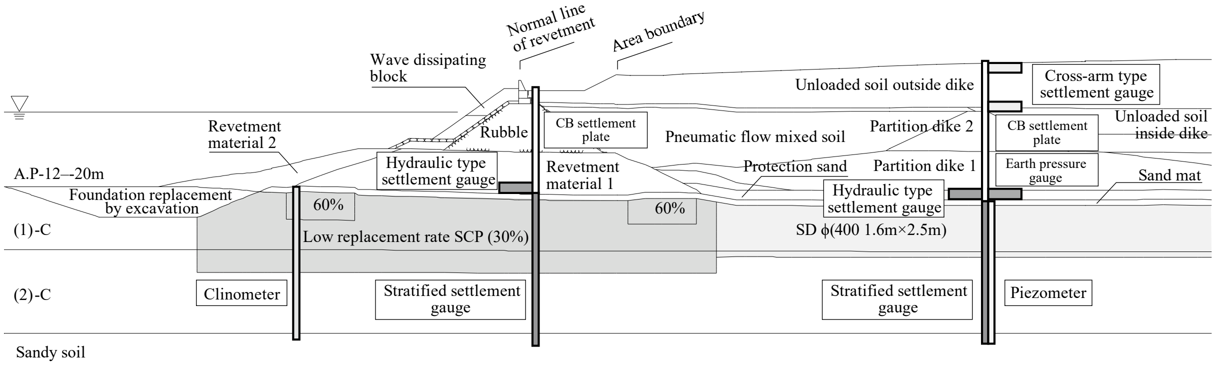


Fig. 3.11.14 Cross Section of Equipment Arrangement

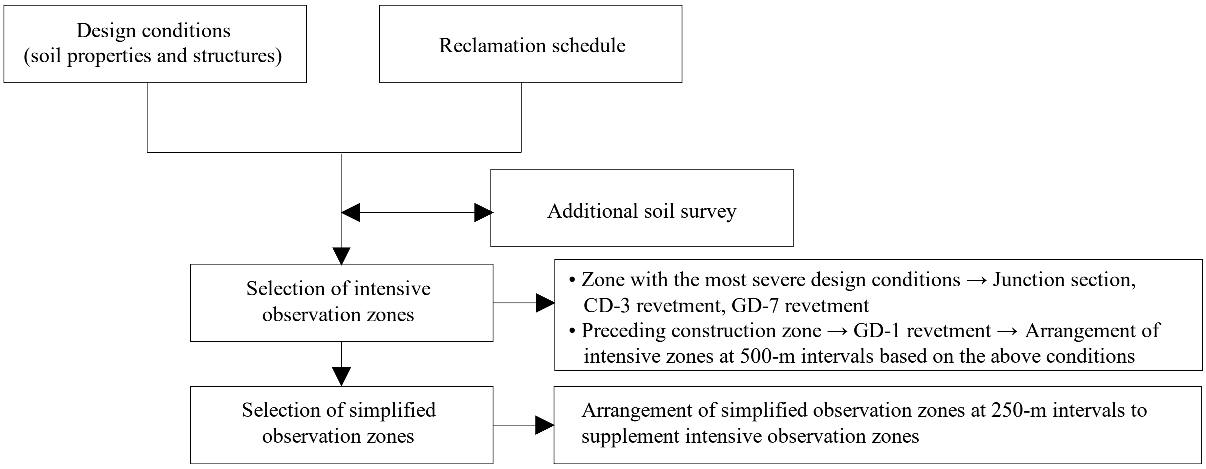


Fig. 3.11.15 Procedure to Select Observation Zones

④ **Management of ground settlement**

A maximum consolidation settlement of 8 m was estimated for the period from the commencement of reclamation work to the 100th year of service. In addition, a settlement of 0.7 to 1 m was estimated for 100 years after the commencement of service (compared to the re-estimated result of 0.5 to 0.7 m upon completion of the reclamation work). Conventional CB settlement plates (refer to **Fig. 3.11.16**), which have favorable performance records, were used for observation of the settlement. However, it was expected that installing many CB settlement plates over a distance of about 500 m between the left and right revetments might interfere with operation of the work vessels. Thus, hydraulic type settlement gauges connected to a magnetic transmission system[78)] were strictly arranged in the construction area with the CB settlement plates used complimentarily for calibrating the hydraulic type settlement gauges. The installation locations of the CB settlement plates were selected in the areas planned to be reclaimed ahead of other areas. Furthermore, because the Type ① and Type ② layer groups were expected to undergo consolidation settlement, stratified settlement gauges and pore water pressure gauges were installed to observe the degree of consolidation of each layer group.

(a) C settlement plates

Although consolidation settlement starts immediately after the soil improvement work, it was difficult to install CB settlement plates and hydraulic type settlement gauges to observe the settlement because offshore soil improvement work vessels were in operation. Thus, C settlement plates (as shown in **Fig. 3.11.17**) were installed immediately after soil improvement using the sand drain method to observe the settlement in the initial stage. After the installation of the C settlement plates, the diver measured the water depth of the C settlement plate with portable hydraulic gauges, corrected from the tide level to the elevations of the C settlement plates. This observation method was repeated until the proper measuring equipment was available to be installed.

(b) Magnetic transmission system and hydraulic type settlement gauges

The magnetic transmission system transmits data using magnetic waves with long wavelengths. By taking advantage of the ability of magnetic waves that enable data to be transmitted even through media with high electric conductivity, such as ground and seawater, despite the fact that their transmittable ranges may be as short as about 100 m,[78] the magnetic transmission system enables data to be acquired wirelessly from equipment underground. Therefore, in addition to settlement gauges, clinometers, pore water pressure gauges and stratified settlement gauges have also been connected to the magnetic transmission system to acquire data. **Fig. 3.11.18** shows schematically the concept of data acquisition through the magnetic transmission system. Although the figure only shows data acquisition during underwater reclamation, once the reclamation ground reached above sea level, survey vehicles were used to approach the transmittable range and collect data. **Fig. 3.11.19** shows the external view of a hydraulic type settlement gauge. The data from the hydraulic type settlement gauge was converted to elevations while taking into consideration the tidal levels when the data was acquired.

(c) CB settlement plates

CB settlement plates have a structure consisting of multiple steel pipes that have a diameter of ϕ 800, which are used as guide pipes for check boring. **Fig. 3.11.16** shows an external view of a CB settlement plate. Steel pipes were extended to match the settlement as settlement progressed.

(d) Stratified settlement gauges

Stratified settlement gauges are anchor rod type settlement gauges inserted into boreholes and transmit data through the magnet transmission system. Some types of clinometers and stratified settlement gauges allow data to be manually collected with probes inserted into them. However, these types of equipment were not used in the field observation this time because there is a risk that the displacement of ground may cause the lower end of the probe insertion pipes to be damaged and the measuring guide pipes to be displaced. As shown in **Fig. 3.11.20**, the stratified settlement gauges were installed at eight locations: midway between the Type ①-C-1 and Type ①-C-2 layers, at the boundary between the Type ① and Type ② layer groups, at the boundary between the cohesive and sandy soil layers in the Type ② layer group, at the boundary between the Type ② and Type ③ layer groups, at the boundary between the cohesive and sandy soil layers in the Type ③ layer group, midway between the Type ③-C-1 and Type ③-C-2 layers, midway between the Type ③-C-2 and Type ③-C-1 layers, and at the boundary between the Type ③ and Type ④ layer groups. The point on the boundary between the Type ③ and Type ④ layer groups was used as a fixed point.

(e) Pore water pressure gauge

Push-fit pore water pressure gauges were inserted into the boreholes so as to be positioned at predetermined depths in the reclamation ground. The installation depths of the pore water pressure gauges are shown in **Fig. 3.11.21**. Because each pore water pressure gauge requires a dedicated borehole, the number of boreholes was equal to that of the pore water pressure gauges. After assessing a proposal to install multiple pore water pressure gauges in one borehole, the plan did not materialize because of the difficulty in making the boreholes waterproof; therefore, one borehole was prepared for each pore water pressure gauge.

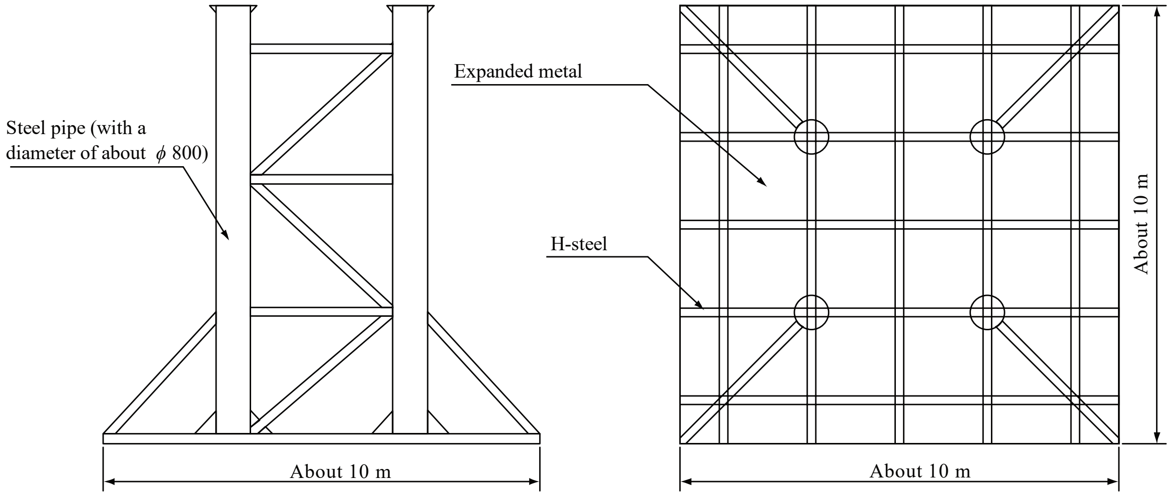


Fig. 3.11.16 CB Settlement Plate

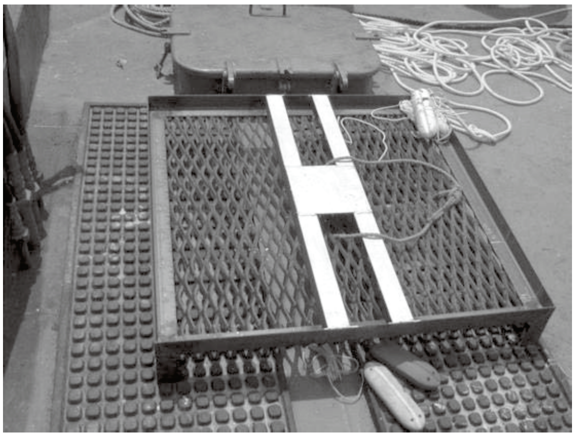


Fig. 3.11.17 C Settlement Plate

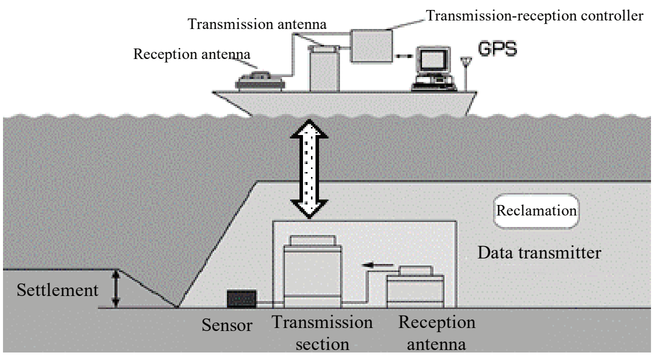


Fig. 3.11.18 Data Collection through the Magnetic Transmission System (For Underwater Reclamation)

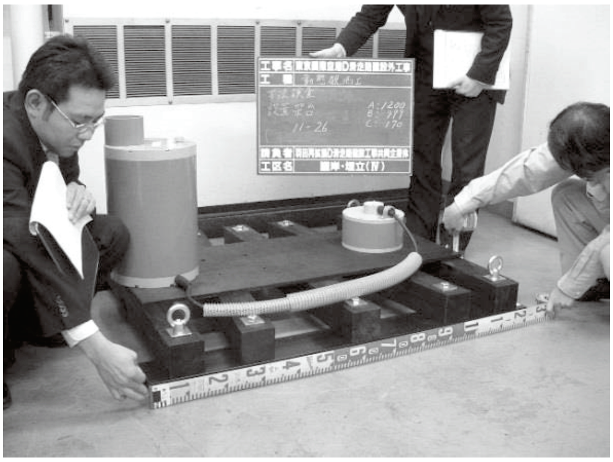


Fig. 3.11.19 Hydraulic Type Settlement Gauge (Magnetic Transmission System)

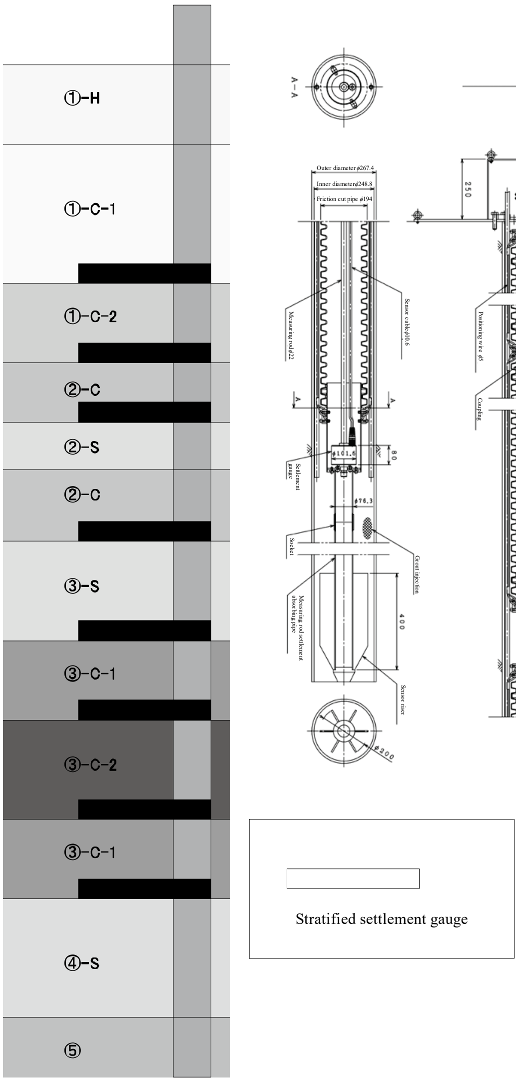


Fig. 3.11.20 Stratified Settlement Gauge

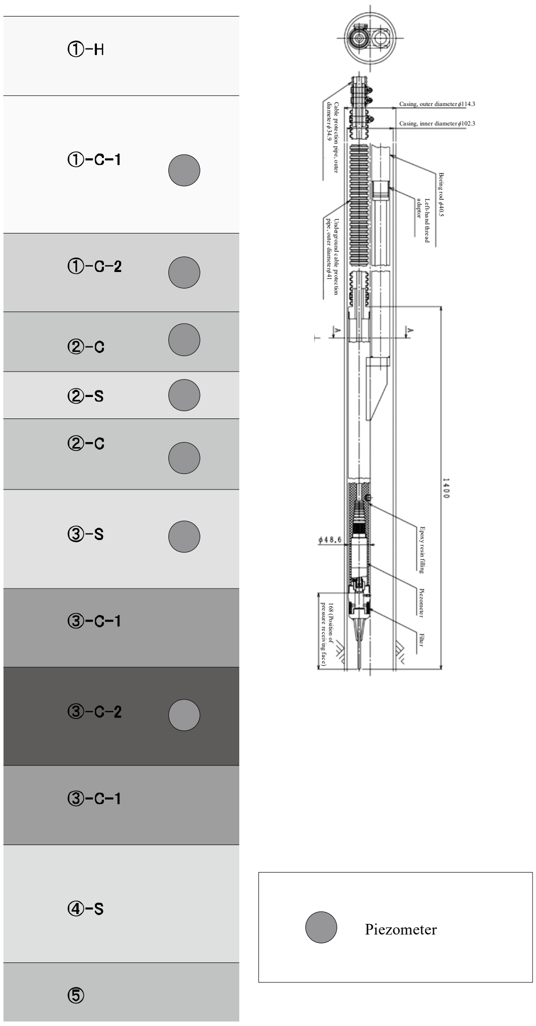


Fig. 3.11.21 Pore water pressure gauge

⑤ **Management of the stability of the revetment**

During the reclamation project, due to constraints on the construction schedule, the revetment and partition dikes were designed by taking into consideration the strength increases in the cohesive soil during the minimum consolidation period (two months after construction with a consolidation rate U of 50%). For reference, in the case of Kansai International Airport, the consolidation period was four months and the consolidation rate U was 80%. The safety factor Fs (corresponding to adjustment factor m in these Standards and Commentaries) was set at Fs ≥ 1.2 for a circular slip surface passing through composite ground, or Fs ≥ 1.3 for a circular slip surface mostly passing through the cohesive soil layer when the reclamation area was completed, and 1.1 ≤ Fs ≤ 1.3 as a safety factor during the construction on the condition that the ground strength was to be confirmed in situ using a "method to estimate the undrained strength through the RI-CPT" for each reclamation steps, thereby reducing the sizes of the counterweights to stabilize the revetment and shortening the construction period. The management of the stability of the revetment was implemented through observation of the inclination using clinometers installed on the revetment during construction and horizontal displacement of the levee crowns of the revetment using GPS after the completion of the revetment. **Fig. 3.11.22** shows a procedure to manage the stability of the revetment.

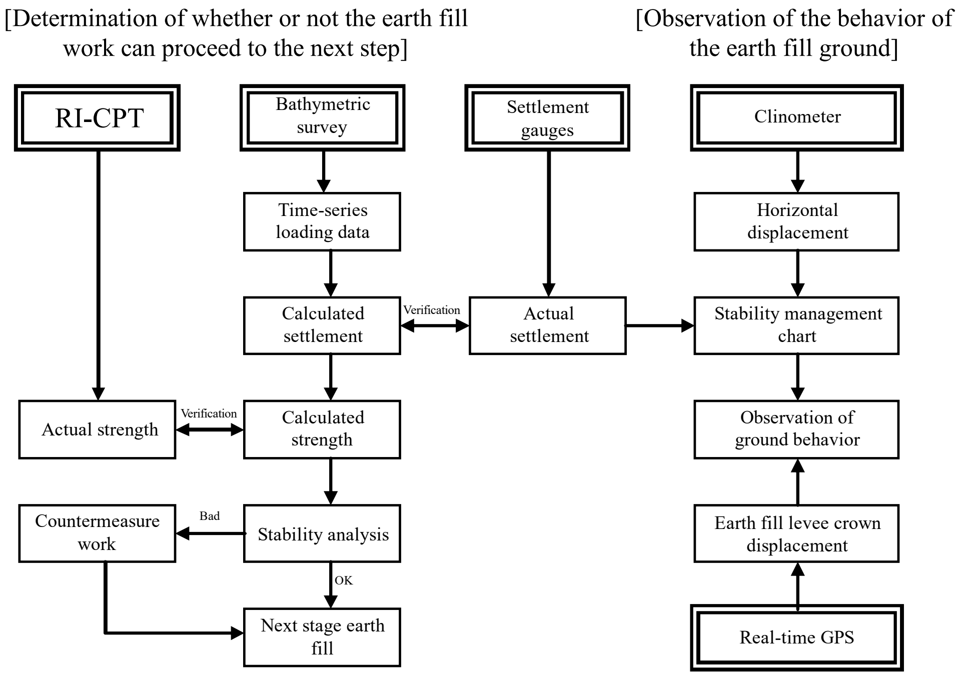


Fig 3.11.22 Procedure to Manage the Stability of the Revetment

(a) Estimation of undrained strength through the RI-CPT

Similar to the electrical static cone penetration test, the RI-CPT is a soil investigation method with a cone-shaped probe mounted with multiple sensors (as shown in **Fig. 3.11.23**) inserted into the ground at slow velocity. In addition to its capability to measure tip resistance and pore water pressure, as with the electrical static cone penetration test, the RI-CPT can measure the wet density of the ground using a radioisotope mounted at the cone tips. For saturated ground, the RI-CPT enables the void ratios to be calculated from the preliminarily obtained soil particle density and wet density.

In the reclamation project, the correlation between the measurement results by the RI-CPT and the constant volume box shear test results was obtained during the preliminary ground survey and used for determining the soil layers and setting the constants of ground. During the reclamation work, the RI-CPT was also used as an investigation method[79), 80)] to obtain the shear strength of the reclaimed ground through the tip resistance. A ground survey using the RI-CPT was conducted before implementing each type of earth fill work that required determining whether or not the reclamation work can proceed to the next step, as shown in **Fig. 3.11.22**. By including the information on the void ratios with the above, the management of the deformation of the ground was implemented by observing the progress of consolidation through a variety of information on the ground available through the RI-CPT.

(b) Management of the stability of the revetment using clinometers

The data obtained using clinometers was transmitted through the magnetic transmission system similarly with data transmission for hydraulic type settlement gauges and stratified settlement gauges. The clinometers were preliminarily installed in boreholes (as shown in **Fig. 3.11.24**) and automatically measured the displacement of the ground. During the construction of the revetment, the management of its stability was implemented by using various safety control charts (Tominaga method, Matsuo-Kawamura method and horizontal displacement velocity method) based on the settlement at the center of the revetment and the horizontal displacement at the slope toes. However, because these safety control charts are established for trapezoidal shaped cross sections earth fill, such as roads constructed on unimproved ground, these charts' accuracy and adaptability were not high for managing the stability of the revetment.

(c) Real-time observation of displacement using GPS

After the completion of the revetment, the scheduled fixed-point observation of the horizontal displacement and settlement of the revetment was implemented through a GPS real-time management system. The GPS real-time management system continuously monitors settlement and horizontal displacement using measurement data transmitted to a monitoring center through wireless LAN, and was used for continuous observation of the displacement behavior of the revetment superstructure while excavating the front sections of the revetment (as shown in **Fig. 3.11.25**). Because the arrangement of the equipment for the system could be continuously used even after the reclamation area reached above sea level, the GPS real-time management system was a management method suitable for managing the stability of the revetment with no loss in accuracy in the observation data.

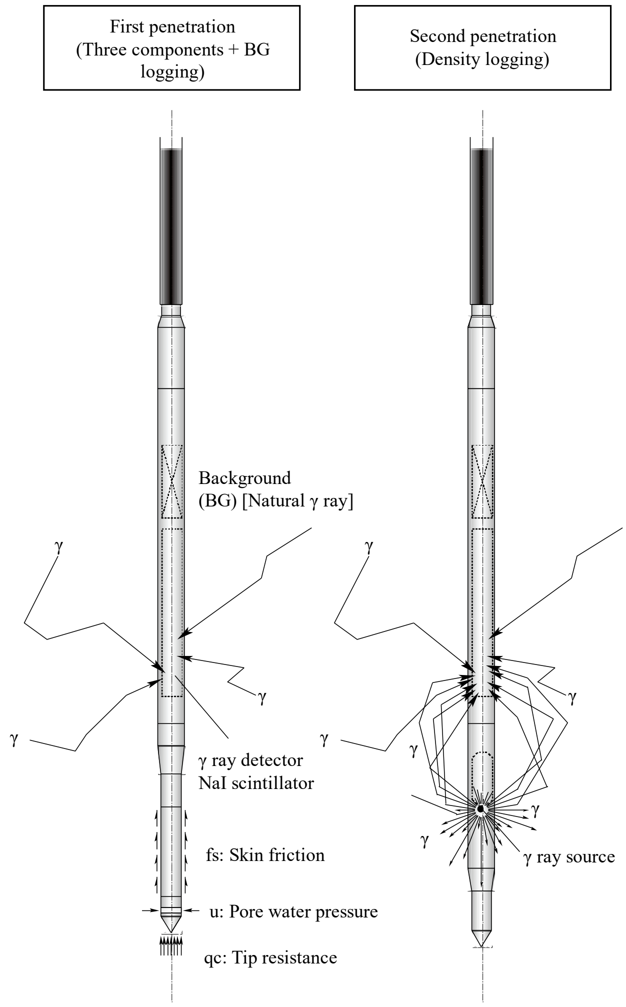


Fig. 3.11.23 RI-CPT Probe

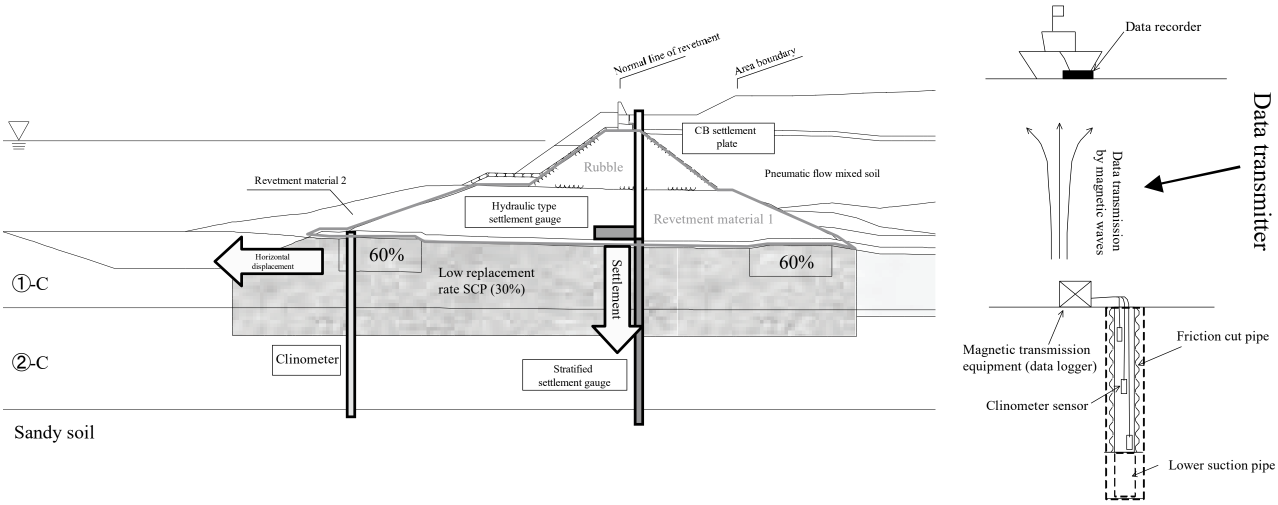


Fig. 3.11.24 Management of Stability through Clinometers and Settlement Gauges

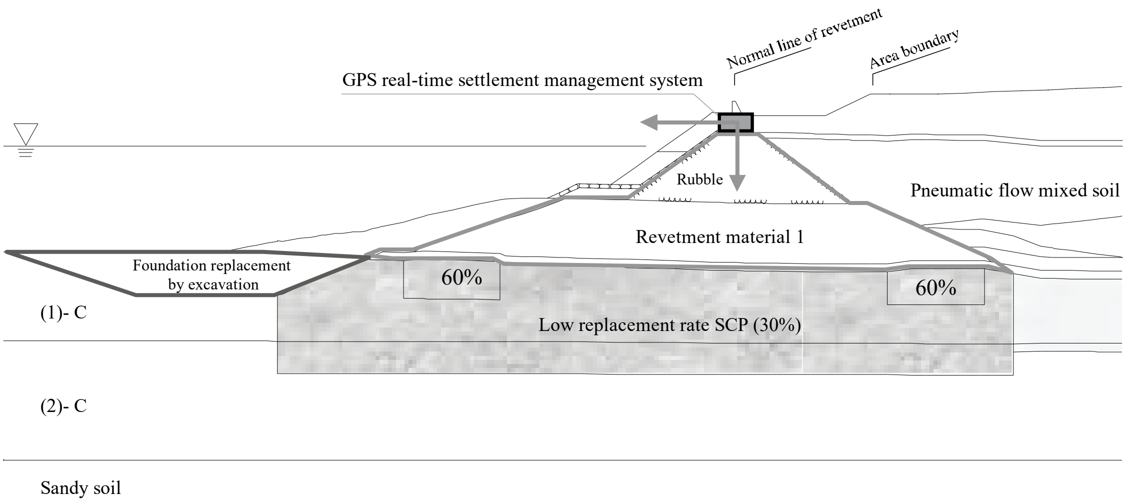


Fig. 3.11.25 Real-Time Management of Stability Using GPS

⑥ **Management of history record of the changes in reclamation layer thicknesses and prediction of the long-term settlement**

It is necessary to accurately acquire information on the history record of the changes in the reclamation layer thicknesses for the prediction of long-term settlement due to reclamation work and the management of the stability of the reclaimed ground. In the reclamation project, the reclamation height of the runway was determined by using a settlement calculation program based on information on the history record of the reclamation ground management obtained through a GPS survey.

(a) GPS survey

In the reclamation project, the management of the history record of the changes in reclamation layer thicknesses was implemented through a bathymetric survey combined with a GPS survey during the underwater reclamation and a GPS survey with buggy-type survey vehicles for the reclamation area above the sea level. In the bathymetric survey combined with the GPS survey (refer to **Fig. 3.11.26**), a catamaran-type survey boat, which is less susceptible to wave motions, and mounted with a narrow multi-beam bathymetric survey system, was used. In addition, a RTK (real-time kinematic) GPS system was combined with oscillation correction divice on board the survey boat to improve the survey accuracy. In the GPS survey for the reclamation area above the sea level, planar observation of the ground elevations was implemented by running survey vehicles mounted with the RTK GPS system throughout the reclaimed area every day, with the widths of the survey vehicles serving as the survey line intervals. **Fig. 3.11.27** shows an example of the bathymetric survey results. Superimposing daily survey results made it possible to conduct a planar observation of the settlement of the reclaimed ground, in addition to the acquisition of history record of the changes in the reclamation layer thicknesses.

(b) Prediction of long-term settlement

Because the reclaimed area was planned to be put into service with a residual consolidation settlement of about 0.7 to 1.0 m left uncompleted, it was necessary to accurately predict the long-term settlement and determine an appropriate sinking allowance based on the predicted long-term settlement. Thus, the future consolidation settlement that the reclamation area will undergo was determined by developing a consolidation settlement prediction and management program, called the HASP (Haneda Airport Settlement Prediction Program)[81), 82)], which can predict the consolidation settlement by taking into consideration the influences of the history record of the planar transition of the reclamation work, load distribution and buoyancy. The HASP is software that simulates the ground behavior based on a three-dimensional settlement analysis, divides the ground into mesh elements and calculates the primary consolidation settlement of each mesh element by taking into consideration the three-dimensional load distribution. **Fig. 3.11.28** visualizes the history record of the changes in the loading conditions which were input into the HASP.

(c) Setting of the final crown height of the reclamation area

Using the following equations, the final crown height of the reclamation area was determined by setting the design height at the time of the commencement of service with the residual consolidation settlement during the service life (100 years) added to the basic design height as the sinking allowance. Furthermore, the consolidation settlement (predicted by the HASP) during the period from the completion of the earth fill until the commencement of service is considered as additional sinking allowance (refer to **Fig. 3.11.29**).

Final crown height = Design height at the time of commencement of service + Consolidation settlement from the completion of the earth fill until the commencement of service

Design height at the time of commencement of service
= Basic design height + Residual consolidation settlement during the service life (100 years)

However, because the sinking allowance could not be applied to the reclamation of the narrow segment from the junction between the reclamation and piled pier sections to the slope change point, the narrow segment was decided to be maintained as a transition segment (refer to **Fig. 3.11.29**) where possible uneven surfaces or changes in the slopes due to consolidation settlement will be repaired by cutting or overlaying pavement.

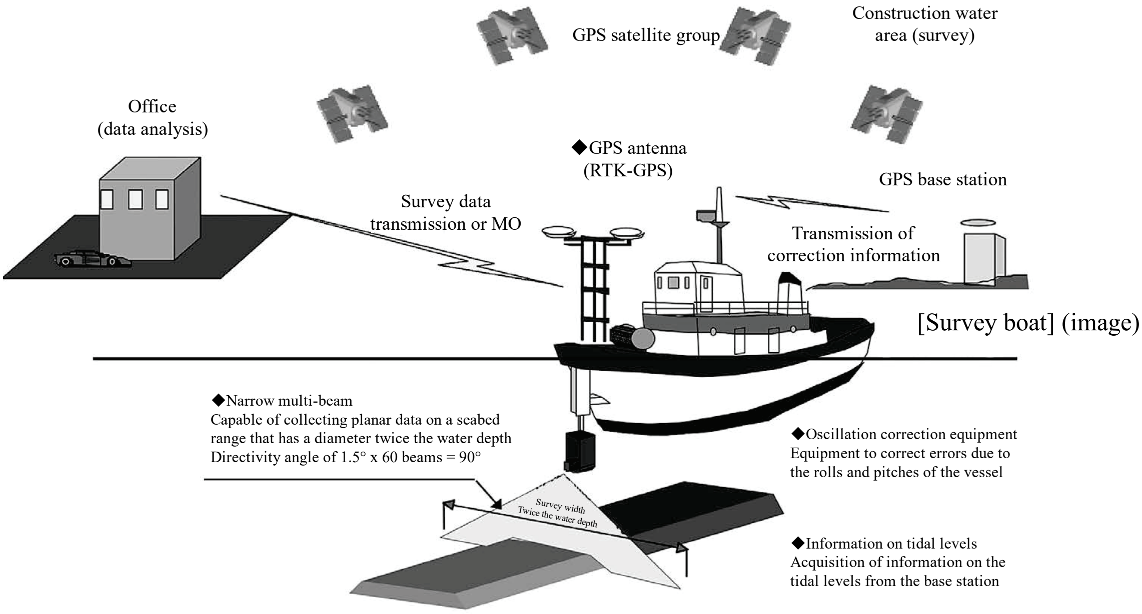


Fig. 3.11.26 Bathymetric Survey Combined with GPS Survey

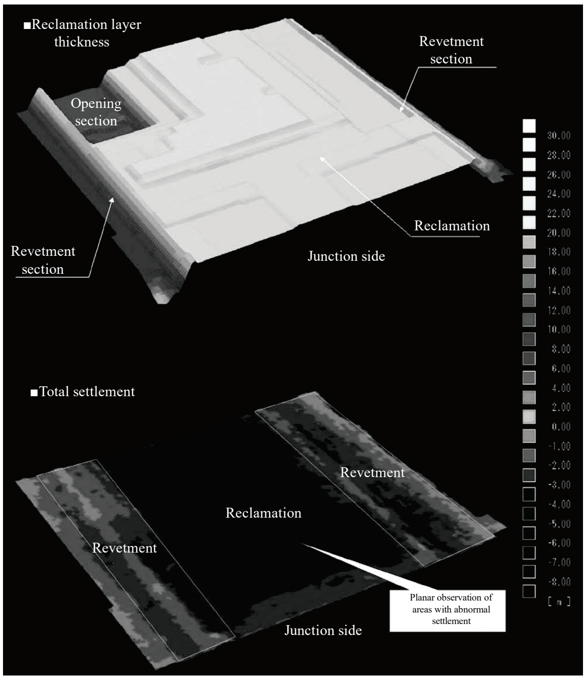


Fig. 3.11.27 Superposition of the Bathymetric Survey Results

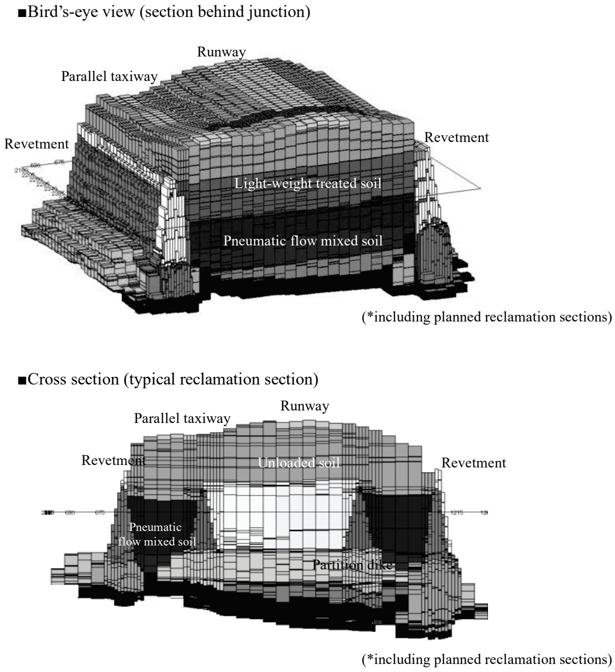


Fig. 3.11.28 Visualization of history record of the Changes in the Reclamation Work

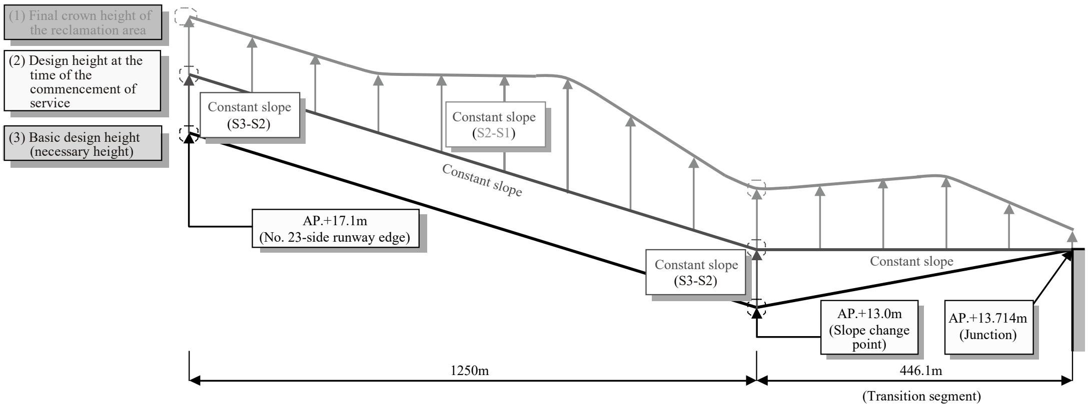


Fig. 3.11.29 Sinking Allowance for the Reclamation Area

⑦ **Management of the lateral flow at the junction between the reclamation and piled pier sections**

It was estimated that the steel pipe sheet pile cellular revetment at the junction between the reclamation and piled pier sections undergoes displacement in a direction toward the piled pier due to large lateral earth pressure from high earth fill caused by a difference in elevation of more than 30 m between the elevation of the seabed of A.P. -18 m (a T.P. of about -19.1 m) at the piled pier side, and an elevation of the reclaimed area of A.P. +13.7 m (a T.P. of about +12.6 m) at the center of the reclamation side. Thus, a field observation of the steel pipe sheet pile cellular and the reclaimed area behind the revetment was implemented to utilize the measured data in the prediction analysis of the displacement of the revetment for determining the positions of the access girders and expansion equipment connecting the reclamation and piled pier sections. In addition, the steel pipe sheet piles at the junction and the steel piles on the neighboring piled pier are also subjected to lateral displacement of the reclaimed ground, these structures were constructed while confirming their structural safety using clinometers and strain indicators.

(a) Field observation of the steel pipe sheet pile cellular revetment

As shown in **Fig. 3.11.30**, clinometers at 2.0-m intervals and strain indicators at 4.0-to-6.0-m intervals were installed on the steel pipe sheet piles close to the center of the reclamation area. As the steel pipe sheet piles are embedded in the bearing layer (Type ③-S layer), the toe level of the steel pipe sheet piles are considered as fixed points and the piles' displacements were evaluated using the inclination angles obtained through the clinometers. In addition, the section force (axial force and bending moment) on the steel piles was evaluated based on the stresses on them, which were calculated from strain data obtained through strain indicators installed on both the compression and tension sides of the steel piles.

(b) Field observation of the area behind the junction

In addition to the field observation of the steel pipe sheet pile cellular structure, the lateral displacement and settlement of the reclaimed area behind the structure were observed with underground clinometers, pore water pressure gauges, stratified settlement gauges and hydraulic type settlement gauges installed as shown in **Fig. 3.11.30**.

(c) Field observation of the steel piles on the piled pier near the junction

The structural safety of the steel pipe piles on the piled pier was evaluated by observing their displacement with clinometers installed at intervals of 2.0 m. The data from the clinometers were also used to determine when to execute the on-site connection of the main girders of the upper jacket of the piled pier.

(d) Prediction of the deformation of the steel pipe sheet pile cellular revetment

A two-dimensional elasto-viscoplastic FEM deformation analysis (constitutive equation based on modified Cam-Clay Sekiguchi and Ota model) was conducted with a modeling of the steel pipe sheet pile cellular revetment.[83)] Deformation analysis was performed while taking into consideration the actual reclamation processes for fitting the parameters of ground in a manner that enable the analysis results to be consistent with the actual measurements of the settlement behavior of the reclamation area behind the revetment and the deformation behavior of the steel pipe sheet piles. At the same time, the deformation analysis with the fitted parameters was also confirmed to be able to accurately reproduce the actual behavior of the steel pipe sheet pile cellular revetment. The deformation of the revetment over 100 years after the commencement of service was predicted through the deformation analysis, and the analysis results were incorporated into the construction management of the reclamation work.

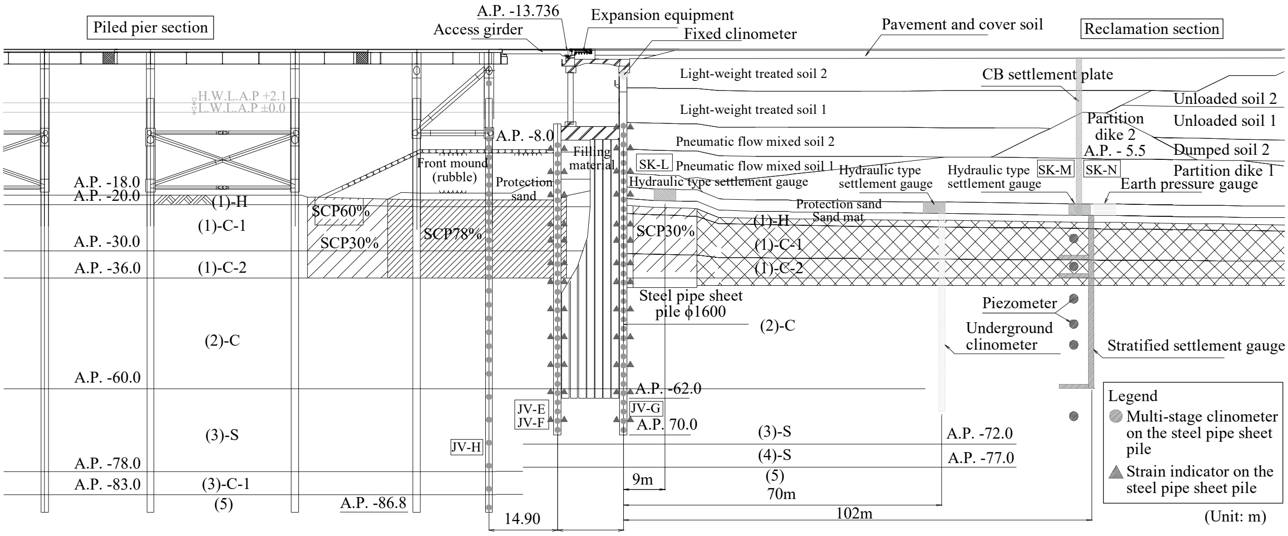


Fig. 3.11.30 Longitudinal Section of the Equipment Arrangement at the Junction between the Reclamation and Piled Pier Sections

⑧ Management of the compression of earth fill

Although the history record of the changes in the layer thicknesses and loading conditions of the reclamation and earth fill could be known through the bathymetric survey. However, in order to seperate between the settlement of surface layer of reclaimed soil and settlement of the original ground, the compression of reclaimed soil was additionally observed, while taking into consideration the immediate settlement and creep of the reclaimed soil. The method is that the compression of earth fill was measured with cross-arm type settlement gauges installed in the earth fill. Furthermore, hydraulic type settlement gauges were installed immediately beneath the cross-arm type settlement gauges to acquire data consistent with the settlement of the original ground.

Fig. 3.11.31 shows a schematic drawing of the arrangement of the gauges. The cross-arm type settlement gauges had vertical pipes made of vinyl chloride, which also served as observation wells to monitor the water levels in the reclamation area. As can be seen in **Fig. 3.11.32**, which shows the time-series change in the compression of the earth fill, the compression $\Delta\varepsilon$ of the earth fill after completion of the earth fill (on the 970th day) was 0.1 to 0.2%.

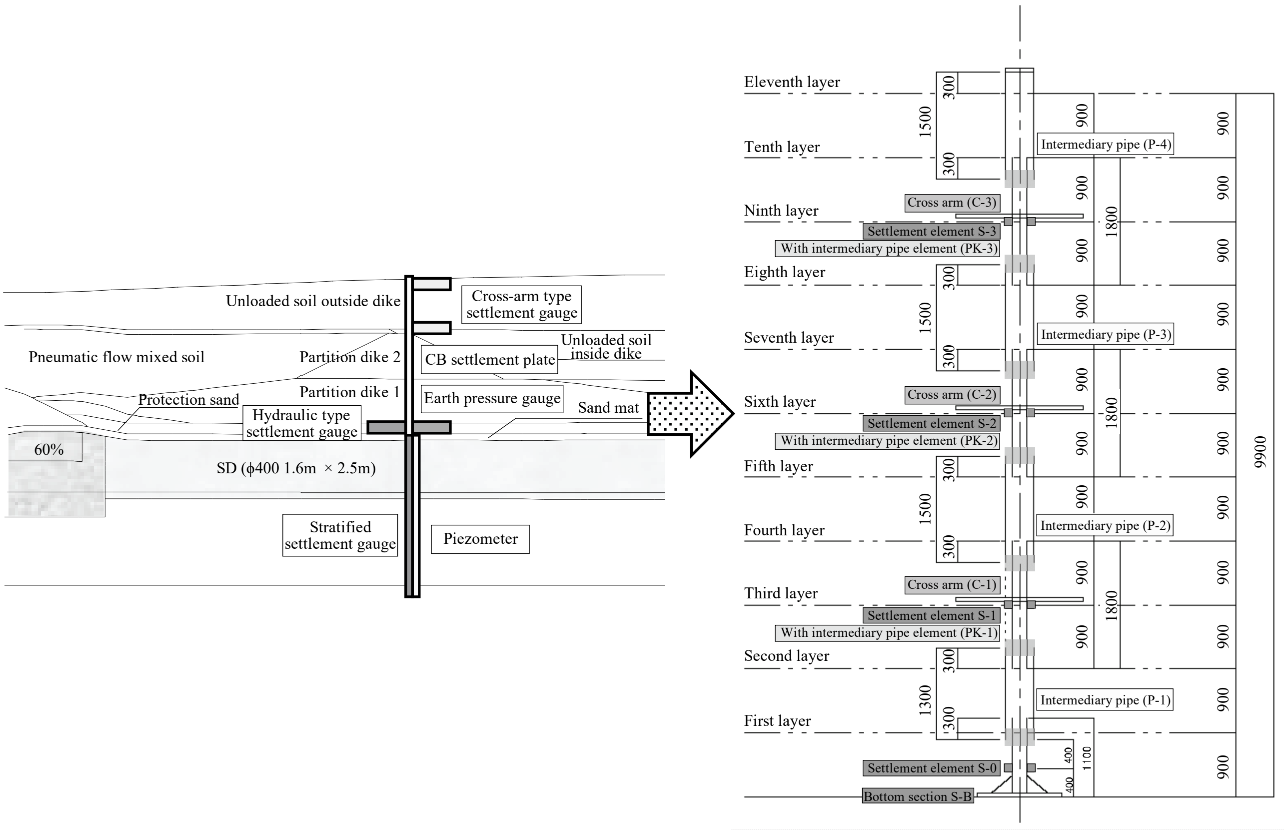


Fig. 3.11.31 Schematic Drawing of the Arrangement of the Cross-Arm Type Settlement Gauges

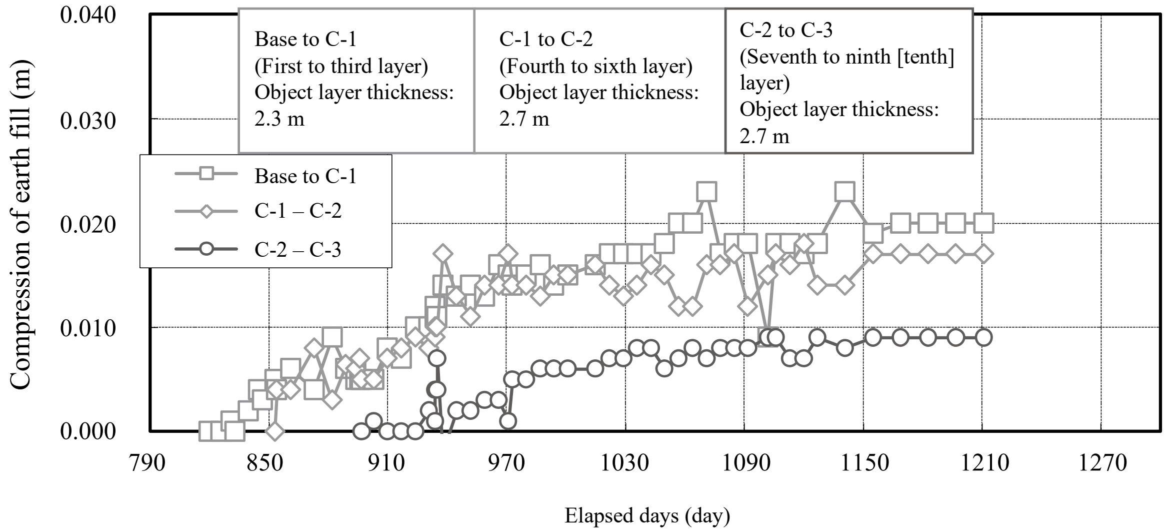


Fig. 3.11.32 Compression of Earth Fill (The base in this figure represents the upper edge of the original ground)

⑨ Challenges and points of caution when conducting a field observation in a large-scale reclamation project on soft ground

The challenges facing this large-scale reclamation project were related to the deformation of the ground due to earth fill on soft ground. In addition, the insufficient consolidation period, compared to normal reclamation projects, due to constraints in the construction period caused the execution of the reclamation work to be especially difficult. Thus, for the safe implementation of the cross-sectional design and construction of the revetment over the short project period, rather than implementing the survey, design, construction and observation independently, these items are implemented consistently based on feedback from the field observation and construction using information technology. Furthermore, the field observation was conducted for an additional important purpose of establishing a maintenance program to be used for 100 years after the commencement of service based on the field observation results. The following are challenges and points of caution for the field observations of each section.

(a) Field observation of the reclamation section

1) Management of the ground settlement

Two challenges which are faced in the management of the ground settlement are: one related to observation methods, and the other related to observation locations.

In regard to the observation methods, although it was considered that direct observation using equipment such as CB settlement plates was ideal for accurate observation of the settlement, installing a large number of CB settlement plates might interfere with the reclamation work. Instead many hydraulic type settlement gauges were used in this project to deal with the above problem.

In regard to the observation locations, the challenge that the project faced was the locations of the observation equipment to obtain representative values to represent the settlement of the reclamation area. As shown in **Fig. 3.11.12**, the ground conditions differ location by location due to the differences between the Type ① layer group, which has a relatively simple stratified structure, and the Type ② layer group, which has a complicated stratified structure with intervening sandy layers. In addition, the loading conditions also differ location by location due to slopes on the reclaimed ground surface. Therefore, there were intense discussions about whether or not the settlement observed at a limited number of locations could be applied for estimating the settlement of the other locations in the design stage. In this situation, observation equipment was installed at the locations with severe loading conditions and the locations where the reclamation was scheduled to be completed in earlier stages. It was also necessary to establish a construction plan which can be flexibly revised based on the field observation results.

2) Management of the revetment stability

The purpose of the management of the revetment stability is to prevent circular slip failure of the reclaimed ground. The foundation ground of the revetment section was improved through the low replacement rate SCP for economic efficiency; however, the prediction of ground behavior in the design stage relied mostly on numerical analyses because of the lack of performance records for the low replacement rate SCP implemented to prevent circular slip failures. Thus, one of the challenges of the project was the early detection of circular slip failure symptoms through the field observation; therefore, a large number of clinometers were used for the field observation.

The points of caution for the field observation include the selection of methods for managing stability. For this project, the strength of the ground was confirmed using two methods: in-situ tests and a field observation. For the in-situ tests, the RI-CPT was used, which enabled the in-situ strength to be confirmed continuously and was quite effective in identifying the locations of the soft layers in the Type ② layer group in the preliminarily boring survey, thereby contributing to the prevention of circular slip failures by changing the soil improvement depths through the sand compaction pile method. In contrast, stability management charts were used for managing stability through the field observation. Because these stability management charts are established for earth fill with trapezoidal shaped cross sections constructed on unimproved soil, the charts turned out to be not especially compatible for stability management of the revetment in terms of accuracy. In the future, better cost performance clinometers are expected to be developed and applying these clinometers in a large number of quantities would be the ideal countermeasurement against the problem mentioned above.

3) Management of history record of the changes in reclamation layer thicknesses and long-term settlement predictions

The challenge with the long-term settlement prediction was the accuracy. For this project, the future settlement of the reclamation area was calculated with the HASP, and an important point for using the HASP was the level of accuracy of the history record of the changes in the loading conditions and the constants of ground that would be the input conditions of the HASP. Acquiring history record of the changes in the loading conditions is very expensive, and attention is required when setting the representative values of the unit weight of soil because they vary widely. In actuality, it was relatively easy to determine the ground conditions of the Type ① layer group, which comprises uniform cohesive soil layers. In contrast, special attention was required when selecting the representative values of the Type ①-H layer and Type ② layer group from the observed layers because of their complicated stratified structures with intervening sand layers. In particular, due consideration was given to setting the consolidation yield stress (P_c) of the Type ② layer group because it could not be clearly identified from the observation results.

4) Management of the compression of earth fill

One of the challenges with the management of the compression of earth fill was the long-term stability associated with increased layer thicknesses. Because the earth fill on the surface was constructed with an increased layer thickness of 90 cm per layer,[84)] the field observation was conducted with the compression of the earth fill as one of the observation items from the commencement of construction to confirm long-term stability. Considering the lack of previous performance data on the evaluation of the long-term compression of earth fill,[85)] a creep test was preliminarily conducted to determine the target value of the compression, and the field observation results were compared with the target value. The field observation results for the compression at the time of completion of the reclamation work were smaller than the target value and the previous performance data.

(b) Field observation of the junction section

1) Installation of observation equipment

For the monitoring of steel pipe piles and steel pipe sheet piles, it is necessary to determine the installation locations of the observation equipment in consideration of the deformation modes of the steel pipes anticipated in the design stage. In the case of electric equipment, such as clinometers and strain indicators, because the values of the inclination angles and strain are available only at discrete locations where the equipment is installed, it is necessary to arrange the equipment densely to some extent when observing deformation modes in detail.

A point of caution when using this type of electric observation equipment is that, because steel piles and steel pipe sheet piles need to be constructed with the observation equipment already installed on the piles, the equipment has an increased risk of being damaged during the construction. For offshore construction, steel piles and steel pipe sheet piles are constructed through the hammer driving method in many cases. Because the risk of observation equipment becoming damaged during construction gets larger with the increases in the number of hammer blows, as with long piles and hard ground, it is necessary to increase the number of equipment installation locations, or preliminarily examine alternative observation methods in preparation for possible damage during construction.

In addition, because the working life of electric observation equipment is generally considered to be about ten years, this type of equipment is effective for field observations during construction, but is not suitable for long-term field observation. Thus, equipment using optical fibers is considered to be more efficient than electric equipment for long-term field observations.

2) Prediction of the lateral displacement of the revetment

The calculations of the consolidation settlement due to earth fill have often been conducted using constitutive equation based on modified Cam-Clay Sekiguchi and Ota model. Generally, this type of model is thought to be capable of accurately predicting ground displacement in the vertical direction as long as accurate constants of ground and the history records of the changes in the construction conditions are available. In contrast, there have been very few cases of predicting the lateral displacement of ground, and no constitutive equations applicable to such predictions have been fixed. Thus, for this large-scale reclamation project, Sekiguchi and Ota model (the modified Cam-Clay model) was used as the constitutive equation based on the verification of its applicability through centrifugal model tests and case studies.

A point of caution when conducting these deformation analyses is the necessity to model not only the ground, but also the structures, because the structures themselves such as steel pipe sheet piles undergo deformation. In the case of the steel pipe sheet pile cellular structure, it is necessary to pay attention to the nonlinearity of the joints of the steel pipe sheet piles[86)] and the selection of the joint elements between the ground and structures. For the prediction of the deformation of the revetment structures due to the lateral displacement of the ground, because it is difficult to predict such deformation with sufficient accuracy only through preliminary analyses, it is necessary to conduct deformation analyses while fitting analysis parameters, such as constants of ground, through field observations during construction.

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4 Observations for Setting Up Ground Motions for Performance Verification

4.1 General

In general, earthquake ground motions are affected by three factors, namely, the source, path and site effects (**Fig. 4.1.1**). The source effects can be defined as the characteristics of seismic waves generated at the earthquake source as a result of a rupture process on the fault. The path effects can be defined as the attenuation and deformation of seismic waves during their propagation from the source to the upper boundary of the seismological bedrock below the site. The site effects can be defined as the influence of sediments above the seismological bedrock on the seismic waves. The seismological bedrock can be defined as the layers having a shear wave velocity greater than or equal to 3 km/s and it is often composed of granite in Japan. Among those effects, the influence of sediments above the seismological bedrock is so significant that it is important to accurately evaluate the site effects to estimate ground motions during future earthquakes at a construction site. Regarding the site effects, it has been increasingly recognized that, in addition to the influence of shallower sediments above the engineering bedrock, the influence of deeper sediments below the engineering bedrock is also significant[1]. In-situ earthquake observations and microtremor measurements can be a useful tool to evaluate the site effects as explained below. The existence of sediments affects not only the amplitude but also the temporal characteristics of earthquake ground motions. In the following, its effects on the amplitude will be called the "site amplification factors". Its effects on earthquake ground motions in general will be simply referred to as the "site effects."

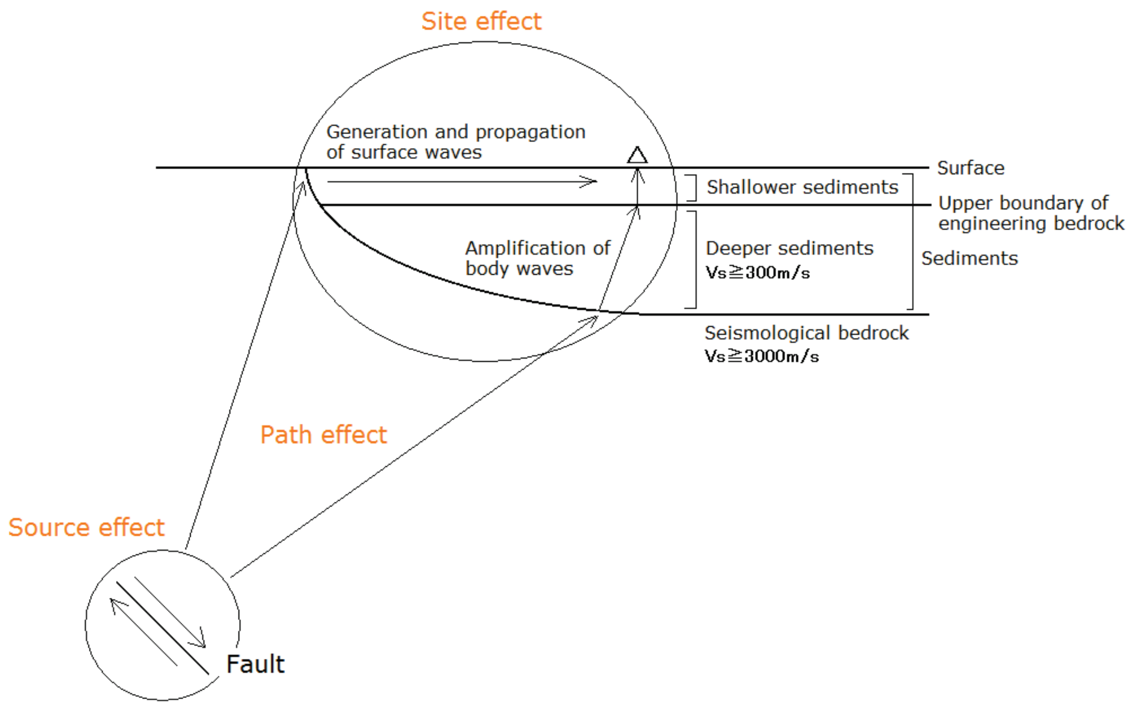

Fig. 4.1.1 Source, path and site effects

As explained in **PartII, Chapter 6, 1 Earthquake Ground Motions**, level-1 and level-2 ground motions for the performance verification of structures should be determined appropriately, taking into account the source, path and site effects. Regarding level-1 ground motions, time history data at major ports, etc. at the engineering bedrock that were determined taking account of regional source and path effects are available at the website of the National Institute for Land and Infrastructure Management at http://www.ysk.nilim.go.jp/kakubu/kouwan/sisetu/sisetu.html. In some cases, however, it cannot be guaranteed that the site amplification factor that was used to calculate a level-1 ground motion is equivalent to the site amplification factor at a construction site. In that case, it is necessary to confirm this equivalence by using microtremor measurements. If they are equivalent, the existing level-1 ground motion available at the website can be used without correction. If they are not equivalent, it is necessary to evaluate the site amplification factor at the construction site by means of earthquake observations and/or microtremor measurements and to correct the existing level-1 ground motion before it is used for the design. Regarding level-2 ground motions, detailed descriptions in terms of the source and path effects can be found in **Part II, Chapter 6, 1 Earthquake Ground Motions**. However, it is necessary to evaluate the site amplification factor at the construction site by means of earthquake observations and/or microtremor measurements.

Therefore, recommended procedures to evaluate a site amplification factor by means of earthquake observations and/or microtremor measurements will be explained in the following section. Microzonation of a port in terms of site amplification factors based on microtremor measurements and its application to port planning will also be addressed. For the methods to correct a level-1 ground motion or to set up a level-2 ground motion based on the evaluated site amplification factor, see **Part II, Chapter 6, 1 Earthquake Ground Motions**.

A method to confirm that the site amplification factor at a construction site is equivalent to the site amplification factor at a nearby strong motion station is explained in **Reference** (**Part II), Chapter 1, 4.2 Microtremor Measurements at the Construction Site and in Its Vicinity**. The same method can be used to confirm that the site amplification factor at a construction site is equivalent to the site amplification factor that was used to calculate an existing level-1 ground motion. In that case, “the strong motion station” in **Reference** (**Part II), Chapter 1, 4.2 Microtremor Measurements at the Construction Site and in Its Vicinity** should be replaced by “the site where the site amplification factor was evaluated and used for calculating the existing level-1 ground motion”.

4.2 Microtremor Measurements at the Construction Site and in Its Vicinity

It is desirable to evaluate the site amplification factor at a construction site based on earthquake observations. In Japanese ports and harbours, strong-motion observations have been conducted. The records obtained by the network can be downloaded from the website of the Ports and Harbours Bureau, Ministry of Land, Infrastructure, Transport and Tourism at http://www.mlit.go.jp/kowan/kyosin/eq.htm. In addition, there are other strong motion networks such as K-NET[2)], KiK-net[3)] and the network of seismic intensity meters by the JMA and local governments. Therefore, it is efficient to first consider the availability of the existing strong motion stations. For that purpose, it is necessary to conduct microtremor measurements at the construction site and a nearby strong motion station (**Fig. 4.2.1**). If the characteristics of microtremors are similar between the two locations, it is reasonable to assume that the dynamic characteristics of the ground are similar between the two locations. In that case the site amplification factor at the nearby strong motion station can be used for the construction site. The nearby strong motion station does not have to be a station that was used for the calculation of an existing level-1 ground motion at the website of the National Institute for Land and Infrastructure Management. It is more preferable to select a strong motion station where the site amplification factor presumably resembles the site amplification factor at the construction site. Geographical and/or geological information is useful for the selection of the strong motion station. For example, if the construction site is a rock site, it is preferable to select a strong motion station at a rock site. If the construction site is located on sediments, it is preferable to select a strong motion station located on sediments. If only one strong motion station is selected for microtremor measurements, it is likely that different microtremor H/V spectra will be obtained at the construction site and the strong motion station; it is recommended to select multiple nearby strong motion stations for microtremor measurements. On the other hand, regarding the microtremor measurements at the construction site, if the measurements are conducted at only one location, peculiar data can be obtained due to some unexpected conditions such as a localized noise source or a buried object; therefore, it is necessary to conduct microtremor measurements at least at three locations. Detailed suggestions for the implementation of microtremor measurements can be found in **Reference (Part II), Chapter 1, 4.4 Evaluation of Site Amplification Factors Based on Microtremor Measurement**.

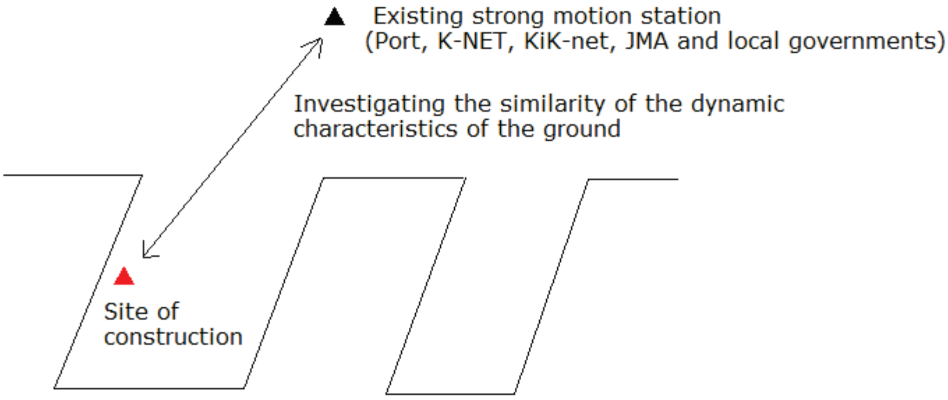

Fig. 4.2.1 Microtremor measurements at a construction site and a nearby strong motion station

Regarding the site amplification factor at the nearby strong motion station, exiting results[4)5)6)] can be used if they are available for the station. If they are not available, the site amplification factor can be evaluated by an approach based on spectral ratios similar to those explained in **Reference (Part II), Chapter 1, 4.3 Evaluation of Site Amplification Factors Based on In-situ Earthquake Observations**.

If the results of the microtremor measurements indicate that the dynamic characteristics of the ground are different between the construction site and any of the nearby strong motion stations of the Strong-Motion Earthquake Observation in Japanese Ports or other strong motion networks, it is desirable to evaluate the site amplification factor at the construction site by means of in-situ earthquake observations, depending on the importance of the project. Details of the evaluation of the site amplification factor by means of in-situ earthquake observations can be found in **Reference (Part II), Chapter 1, 4.3 Evaluation of Site Amplification Factors Based on In-situ Earthquake Observations**. If it is difficult to conduct in-situ earthquake observations because of, for example, an insufficient period of construction, the site amplification factor at the construction site can be evaluated by using the results of microtremor measurements at the construction site following the method described in **Reference (Part II), Chapter 1, 4.4 Evaluation of Site Amplification Factors Based on Microtremor Measurements**, based on the site amplification factor at a nearby strong motion station.

In the following example, the equivalence of site amplification factors was examined based on microtremor measurements. Nozu and Wakai[7)] observed microtremors at public wharves of Ishinomaki Port, Japan, and at a nearby strong motion station "K-NET Ishinomaki" to understand the variation of site amplification factors. The locations where microtremors were observed are shown in **Table 4.2.1** and **Fig. 4.2.2**. At No.9, aftershocks of the 2011 Tohoku earthquake were also observed.

First, at K-NET Ishinomaki, the site amplification factor based on earthquake observations[4)] was compared with the microtremor H/V spectrum. The H/V spectrum was calculated following the procedure described in **Reference (Part II), Chapter 1, 4.4 Evaluation of Site Amplification Factors Based on Microtremor Measurements**. The same procedure was applied to all the H/V spectra in the following sections. The result is shown in **Fig. 4.2.3**. At K-NET Ishinomaki, the H/V spectrum has a clear peak at around 0.95 Hz, while the site amplification factor also has a clear peak almost at the same frequency and thus they are consistent with each other. This consistency was considered in examining the results of microtremor measurements at other locations.

In **Fig. 4.2.4**, the microtremor H/V spectrum at K-NET Ishinomaki is compared to those at other locations. In each panel, the vertical dotted lines indicate the peak frequencies at the aftershock observation site (No.9) and K-NET Ishinomaki (No.1), which are 0.7 Hz and 0.95 Hz, respectively. The characteristics of microtremors at No.2 through No.6 resemble those at K-NET Ishinomaki. The characteristics of microtremors at No.7 also resemble those at K-NET Ishinomaki rather than those at the aftershock observation site, although from a geographical point of view No.7 is closer to the aftershock observation site than K-NET Ishinomaki. However, at No.9 and No.10 out of the three locations at Hibarino Wharf Quay Wall (-13m), the peak frequencies are lower than that of K-NET Ishinomaki.

Based on these results, it was suggested[7)] that, while the existing site amplification factor at K-NET Ishinomaki can be applied to facilities in Zone 1 in **Fig. 4.2.2**, the site amplification factor at Hibarino Wharf Quay Wall (-13m) in Zone 2 in **Fig. 4.2.2** should be newly evaluated.

Table 4.2.1 Locations where microtremors were observed at Ishinomaki Port, Japan

ID	Location
No.1	K-NET Ishinomaki (Strong motion station)
No.2	Nakajima Wharf
No.3	Oote Wharf
No.4	Hiyori Wharf
No.5	Shiomi Wharf
No.6	Minamihama Wharf
No.7	Hibarino Wharf Quay Wall (-10m)
No.8	Hibarino Wharf Quay Wall (-13m) North
No.9	Hibarino Wharf Quay Wall (-13m) Center (Aftershocks were also observed)
No.10	Hibarino Wharf Quay Wall (-13m) South

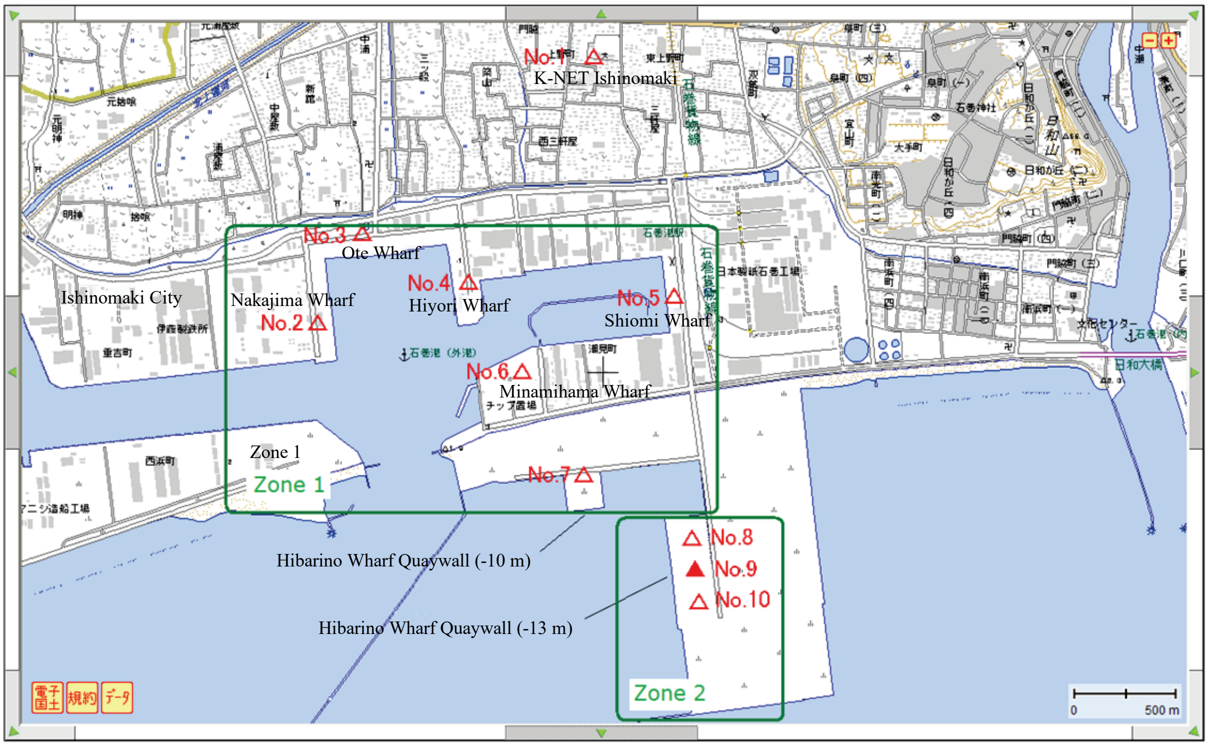

Fig. 4.2.2 Locations where microtremors (open triangles) and aftershocks of the 2011 Tohoku earthquake (solid triangles) were observed at Ishinomaki Port, Japan

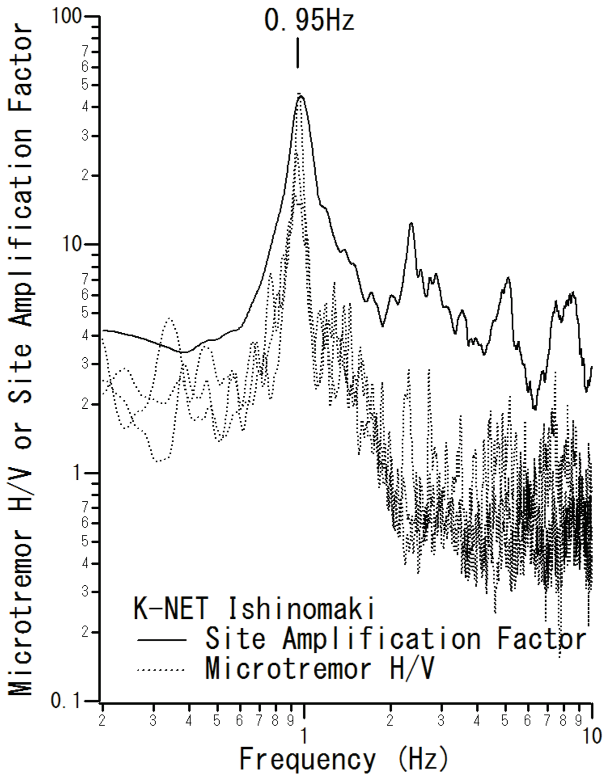

Fig. 4.2.3 Microtremor H/V spectrum and site amplification factor at K-NET Ishinomaki

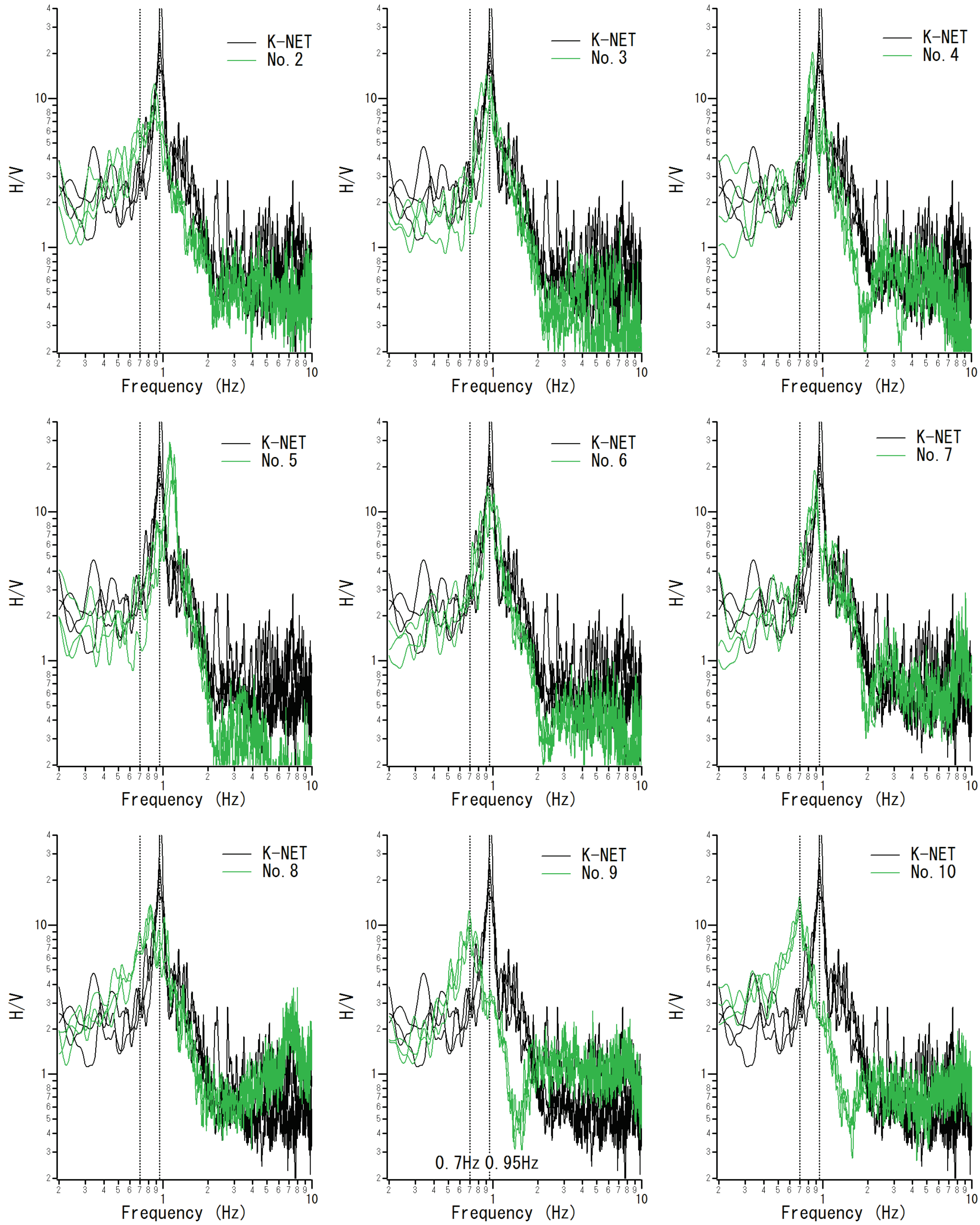

Fig. 4.2.4 Microtremor H/V spectra at mooring facilities compared with that at K-NET Ishinomaki

4.3 Evaluation of Site Amplification Factors Based on In-situ Earthquake Observations

If the results of microtremor measurements at a construction site and nearby strong motion stations of the Strong-Motion Earthquake Observation in Japanese Ports or other strong motion networks indicate that the dynamic characteristics of the ground are different between the construction site and any of the nearby strong motion stations, it is desirable to evaluate the site amplification factor at the construction site by means of in-situ earthquake observations, depending on the importance of the project. In that case, it is necessary to confirm that the dynamic characteristics of the ground are similar between the construction site and the site of the in-situ earthquake observations. Although the required period of observation can depend on regional seismicity, if the observation is continued for one to several years, weak motion records of regional small events or distant large events can generally be obtained, which can be used to evaluate the site amplification factor. When there is a restriction in the observation period, for the purpose of collecting as many records as possible, a smaller trigger level is generally used for the seismometer to start compared to observations targeted for strong motions. One of the recommended approaches is to use a seismometer which is triggered when the velocity rather than the acceleration exceeds a prescribed value to avoid the effects of background noise. Another recommended approach is to use a seismometer which can continuously observe ground motions irrespective of the occurrence of earthquakes and to pick up data corresponding to earthquakes afterwards.

Once the same event is observed at the construction site and at a nearby strong motion station with a known site amplification factor, the site amplification factor at the construction site can be evaluated as follows: If the hypocentral distance is sufficiently large, *e.g.*, if the hypocentral distance is 10 or more times greater than the distance between the observation sites, the Fourier spectral ratio between the sites can be regarded as the ratio of the site amplification factors, because the two records share the same source and path effects. Thus, if the known site amplification factor at the strong motion station is multiplied by the Fourier spectral ratio, it will yield the site amplification factor at the construction site. In this process, Fourier spectra are generally smoothed with a Parzen window with a band width of 0.05 Hz and the geometric mean of the Fourier spectral ratios for different events is generally used. It is preferable to use at least three earthquakes. However, even if there is only one earthquake available, the site amplification factor evaluated based on the records of the earthquake is more reliable than that evaluated based only on microtremor measurements following the procedure described in **Reference (Part II), Chapter 1, 4.4 Evaluation of Site Amplification Factors Based on Microtremor Measurements**.

It should be noted that records of earthquakes with very small magnitudes are often unreliable in a low frequency range. The omega-square model[8)] can be referred to in order to confirm the reliability of a record in a low frequency range. According to the omega-square model, the acceleration Fourier spectrum of a seismic wave radiated at the earthquake source, *i.e.*, the acceleration source spectrum can be represented as

$$S(f) = C \frac{M_0}{4\pi\rho V_S^3} \frac{(2\pi f)^2}{1+(f/f_c)^2} \tag{4.3.1}$$

where

M_0 : Seismic moment

f : Frequency

f_c : Corner frequency

ρ : Density in the seismological bedrock

V_s : Shear wave velocity in the seismological bedrock

C : Constant (see equation (1.3.5) in **Part II, Chapter 6, 1 Earthquake Ground Motions**)

Fig. 4.3.1 shows the displacement, velocity and acceleration source spectra following the omega square model. **Equation (4.3.1)** and **Fig. 4.3.1** indicate that the acceleration source spectra following the omega square model are proportional to the squared frequency for frequencies below f_c and are constant for frequencies above f_c. It is obvious that an acceleration Fourier spectrum observed at a site does not necessarily follow the omega-square model because it has been affected by the path and site effects, however, the spectral shape in **Fig. 4.3.1** is still informative. **Fig. 4.3.1** indicates that the acceleration Fourier spectrum of an earthquake with a very small magnitude is small in a low frequency range and easily masked by a noise. If the acceleration Fourier spectrum of a record is not increasing with frequency in a low frequency range, it is reasonable to decide that the record is not reliable at the frequency range. In the case of a port facility, it is preferable to use a record which is reliable down to 0.2 Hz, which means that a record with a very small magnitude is not preferable. On the other hand, records of distant large earthquakes are often reliable down to lower frequencies and available for the evaluation of site amplification factors, although they may not be suitable for a generalized inversion[4)].

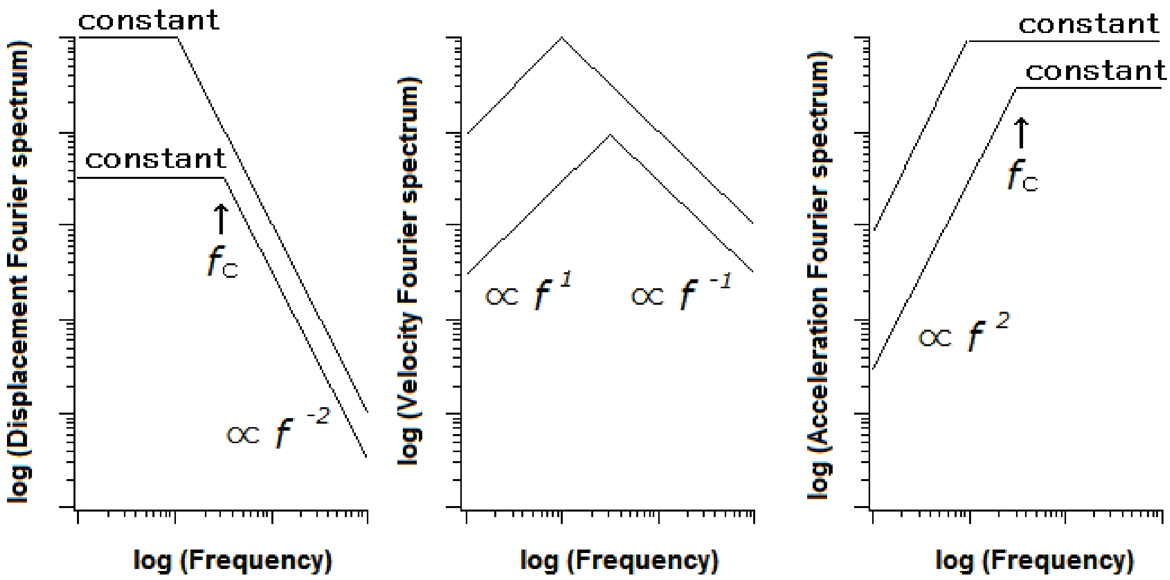

Fig. 4.3.1 Displacement, velocity and acceleration source spectra following the omega-square model

It is also necessary to pay attention to the length of the records to be analyzed. In some literature, the "S wave portion" is isolated from an observed ground motion and the Fourier spectrum is calculated for that portion. However, if the design of a port structure is concerned, it is necessary to account not only for the S waves but also for the surface waves and to calculate a Fourier spectrum that includes the effects of later phases. If a record with an insufficient length is used, the Fourier spectrum and the spectral ratio may be inappropriately calculated.

If the hypocentral distance is not sufficiently large, it is not reasonable to postulate that the records of the same earthquake at the construction site and the strong motion station share the same source and path effects. In that case, the following procedure can be followed: First, the source spectrum of the earthquake under study should be determined appropriately so that the records at nearby strong motion stations can be reproduced. Then, the site amplification factor at the construction site can be evaluated by dividing the Fourier spectrum at the construction site with the source and path effects[9]. It should be noted that, if the azimuth and/or the take-off angle are quite different for the construction site and the strong motion stations, the accuracy of the evaluation could be degraded because the source spectrum can be dependent on the azimuth and the take-off angle.

The following is an example of the evaluation of a site amplification factor based on in-situ earthquake observations. At Ishinomaki Port, earthquake observations were conducted from the evening of May 13 through the morning of May 16, 2011, in Japan Standard Time at Hibarino Wharf Quay Wall (-13m), where the existing site amplification factor at a nearby strong motion station "K-NET Ishinomaki" was not applicable (see **Reference (Part II), Chapter 1, 4.2 Microtremor Measurements at the Construction Site and in Its Vicinity**). Because the observations were conducted soon after the occurrence of the 2011 Tohoku earthquake and the aftershock activities were high during the observations, a sufficient number of valid records were obtained in a short period. The observations were conducted at "No.9" indicated by a solid triangle in **Fig. 4.2.2**. At K-NET Ishinomaki, *i.e.*, "No.1" in **Fig. 4.2.2**, the seismometer was under operation during the above mentioned period. During the period, seven events shown in **Table 4.3.1** were recorded both at No.9 and K-NET Ishinomaki. **Fig. 4.3.2** compares the Fourier spectra for the two locations for each event. The Fourier spectra are the composition of two horizontal components. They were smoothed with a Parzen window with a band width of 0.05 Hz. While the Fourier spectra at K-NET Ishinomaki always have a peak at 0.95 Hz, those at No.9 always have a peak at 0.7 Hz, indicating the difference of the dynamic characteristics of the ground. **Fig. 4.3.3** shows the spectral ratios at No.9 with respect to K-NET Ishinomaki. In accordance with the above mentioned characteristics of the Fourier spectra, the spectral ratios always have a positive peak at 0.7 Hz and a negative peak at 0.95 Hz. The scatter is relatively small for different events. The existing site amplification factor at K-NET Ishinomaki[4] was multiplied by the geometric mean of the spectral ratios to evaluate to the site amplification factor at No.9, which is shown in **Fig. 4.3.4**. While the site amplification factor at K-NET Ishinomaki has a peak at 0.95 Hz, the site amplification factor at No.9 has a peak at 0.7 Hz. The obtained site amplification factor is consistent with the microtremor H/V spectrum at the same location. The lower peak frequency at No.9 presumably indicates the existence of thicker sediments for that part of the port.

Table 4.3.1 Aftershocks of the 2011 Tohoku earthquake observed at Ishinomaki Port, Japan

ID	Date and time (JST)	Epicenter	Approximate depth	Magnitude
EQ1	5/14/2011 5:17	Off Fukushima	40km	4.4
EQ2	5/14/2011 8:36	Off Fukushima	30km	5.7
EQ3	5/15/2011 1:45	Off Miyagi	40km	4.0
EQ4	5/15/2011 8:51	Off Fukushima	50km	5.0
EQ5	5/15/2011 18:56	Off Miyagi	50km	4.1
EQ6	5/15/2011 21:14	Off Fukushima	10km	5.4
EQ7	5/16/2011 4:07	Off Miyagi	50km	4.6

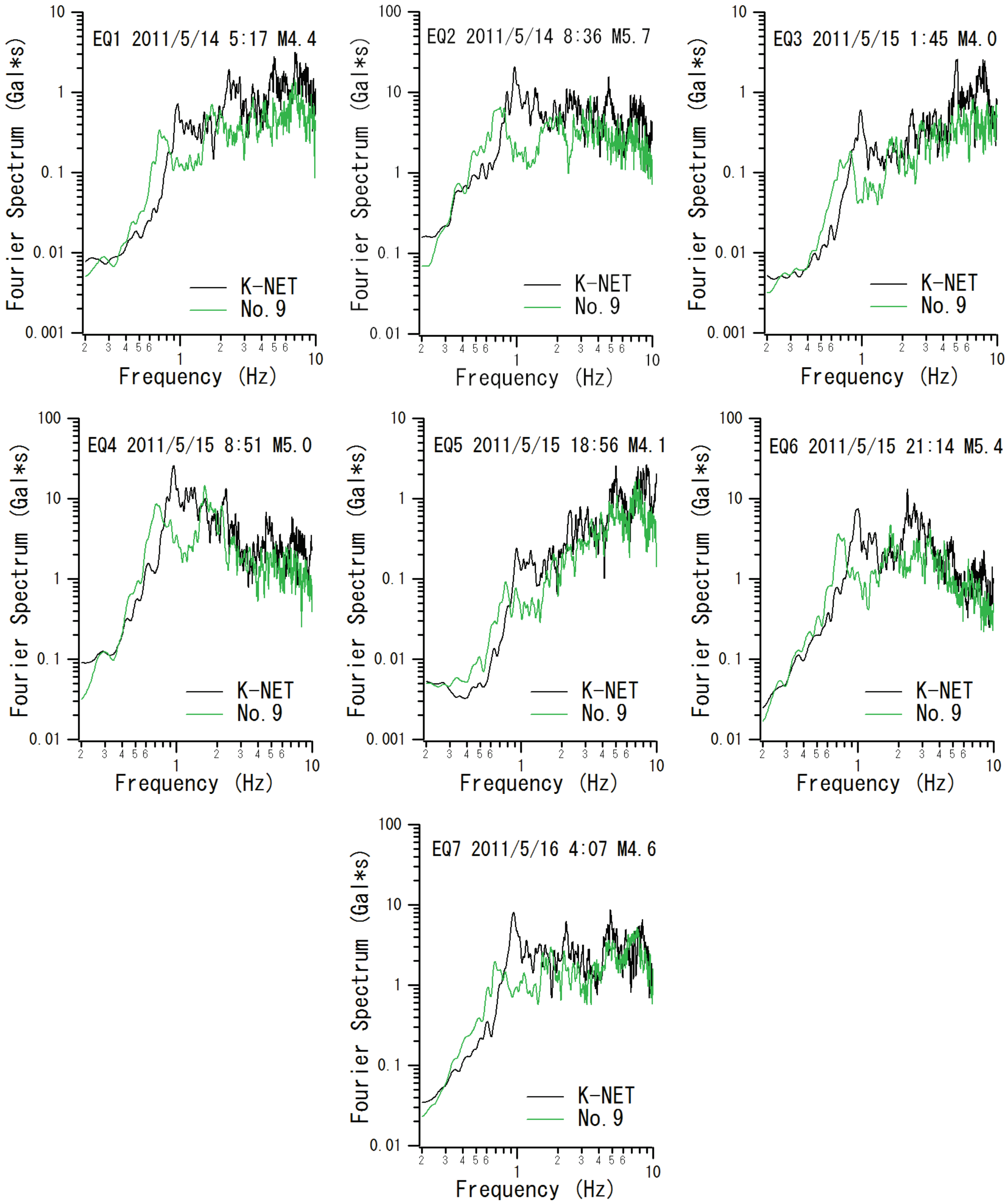

Fig. 4.3.2 Fourier spectra observed at K-NET Ishinomaki and No.9

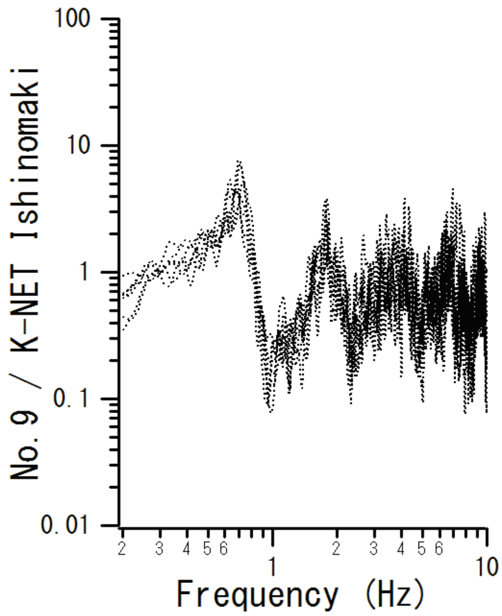

Fig. 4.3.3 Spectral ratios at No.9 with respect to K-NET Ishinomaki for seven events

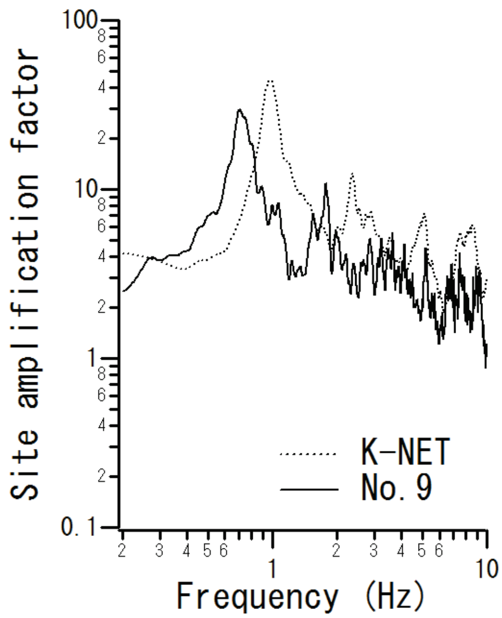

Fig. 4.3.4 Site amplification factor newly evaluated at No.9 at the ground surface with respect to the seismological bedrock, compared with that at K-NET Ishinomaki

4.4 Evaluation of Site Amplification Factors Based on Microtremor Measurements

If the results of microtremor measurements at a construction site and nearby strong motion stations of the Strong-Motion Earthquake Observation in Japanese Ports or other strong motion networks indicate that the dynamic characteristics of the ground are different between the construction site and any of the nearby strong motion stations, it is desirable to evaluate the site amplification factor at the construction site by means of in-situ earthquake observations, depending on the importance of the project (see **Reference (Part II), Chapter 1, 4.3 Evaluation of Site Amplification Factors Based on In-situ Earthquake Observations**). However, if it is difficult to conduct in-situ earthquake observations because of, for example, an insufficient period of construction, the site amplification factor at the construction site can be evaluated by using the results of microtremor measurements at the construction site based on the site amplification factor at a nearby strong motion station as follows:

First, detailed suggestions for the implementation of microtremor measurements are as follows:

Velocity or acceleration sensors are generally used for microtremor measurements. At a site on thick sediments, velocity sensors can be more advantageous because they are applicable to even lower frequencies.

The purpose of the microtremor measurements here is to measure the vibration of the ground. Therefore, the microtremor measurements should not be conducted at a place where the effects of the existence of structures are significant. For example, they should not be conducted on a quay wall, on a pile-supported structure, above a cavity, above a large buried structure, at an immediate vicinity of a large building, etc. In a recent study[10)], it was reported that, even if microtremor measurements are not conducted on a quay wall, if they are conducted on the ground just behind a quay wall, the results can be affected by the existence of the quay wall and a peak that cannot be attributed to the characteristics of the ground can appear in the spectrum especially for the face-line normal component. In the study, it was suggested that the effects of the existence of the quay wall can be avoided if the microtremor measurements are conducted a sufficient distance away from the face line of the quay wall. Therefore, when microtremor measurements are conducted near a quay wall, they should preferably be conducted away from the face line with a distance of three times the height of the wall and, in addition to the ordinary H/V spectra calculated from the root mean square of the two horizontal components as explained later, the H/V spectra should also be calculated from the face-line parallel component and be used for further analyses upon necessity.

The quality of the observation results can be degraded if a localized noise source such as a car or heavy machinery exists in the immediate vicinity. In that case, it could be necessary to conduct microtremor measurements during quiet hours of the day. It is preferable to put the sensor on asphalt or concrete. If the measurements are done on soils, sands or grasses, the ground should first be compacted by foot. If it is difficult to compact the ground, the sensor should be put on a steel plate placed on the ground. Weather conditions such as rain, wind or snow can affect the results; it is necessary to pay attention to the weather conditions to determine the schedule. Because the effects of wind are significant, a windbreak should always be used unless there is no wind at all.

Regarding the microtremor measurements at the construction site, if the measurements are conducted at only one location, peculiar data can be obtained due to some unexpected conditions such as a localized noise source or a buried object; therefore, it is necessary to conduct microtremor measurements at least at three locations.

Three components of microtremors including two horizontal and one vertical components are generally observed simultaneously with a sampling frequency of 100 Hz.

For the performance verification of a port structure, the H/V spectrum should be calculated as follows:

(1) The measurement should be conducted continuously for more than 11 minutes and three time sections with a duration of 163.84 s with less contamination from traffic noise, etc. should be selected.

(2) The Fourier amplitude spectra should be calculated for horizontal and vertical components and smoothed with a Parzen window with a band width of 0.05 Hz.

(3) The root mean square of two orthogonal horizontal components should be calculated to be used as the numerator of the H/V spectrum. However, as mentioned above, the face-line parallel component should be used as the numerator upon necessity.

(4) The numerator mentioned above should be divided by the denominator, that is, the vertical component and averaged for the three time sections to obtain the H/V spectral ratio.

It is important to make sure that the H/V spectral ratios calculated from different time sections do not exhibit a significant scatter. If they exhibit a significant scatter, it may imply that the H/V spectra are contaminated by localized noise sources; it may be necessary to conduct the measurements on another day. For the allowable scatters, refer to **Fig. 4.2.4**. The low frequency components of an H/V spectrum are relatively easily contaminated by localized noise sources, leading to wrong recognition of a "peak". The reliability of the low frequency components of an H/V spectrum should be examined carefully.

The site amplification factor at the construction site can be evaluated by translating the existing site amplification factor at a nearby strong motion station on log-log axes so that the peak frequency of the site amplification factor coincides with that of the microtremor H/V spectrum at the construction site[11)]. The method was applied to the existing site amplification factor at K-NET Ishinomaki to evaluate the site amplification factor at No.9. The result is compared with the site amplification factor at the same location evaluated based on in-situ earthquake observations in **Fig. 4.4.1**.

If the peak of the microtremor H/V spectrum is much higher for the construction site than for the strong motion station, it is anticipated that the peak of the site amplification factor may also be much higher for the construction site. In that case, to avoid possible underestimation of the site amplification factor at the construction site, the site amplification factor after the translation mentioned above should be multiplied by the following function:

$$r(f) = \begin{cases} \dfrac{1}{\sqrt{\cos^2\left(\dfrac{\pi f}{2f_0}\right) + R^2 \sin^2\left(\dfrac{\pi f}{2f_0}\right)}} & (f \le 2f_0) \\ 1 & (f > 2f_0) \end{cases} \quad \textbf{(4.4.1)}$$

where

f_0 : Peak frequency of the microtremor H/V spectrum at the construction site

R : p_1/p_2

p_1 : Peak height of the site amplification factor before correction

p_2 : Peak height of the site amplification factor after correction

p_2 should be evaluated as follows:

$$p_2 = 26.1 p_m^{\ 0.21} \quad \textbf{(4.4.2)}$$

where

p_m : Peak height of the microtremor H/V spectrum at the construction site

One of the limitations of the evaluation of the site amplification factor mentioned above is that it is only applicable when the peak frequencies of the microtremor H/V spectra are clear both for the strong motion station and the construction site. The peak frequency of a site amplification factor can become ambiguous when the site is located at a place where the seismological bedrock is significantly deep, for example. In addition, as mentioned in **Part II, Chapter 6, 1 Earthquake Ground Motions**, uncertainties are inherent in the evaluation of the peak height of a site amplification factor based on microtremor measurements. Therefore, in the event of setting up design ground motions for a very important structure, it is desirable to evaluate the site amplification factor at the construction site based on earthquake observations. On the other hand, microtremor measurements are advantageous in setting up design ground motions for a number of facilities at the same time or in setting up ground motions for a preliminary study.

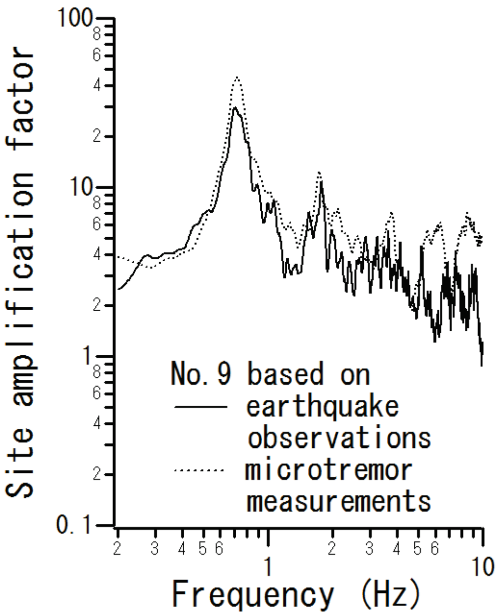

Fig. 4.4.1 Site amplification factor newly evaluated at No.9 at the ground surface with respect to the seismological bedrock based on microtremor measurements, compared with that based on earthquake observations

4.5 Microzonation of a Port Based on Microtremor Measurements

If microtremors are densely observed at a port, microzonation of the port in terms of site amplification factors can be implemented based on the results. The procedure is shown in **Fig. 4.5.1**.

① Collect and compile geotechnical data: Collect existing geotechnical data for shallower and deeper soils and compile existing knowledge on the engineering bedrock, the seismological bedrock and the sediments.

② Determine locations for microtremor measurements: Based on the above results, determine the locations for microtremor measurements. The measurements are often conducted at intervals of 200 m.

③ Execute measurements: Execute microtremor measurements at the designated locations. Refer to **Reference (Part II), Chapter 1, 4.4 Evaluation of Site Amplification Factors Based on Microtremor Measurements**.

④ Calculate microtremor H/V spectrum: Refer to **Reference (Part II), Chapter 1, 4.4 Evaluation of Site Amplification Factors Based on Microtremor Measurements**.

⑤ Implement microzonation: Based on the peak frequencies and shapes of the microtremor H/V spectra, divide the port into several zones, each having similar characteristics of microtremors.

⑥ Evaluate the site amplification factor for each zone: Based on weak motion records at an existing strong motion station or at a temporary station for earthquake observation, evaluate the site amplification factor for each zone (see **Reference** (**Part II), Chapter 1, 4.3 Evaluation of Site Amplification Factors Based on In-situ Earthquake Observations**).

As an example, **Fig. 4.5.2** shows the locations where microtremors were observed at Kushiro Port, Japan, and the resultant microzonation of the port.

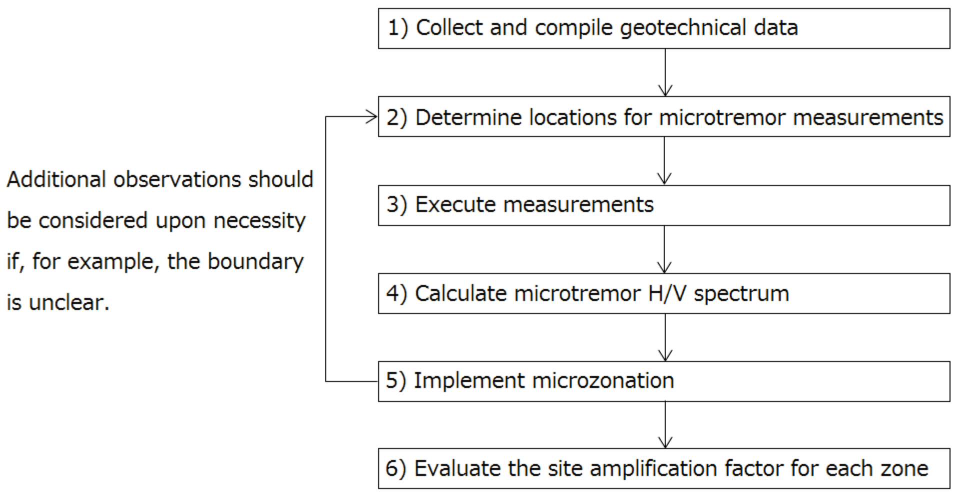

Fig. 4.5.1 Procedure for microzonation of a port based on microtremor measurements

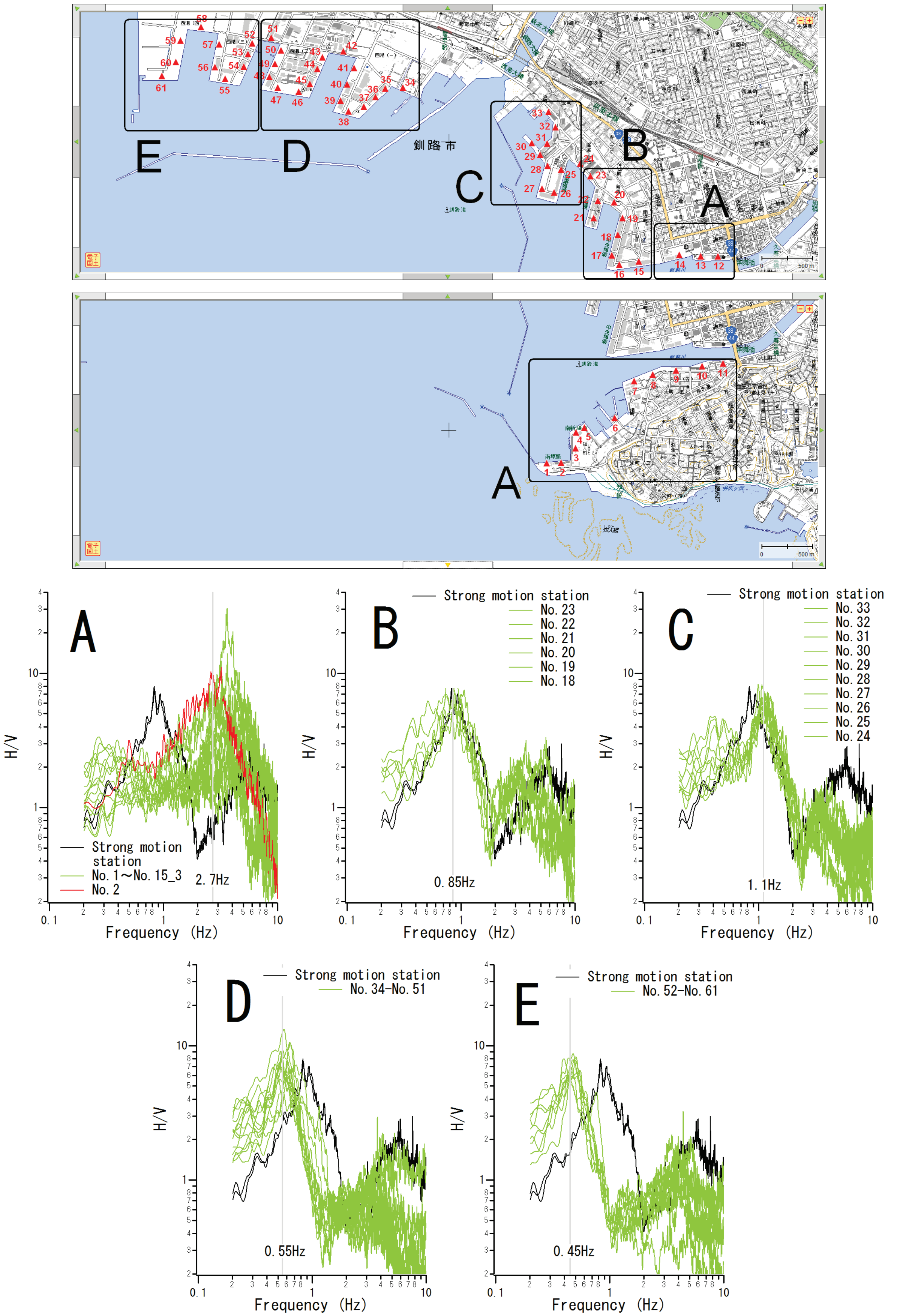

Fig. 4.5.2 Locations where microtremors were observed (solid triangles) at Kushiro Port, Japan, and the resultant microzonation of the port

4.6 Application of Microtremor Measurements to Port Planning

Damage to port structures due to strong ground motions during the 2011 Tohoku earthquake revealed that ground motions can vary significantly even within a port, resulting in variations of the extent of damage. In preparation for future earthquakes, it is necessary to reveal the distribution of the site amplification factors within a port and to take careful countermeasures for facilities located in a zone with a large site amplification factor. Furthermore, construction costs can be lowered and earthquake resistance can be increased if priority is given to locating a port facility in a zone with a small site amplification factor. For example, when an existing quay wall is to be reinforced and designated as a high seismic resistant quay wall, if an existing quay wall located in a zone with a small site amplification factor is selected, the reinforcement can be completed at less cost.

As mentioned in **Part II, Chapter 6, 1 Earthquake Ground Motions Atmospheric Pressure**, the microtremor H/V spectrum and the site amplification factor at the same site are generally consistent with each other. Microtremor measurements can be used to answer such questions as "Is the amplification due to the existence of sediments anticipated at the construction site?" or "At which frequency does the amplification occur?" If a zone is characterized by microtremor H/V spectra with a peak in the frequency range of 0.3 – 1 Hz, which can easily affect mooring facilities, the zone is disadvantageous from a viewpoint of earthquake resistance of a mooring facility.

Microtremor measurements can be used in a port planning as follows:

STEP1 Execution of microtremor measurements

- Microtremor measurements are generally cost effective.
- Microtremor data may be already available; they may have been collected for the design of a structure.

STEP2 Implementation of microzonation

- Zones where relatively strong ground motions are anticipated should be distinguished from zones where relatively weak ground motions are expected.
- Microzonation has already been implemented for some ports for the design.

STEP3 Application of the results for planning mooring facilities, etc.

- A comprehensive judgement is required, taking into account other factors.

As an example, if a high seismic resistant quay wall is to be planned in Kushiro Port shown in **Fig. 4.5.2**, Zone A is advantageous because there is no peak in the microtremor H/V spectra in the frequency range of 0.3 – 1 Hz, which can easily affect mooring facilities.

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5 General Matters for Examinations and Tests of Structures and Raw Materials

5.1 Outline

5.1.1 Tests and Examinations of Raw Materials in Facility Development

Raw materials to be used for port facilities should be appropriately selected in consideration of actions, deterioration, design service life, shapes, constructability, economic efficiency, influence on the environment and other factors. Quality and durability should be appropriately considered when making selections.

The main raw materials of members to be used for facilities to which the technical standards apply are steel, concrete, bituminous materials, stone, wood, other metallic materials, plastic, rubber, coating materials, grouting materials, reclamation materials (including waste) and recyclable resource materials (e.g., slag, coal ash, crushed concrete, dredged soil, asphalt concrete modules and shells). Recently, hybrid members in which these raw materials are combined have also been used. New raw materials are included in some cases, so details of performance and property tests also vary.

Physical properties are checked in the examinations and tests of raw materials and members. The required physical properties vary depending on the type of raw material (the application purpose of the material), so attention must be paid to the physical properties (unit weight and friction coefficient), mechanical properties (e.g., tensile strength), plasticity, elasto-plasticity, thermal properties, electrical properties, biological properties, chemical properties and optical properties.

The physical properties of the raw materials should be appropriately determined based on the specification values through testing based on JIS standards or other highly reliable quality data. In addition, when conducting tests or experiments using a raw material at an actual service site is impossible or unfeasible, the long-term conditions and fracture processes of the raw materials should be estimated based on the physical property test results. Attention should be paid to ensure quality and safety by adding the appropriate examinations and tests, and by selecting inspection methods and carrying out analyses in an appropriate manner.

5.1.2 Tests and Examinations of Raw Materials in Improvement Design

(1) Examinations of existing structures

Existing structures should be maintained through periodic inspections and monitoring. When deterioration or damage is found, the cause should be understood. In this stage, the documents and outward appearance should be examined as a rough examination, and based on the results of the rough examination, detailed examinations on the necessary items should be planned and carried out. For the examination items and procedures, it is important to consider the required accuracy and select a satisfactory method and equipment.

① **Steel structures**

A possible test type is wall thickness measurement testing over a wide range to determine the scope to be repaired and improved and the construction method. When wall thickness data continuously measured at measurement points is kept from the time of development or from the time of maintenance, it makes it easier to understand the deterioration conditions and identify causes.

As a result of the evaluation, if repair or reinforcement countermeasures are required, additional detailed examinations should be planned. The **Corrosion Control and Repair Manual for Port Steel Structures**[1] can be referred to for steel structures for which corrosion allowance was used for the design including cases in which macrocell corrosion was found.

② **Concrete structures**

As rough examinations, traceability records for aggregates, cement, admixtures, mix proportions and other components should be closely checked in document examinations. It should be noted that, unlike steel, concrete is a raw material type for which the quality tends to largely vary due to the influence of variations in the raw material itself, the weather conditions at the time of placing and the curing conditions after placing.

For performance evaluations, the **Guidelines for Evaluating the Performance of Existing Concrete Structures**[2] can be referred to. These guidelines target buildings and civil engineering structures and show methods to evaluate if the actual performance of the structures, in consideration of the characteristics of the raw materials, variations in histories and changes over several years, exceeds the required performance quantitatively. For concrete structures, deterioration often appears as cracks. For examinations for studying causes and repair proposals, the **Guidelines for Checking for Cracks in Concrete and Repairing and Reinforcing Concrete**[3], the **Piled Pier Deterioration Examination and Repair Manual**[4] and the **Port Facility Maintenance Technology Manual**[5] are informative. Alkali-silica reaction is often mistaken for cracks due to other types of deterioration and countermeasures taken without sufficiently identifying the true cause, so attention is required.

(2) Tests of raw materials to be used for improvement

The characteristics of newly used raw materials and existing raw materials should be appropriately evaluated. At that time, when traceability records of the raw materials have been maintained and managed, sampling tests and other types of tests can be sometimes omitted. In addition, when a structure is improved or removed, strength tests and other necessary tests should desirably be performed for steel, concrete and other raw materials using test pieces taken from the existing facility.

5.1.3 Tests and Inspections of Foreign-Produced Raw Materials

When foreign-produced materials are used, they should satisfy the quality and performance standards. In addition, inspections and evidence are required for product management and general risk management such as risks of bringing in, handling of complaints and rejected materials. For this task, the **Procedure for Checking the Quality of Foreign-Produced Materials and Demonstration**[6] is informative. To date, certificates have been issued for aluminum alloy anodes, rubber fenders, anchor chains and anchors.

5.2 Steel

5.2.1 Tests of the Quality of Raw Materials

(1) Tests by steelmakers during manufacturing and at the time of shipment

When steel is used in the construction of ports, steel products manufactured by steelmakers are purchased and used directly in some cases, and manufactured products are processed and joined at construction sites in other cases. For securing the safety of port structures, JIS standards include procedures for testing the chemical compositions and mechanical properties of the predetermined steel. As long as no harmful defects are found in the steel to be used for port construction or members, which are made by processing and joining the steel, no particular examinations or tests are required.

Steelmakers ship steel after confirming that the basic physical properties, in particular, have satisfied the designated standards or specifications. Steel inspection certificates (mill sheets), which show the history and properties of the steel, are issued for steel products. Mill sheets are issued separately for each raw material type such as steel plates, H-beams, angle bars and steel rods. For example, for round steel tubes, mill sheets show the quantity, material quality, test values and other data for each charge number, test number, product number (single piece) and serial number (multiple pieces).

Mill sheets show the steel's chemical composition and tensile test, bending test and impact test results as its mechanical properties. Tensile tests should follow "**JIS Z 2241 Metallic materials - Tensile testing - Method of test at room temperature**" and "**JIS B 7721 Tension/compression testing machines - Calibration and verification of the force-measuring system**," and bending tests should follow "**JIS Z 2248 Metallic materials - Bend test**." For the tests, a test sample should be taken from each lot (when the weight of one lot exceeds 50 tons, two samples should be taken). For impact tests, three test pieces should be taken from the thickest steel in each test lot and the average of the test results used as the test value.

The chemical compositions that are shown are often the results of analyses of samples for each ladle at the time of molten steel tapping with various types of emission spectrometry.

(2) Quality tracing when steel is brought in

Generally, users of raw materials (e.g., entities that received a construction order) stock general-purpose steel products in certain quantities and use them for each construction project, so it is important to identify which serial numbers (raw materials) were used for each construction.

Users of raw materials (e.g., entities that receive a construction order) carry out a receiving inspection each time the ordered products are delivered and manage them during usage so that it is possible to trace which raw materials were used for each construction project, including cases where multiple types of steel products were used. To ensure the traceability of the construction steel, the **Steel Quality Demonstration Guidelines**[7] are provided. In receiving inspections at the time of delivery, the delivered products should be appropriately measured, and the specifications and dimensions should be checked to see if there are differences from those at the time of shipment from the manufacturer.

(3) Field tests

In port construction, steel products are processed at the actual construction sites for use. Attention is required for the welding of steel pipe piles and steel bars, shoring in which different types of steel are combined, members for installing steel shell caissons, etc., in addition to quality management at factories.

For quality management of steel when structures or members are combined and processed at actual sites, there is a possibility that the characteristics and physical properties of the raw materials will need to be checked in addition to checking the mill sheets. However, destructive inspections at the actual sites under service environments are impossible or unfeasible. Therefore, the goal shall be predicting the behavior to actual breakage through various types of physical property tests for the raw materials. However, reproducing the conditions, such as loading speeds and thermal environments, is difficult with such tests.

Radiographic testing and ultrasonic testing that are often used to test welds are described below among non-destructive inspections (http://www.jisc.go.jp/app/jis/general/GnrJISSearch.html).

When the **Road Bridge Specifications II - Steel Bridges** or other standards specify sections to which radiographic testing ① and ultrasonic testing ② are applied, such testing should be applied.

① Radiographic testing

The radiographic test method is used to examine the inside of the steel by radiographing based on differences in radiation absorption and is used to understand the locations and sizes of internal defects. Internal defects at welds affect the strength degradation, and, therefore, the safety of the structures themselves is concerned. As for which classification of images is allowed is determined in consideration of the importance of the structures, steel types and other factors. "**JIS Z 3104 - Methods of radiographic examination for welded joints in steel**" and the classification methods of radiographic images specify the testing procedures for welded steel joints, in particular, and the classifications of radiographic images. X-rays or γ-rays are used as radiation, and persons who carry out such tests must have qualifications as an operation chief of radiography with X-rays or a radiation protection supervisor to handle these rays safely.

For the quality management of welds, X-ray films used as test results serve as lasting records in the detection of internal defects at welds and they make it possible to distinguish the types and sizes of defects rather easily, so radiographic testing is trusted as a method that can obtain objective results. X-rays and γ-rays are very penetrative, so very small portions are absorbed by sensitive emulsions. Therefore, a sensitive emulsion has been applied on both sides of the X-ray film base to double the sensitivity and contrast. With X-ray films, the sensitivity is in contrast to the graininess, and since high-speed film is grainy, a proper film type should be used based on the penetration of the radiation source and the type of test piece.

② Ultrasonic testing

"**JIS Z 2344 - General rule of ultrasonic testing of metals by pulse echo technique**" and "**JIS Z 3060 - Method for ultrasonic examination for welds of ferritic steel**" specify the ultrasonic testing procedures for steel. The procedures for classifying test results are also provided. In this method, high-frequency (usually, 0.4 to 15 MHz) sound waves are shot via an appropriate contact medium to the steel to determine the locations and sizes of the defects based on the conditions of the reflected waves. Ultrasonic waves are used for detection for several reasons: mainly, they are reflected back at the boundary surface, their directivity is high because their wavelengths are short so the direction of the defect can be detected, and they are reflected back well even from small defects because their wavelengths are short. When ultrasonic waves travel in the air or in liquid, they are divergent waves, whereas when they travel in a solid, transverse waves travel in addition to divergent waves. In the vertical beam method, ultrasonic waves (divergent waves) that travel perpendicular to the test surface of the target steel are used for detection, and in the angle beam method, ultrasonic waves (transverse waves) that travel diagonally to the test surface are used.

The performance of ultrasonic test equipment varies depending on the manufacturer and model. Frequencies, indicating types, probes and other factors are also different, so test equipment with performance that matches the purpose of the test or inspection should be selected.

5.2.2 Steel Corrosion

Steel materials used in port structures are in highly corrosive environments. When the steel's corrosion rate is determined under an environment in which a structure is installed during the design phase, the values shown in documents such as **Table 2.3.1** in **Part II, Chapter 11, 2.3.3 Actions and Material Strength Requirements, Distribution of Corrosion Rates of Steel Structures,**" and the **Corrosion Control and Repair Manual for Port Steel Structures**[1] may be used.

The corrosion rate may be different from the general values in some environments. In these situations, a test piece should be desirably installed to determine the corrosion rate.

5.2.3 Corrosion Control of Steel

(1) Electric corrosion control

① **In design**

When the specifications (e.g., various performance factors, water contact resistance and density of corrosion control current) of an anode are determined in the design phase, the values specified in "**Part II Actions and Material Strength Requirements, Chapter 11, 2 Steel**" and in general manuals may be used.

The density of corrosion control current and resistivity may be different from the general values in some environments. In these situations, the density of corrosion control current should desirably be calculated based on existing documents and through temporary energization tests at the actual sites.

② **During construction**

Raw material inspections should be carried out by checking test records and other documents while referring to the **Corrosion Control and Repair Manual for Port Steel Structures**,[1] and welding inspections should be carried out by visual inspections and using photographs while referring to the **Corrosion Control and Repair Manual for Port Steel Structures**.[1] In completion inspections, the electric potential should be measured and checks should be carried out using test records, photographs of construction (records) and other documents. For the procedures for measuring electric potential, refer to the Reference 1).

(2) Corrosion control using coatings

The main coatings used for corrosion control are paint, organic coatings (heavy coating for corrosion control, super high build coating and underwater hardening coating), anticorrosion metal lining, petrolatum lining and mortar lining. Common specifications are described below. For details, refer to the **Corrosion Control and Repair Manual for Port Steel Structures**.[1]

① **In design**

In the design phase, the specifications of various raw materials to be used should be determined. For construction methods that have been used, values specified in **Part II, Chapter 11, 2 Steel** and in general manuals may be used.

Furthermore, for construction methods that are not often used, various properties should be appropriately determined through inquiries to the manufacturers or other means.

② **During construction**

Raw materials should be inspected by checking test records and other documents.

Completion inspections vary depending on the coating for corrosion control, but they should be carried out mainly by visual inspections. In addition, test records and other documents should be used for checks. For details regarding inspections, the **Corrosion Control and Repair Manual for Port Steel Structures**[1] may be referred to.

③ The thickness of coatings on steel structures such as movable bridges, piled piers and steel pipe piles should be checked at factories (if they are processed at actual sites, they should be checked at the sites). For details regarding inspections, refer to the **Steel Road Bridge Corrosion Control Manual**,[8] the **Corrosion Control and Repair Manual for Port Steel Structures**[1] and other similar documents.

5.3 Concrete Testing

(1) Objectives of various test types and testing flow

Concrete testing is broadly divided into the testing of concrete materials, fresh concrete, hardened concrete, concrete durability and concrete structures. **Table 5.3.1** lists the types of tests in the planning and construction stages as well as the examination objectives.

① When the concrete production processes are divided into planning, construction and the start of usage, the concrete needs to be tested in each stage as shown in **Fig. 5.3.1**.

② When planning, the concrete materials, fresh concrete and hardened concrete should be tested and the durability needs to be tested, when required, to determine the raw materials and mix proportions.

③ Regarding testing during the construction of concrete, when concrete is brought in, fresh concrete should be tested to check the properties. At the same time, test pieces for the hardened concrete test should be made and the hardened concrete at the required material age should be tested.

④ Afterwards, at the required material age, the concrete quality should be tested as a structure once it has started being used.

⑤ The **Standard Specifications for Concrete Structures - Standards (JSCE Standard and related standards)** [9] and the **Japan Concrete Institute (JCI) Standards**[10] compile the test procedures in each stage.

Table 5.3.1 Objectives of Concrete Testing

Manufacturing process	Test type	Objective of testing and how to use test results
Planning	Testing of concrete materials	When a non-standard raw material is used, its physical properties should be tested.
Planning and construction	Testing of fresh concrete	To be performed to design the mix proportions of the concrete and manage the quality when the concrete is unloaded
Planning and construction	Testing of hardened concrete	To be performed to design the mix proportions of the concrete and manage the quality of the hardened concrete
Planning	Durability testing	When it is expected that the concrete will deteriorate due to frost damage, chloride-induced corrosion or other factors, this type of test should be performed to select the raw materials of the concrete and design the mix proportions.
Start of usage	Quality testing of structures	To be performed to check the quality of the structures before the start of usage
Planning and construction	Testing of special concrete	This type of test should be performed when special concrete is used to select the raw materials of the concrete, design the mix proportions and manage the quality.

Note: For the relationship between the manufacturing processes and tests, refer to Fig. 5.3.1.

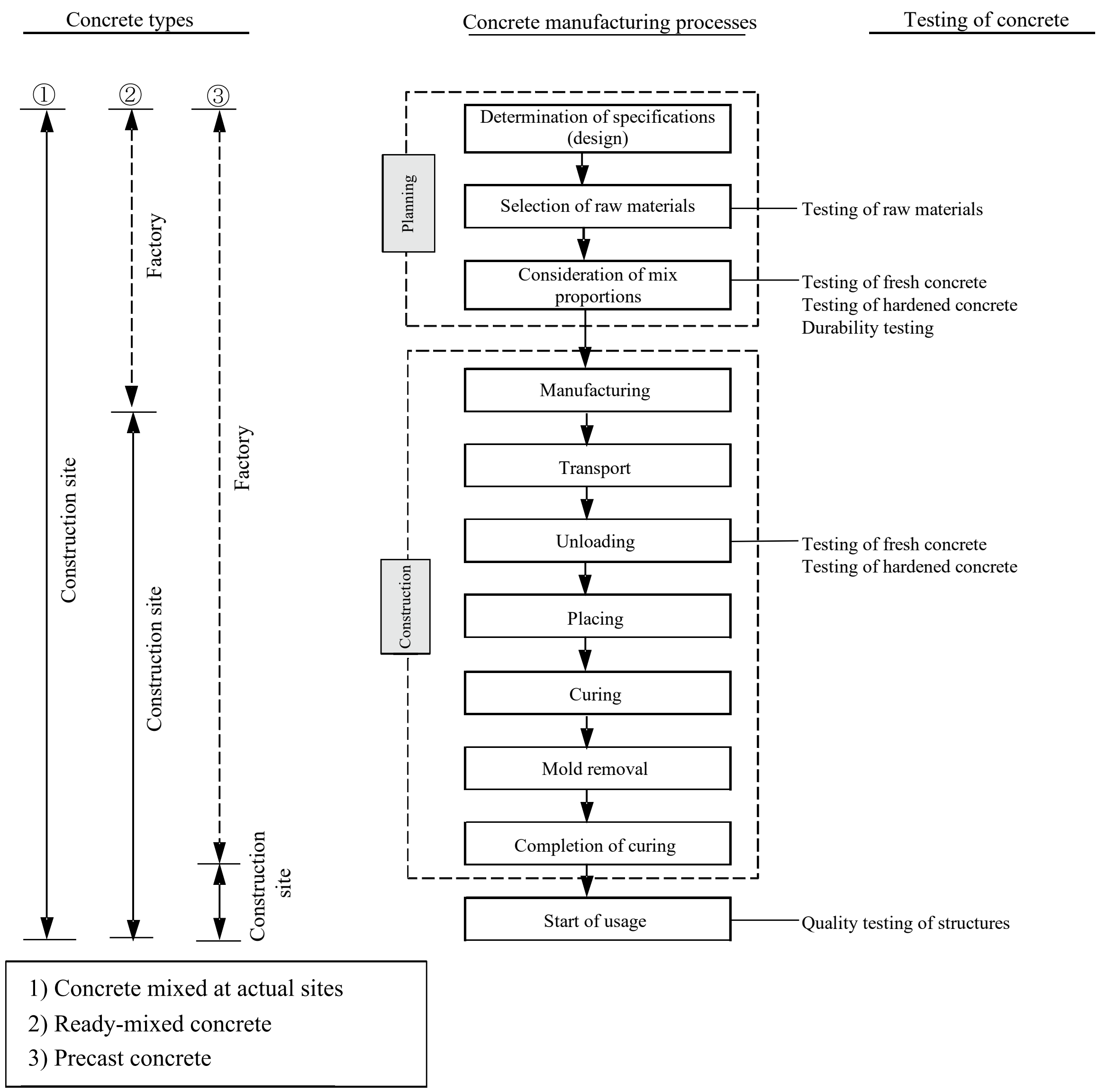

Fig. 5.3.1 Testing Flow
(Relationship between the Manufacturing Processes of Concrete Structures and Concrete Testing)

(2) Testing of concrete materials

As concrete for port structures, usually ready-mixed concrete that conforms to **JIS A 5308 Ready-mixed concrete** is used. When concrete is prepared at actual sites, it is desirable to use raw materials that conform to JIS and JSCE standards and other quality standards. For raw materials that do not conform to the quality standards, whether the raw materials are appropriate can be determined through testing of the concrete materials or based on past results. For the procedures for testing concrete materials, refer to **Table 5.3.2**.

Table 5.3.2 Concrete Material Test Standards and Test Procedures

Cement	JIS R 5201	Physical testing methods for cement
	JIS R 5202	Methods for chemical analysis of cements
	JIS R 5203	Determination of the heat of hydration of cement-Solution method
	JIS R 5204	Chemical analysis method of cement by X-ray fluorescence
Water Aggregate Cement	JSCE-B 101	JSCE Standard Qualities of Water for Concrete
	JIS A 1102	Method of test for sieve analysis of aggregate
	JIS R 5201	Physical testing methods for cement

	JIS R 5202	Methods for chemical analysis of cements
	JIS A 1103	Method of test for amount of material passing test sieve 75 μm in aggregates
	JIS A 1104	Methods of test for bulk density of aggregates and solid content in aggregates
	JIS A 1105	Method of test for organic impurities in fine aggregate
	JIS A 1109	Methods of test for density and water absorption of fine aggregates
	JIS A 1110	Methods of test for density and water absorption of coarse aggregates
	JIS A 1111	Method of test for surface moisture in fine aggregate
	JIS A 1121	Method of test for resistance to abrasion of coarse aggregate by use of the Los Angeles machine
	JIS A 1122	Method of test for soundness of aggregates by use of sodium sulfate
	JIS A 1125	Methods of test for moisture content of aggregate and surface moisture in aggregate by drying
	JIS A 1126	Method of test for content of soft particles in coarse aggregate by scratching, abolished on Dec. 7, 2015
	JIS A 1134	Methods of test for particle density and water absorption of lightweight fine aggregates for structural concrete
	JIS A 1135	Methods of test for particle density and water absorption of lightweight coarse aggregates for structural concrete
	JIS A 1137	Method of test for clay lumps contained in aggregates
	JIS A 1141	Method of test for particles less than density of 1.95 g/cm^3 in aggregate, abolished on Dec. 7, 2015
	JIS A 1142	Method of test for fine aggregate containing organic impurities by compressive strength of mortar
	JIS A 1145	Method of test for alkali-silica reactivity of aggregates by chemical method
	JIS A 1146	Method of test for alkali-silica reactivity of aggregates by mortar-bar method
	JIS A 1801	Methods of test for production control of concrete-Method of test for sand equivalent value of fine aggregates for concrete
	JIS A 1802	Methods of test for production control of concrete-Method of test for surface moisture in fine aggregate by centrifugal force
	JIS A 1108	Method of test for compressive strength of concrete
	JIS A 1804	Methods of test for production control of concrete-Method of rapid test for identification of alkali-silica reactivity of aggregate
	JIS K 0058 - 1	Test methods for chemicals in slags−Part 1: Leaching test method
	JIS K 0058 - 2	Test methods for chemicals in slags−Part 2: Test method for acid extractable contents of chemicals
	JIS A 1158	Method for reducing samples of aggregate to testing size
	JSCE-C 502	JSCE Standard: Test method for chloride ion content in sea sand (Titration method)
	JSCE-C 503	JSCE Standard: Test method for chloride ion content in sea sand (Simple measuring instrument method)
	JSCE-C 504	JSCE Standard: Test method for content ratio of blast-furnace slag fine aggregate in fine aggregate mixed blast-furnace slag
	JSCE-C 505	JSCE Standard: Test method for crushing load of high strength lightweight aggregate made of fly ash
	JSCE-C 506	JSCE Standard: Test method for density and water absorption of slag fine aggregate for concrete by measurement of electric resistance
	JSCE-C 511	JSCE Standard: Test method for evaluation of alkali silica reactivity of aggregates (Modified chemical method)
Admixture	JSCE-D 501	JSCE Standard: Test method for content and replacement ratio of ground granulated blast-furnace slag in concrete
	JSCE-D 503	JSCE Standard: Test method for replacement ratio of fly ash in concrete

(3) Testing of fresh concrete

Concrete from immediately after mixing to setting (curing) after being placed in a mold is called fresh concrete. To construct concrete properly and make concrete structures with fewer defects, the concrete should have workability appropriate for the construction work. To check consistency (degrees of softness and liquidity), plasticity (character where concrete is ease to place to a mold, and once the mold is removed, the shape of concrete changes slowly but does not collapse and the raw materials do not separate), pumpability (ease of pumping), finishability (ease of finishing work), and other properties, refer to **Table 5.3.3**.

Table 5.3.3 Test Items and Procedures for Testing Fresh Concrete

Test item	Test procedure
Consistency	JIS A 1101 Method of test for slump of concrete
Air content	JIS A 1128 Method of test for air content of fresh concrete by pressure method JIS A 1118 Method of test for air content of fresh concrete by volumetric method Note: Can be applied to porous aggregates (e.g., artificial lightweight aggregates).
Content of chlorides	JIS A 1144 Method of test for chloride concentration in water of fresh concrete
Bleeding test	JIS A 1123 Method of test for bleeding of concrete
Setting test	JIS A 1147 Method of test for time of setting of concrete mixtures by penetration resistance
Pressurized bleeding test	JSCE-F 502 Test method for bleeding of concrete under pressure

(4) Testing of hardened concrete

This type of test should be performed to design the mix proportions of the concrete and manage the quality of the hardened concrete. **Table 5.3.4** may be referred to.

Table 5.3.4 Test Items and Procedures for Testing Hardened Concrete

Test item	Test procedure
Compressive strength test	JIS A 1108 Method of test for compressive strength of concrete
Static elasticity modulus test	JIS A 1149 Method of test for static modulus of elasticity of concrete
Tensile strength test	JIS A 1113 Method of test for splitting tensile strength of concrete
Length change test	JIS A 1129-1 Methods of measurement for length change of mortar and concrete−Part 1: Method with comparator JIS A 1129-2 Methods of measurement for length change of mortar and concrete−Part 2: Method with contact-type strain gauge JIS A 1129-3 Methods of measurement for length change of mortar and concrete−Part 3: Method with dial gauge
Creep test	JIS A 1157 Method of test for compressive creep of concrete

(5) Durability testing

When it is expected that the concrete will deteriorate due to frost damage, chloride-induced corrosion or other factors, this type of test should be performed to select the raw materials of the concrete and design the mix proportions targeting hardened concrete. **Table 5.3.5** may be referred to.

Table 5.3.5 Test Items and Procedures for Testing the Durability of Concrete

Test item	Test procedure
Frost damage	JIS A 1148 Method of test for resistance of concrete to freezing and thawing JIS A 1127 Methods of test for dynamic modulus of elasticity, rigidity and Poisson's ratio of concrete by resonance vibration
Alkali-silica reaction	ZKT-206:2007 Rapid test procedure for alkali-silica reaction on concrete JCI-S-010-2017 Method of test for alkali-silica reactivity of concrete
Neutralization	JIS A 1153 Method of accelerated carbonation test for concrete
Chloride-induced corrosion	JSCE-G 571 Test method for effective diffusion coefficient of chloride ion in concrete by migration JSCE-G 572 Test method for apparent diffusion coefficient of chloride ion in concrete by submergence in salt water

(6) Testing of concrete structures

This type of test is performed to check the quality of the constructed structures. **Table 5.3.6** may be referred to.

Table 5.3.6 Test Items and Procedures for Testing Concrete Structures (Non-Destructive Inspection)

Test item	Test procedure
Resilience and quality of concrete	JIS A 1155 Method of measurement for rebound number on surface of concrete JSCE-G 504 Test method for concrete strength by test hammer NDIS 2426-1 Non-destructive testing of concrete - elastic wave method, Part 1 Ultrasonic method NDIS 2426-2 Non-destructive testing of concrete - elastic wave method, Part 2 Impact elastic wave method NDIS 2426-3 Non-destructive testing of concrete - elastic wave method, Part 3 Impact acoustics method
Arrangement of reinforcement	NDIS 3429 Method for investigating location of reinforcing bars in concrete structure by radar NDIS 3430 Investigation for locating rebars in concrete structure by electromagnetic method
Appearance	NDIS 3418 Method of visual test for concrete structures

(7) Testing of special concrete

Table 5.3.7 lists the types of tests. For the specific test procedures for anti-washout underwater concrete, refer to the **Anti-Washout Concrete Design and Construction Manual**.[11] For the specific test procedures for high-fluidity concrete, refer to the **High-Fluidity Concrete Mix Proportion Design and Construction Manual**.[12]

Table 5.3.7 Test Items and Procedures for Testing Special Concrete

Type	Test item	Test procedure, etc.
Anti-washout underwater concrete	Consistency	JIS A 1150 Method of test for slump flow of concrete
	Compressive strength	JSCE-F 504 Method of making compressive strength specimens of anti-washout concrete cast in water
	Admixture mineral	JSCE-D 104 Specification for anti-washout admixture for concrete
High-fluidity concrete	Consistency	JIS A 1150 Method of test for slump flow of concrete
	Liquidity	JSCE-F 514 Test method for L-type flow of self-compacting concrete
	Repletion	JSCE-F 511 Test method for passability through obstacle of self-compacting concrete JSCE-F 512 Test method for flowability of self-compacting concrete (Funnel method)

(8) For the tests types required for other types of special concrete, **Table 5.3.8** may be referred to. When other types of concrete, such as short-fiber reinforced concrete, spray concrete for repair and reinforcement, concrete for flexible repair and chemically prestressed concrete are used, the traceability of the test results of the fillers and other materials should be secured.

Table 5.3.8 Test Items and Procedures for Testing Other Types of Special Concrete

Type	Test item	Test procedures, etc.
Short fiber reinforced	Toughness (bending strength test and bending toughness test)	JSCE-G 552 Test method for bending strength and bending toughness of steel fiber reinforced concrete JCI-SF 4 Procedure for testing the bending strength and toughness of fiber reinforced concrete
Mass concrete	Adiabatic temperature rise characteristic	Parameters in temperature stress analysis to be measured in an adiabatic temperature rise test (equipment)

Type		Test item	Test procedures, etc.
Pavement concrete		Bending strength test	JIS A 1106 Method of test for flexural strength of concrete
		Consistency	JSCE-F 501 Test method for consistency of concrete for pavement using vibrating table
Spray concrete	For tunnels	Compressive strength test	JSCE-F 561 Method of making specimens for compressive strength of sprayed concrete (mortar) JIS A 1107 Method of sampling and testing for compressive strength of drilled cores of concrete
		Initial strength test	JSCE-G 561 Test method for early strength of sprayed concrete by pull-out method JSCE-G 562 Test method for early strength of sprayed concrete using prism specimens
	For slope faces	Compressive strength test	JSCE-F 561 Method of making specimens for compressive strength of sprayed concrete (mortar) JIS A 1107 Method of sampling and testing for compressive strength of drilled cores of concrete
Prepacked concrete		Mortar	JSCE-F 521 Test method for flowability of grout mortar for prepacked concrete (P-type funnel method) JSCE-F 522 Test method for bleeding ratio and expansion ratio of grout mortar for prepacked concrete (Polyethylene bag method)
Expansive concrete		Expansion Compressive strength test	JIS A 6202 Expansive additive for concrete Annex A (Regulations) Procedure for testing the expansion of expanding materials by mortar Annex B (Reference) Procedure for testing restrained expansion and contraction of expansive concrete Annex C (Reference) Procedure for testing compressive strength of expansive concrete by restrained curing

5.4 Asphaltic Materials

5.4.1 General

This section organizes the main test procedures for asphalt mats, sand mastics and paving materials as part of the asphaltic materials used for port construction work. Asphalt is a viscoelastic material with excellent deformability and flexibility, and its deformation characteristics depend on the temperature and loading rate. However, if the design and construction are not appropriate, it may break (e.g., embrittlement due to aging), excessively deform, or have other problems.

Asphalt mixtures should be designed based on the appropriate material test procedures by considering the purpose of use, installation locations, weather conditions and oceanic phenomena at actual sites, construction methods and other factors.

Table 5.4.1 organizes the objectives of each test and the criteria regarding the test results for the main test procedures for asphalt mats, sand mastics and pavement materials.

Table 5.4.1 Objectives of the Testing of Asphalt Mixtures and Acceptance Criteria

Testing of asphalt mixture		Objective of test	criteria
Testing of asphalt mats		Notes on design • An asphalt mat should be designed such that the specific gravity, strength, deflection and other requirements should be appropriate by considering the purpose of use, construction method and other factors.	
	Specific gravity test	• To be performed to determine the mix design with the appropriate specific gravity	• Specific gravity 2.2 or more
	Bending test	• To be performed to measure the bending strength and deflection to use the data when determining the mix design and managing the production	• Bending strength Friction enhancement mat: 2.0 N/mm^2 or more Scouring and sand washing-out prevention mat: 1.0 N/mm^2 or more • Deflection 3 mm or more
	Compression test	• To be performed to measure the compressive strength to use the data when determining the mix design and managing the production	• Compressive strength Friction enhancement mat: 2.0 N/mm^2 or more Scouring and sand washing-out prevention mat: 1.0 N/mm^2 or more
	Push-off test	• To be performed to measure the maximum push-out load and displacement volume to use the data when determining the mix design and managing the production. A push-off test should be performed only for scouring and sand washing-out prevention mats.	• Maximum load Normal mat: 8 kN or more Reinforced mat: 15 kN or more • Displacement volume Normal mat: 10 mm or more Reinforced mat: 30 mm or more
Testing of sand mastics (for waste rock consolidation)		Notes on design • Sand mastics should be designed such that the specific gravity, strength, deflection and fluidity should be appropriate considering the purpose of use, construction method and other factors.	
	Specific gravity test	• To be performed to determine the mix design with the appropriate specific gravity	• Specific gravity 1. 95 or more
	Bending test	• To be performed to measure the bending strength and deflection to use the data when determining the mix design and managing the production	• Bending strength 1. 0 N/mm^2 or more • Deflection 5.0 mm or more
	Compression test	• To be performed to measure the compressive strength to use the data when determining the mix design and managing the production	• Compressive strength 1. 0 N/mm^2 or more
	Liquidity test	• To be performed to determine the mix design with the appropriate fluidity • To be performed to study changes in the fluidity due to variations in the mix design	• Flow - down time 10 to 60 sec • There should be no significant separation of the raw materials in a visual inspection.
Testing of paving asphalt mixture		Notes on design • For the asphalt paving of surfaces and base courses, straight asphalt and modified asphalt are used based on the application purpose. As an asphalt stabilizer, usually, a material that was made by heating and mixing straight asphalt is used.	
	Marshall stability test	• To be performed to design the appropriate mix proportions for asphalt paving and asphalt stabilizers, and to manage construction	• To be used mainly to check the Marshall stability and determine the design asphalt volume based on the application purpose.

5.4.2 Test Procedures for Asphalt Mats and Sand Mastics

(1) Test

Test to determine the mix design for asphalt mats and sand mastics consist of specific gravity, bending, compression, push-off and fluidity tests.

Basically, a same-test specimen should be used for specific gravity, bending and compression tests in a series of measurements. **Table 5.4.2** lists the test items and conditions.

Table 5.4.2 Test Items and Conditions for Asphalt Mats and Sand Mastics

Test item		Asphalt mat			Sand mastic (for waste rock consolidation)
		Friction enhancement mat	Scouring and sand washing-out prevention mat		
			Normal mat	Reinforced mat	
Specific gravity test	Specimen size	40 × 40 × 160 mm			
	Test temperature	20 ± 1°C			10 ± 1°C
Bending test	Specimen size	40 × 40 × 160 mm			
	Test temperature	20 ± 1°C			10 ± 1°C
	Loading rate	20 mm/min			
Compression test	Specimen size	40 × 40 × 80 mm (half piece after the bending test*1)			
	Test temperature	20 ± 1°C			10 ± 1°C
	Loading rate	20 mm/min			
Push-off test	Specimen size	-	50 × 300 × 300 mm		-
	Test temperature		20 ± 1°C		
	Loading rate		50 mm/min		
Fluidity test	Test temperature	-	-		180°C

*1: For a friction enhancement mat specimen, when Class 5 broken stones (grain sizes of 13 to 20 mm) pile up in the compressional axis direction, an excessive load is measured. In this case, the test results should be regarded as unacceptable and the other half pieces should be tested.

(2) Specific gravity test

① Test instruments

The required test instruments are a mold for forming mortar specimens (**JIS R 5201, Fig. 5.4.1**), a scale (capacity: 1 kg or more; reciprocal sensitivity: 0.1 g), a net cage (large enough to accommodate the specimens easily with a mesh to prevent the specimens from falling when taking measurements), a water tank (large enough so that the net cage can be put into water), and a thermometer.

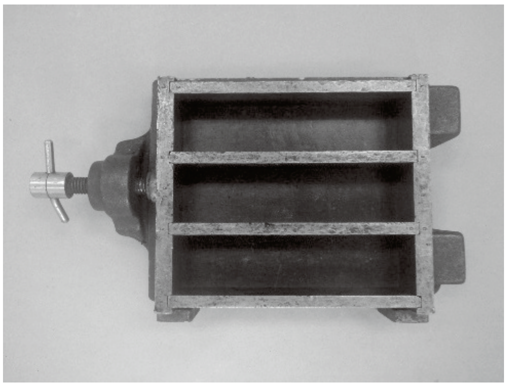

Fig. 5.4.1 Mold for Forming Mortar Specimens

② Test procedures

Use the mold for forming mortar specimens to produce three specimens (40 × 40 × 160 mm). Apply a silicone release agent or other type of agent to the inside of the mold to make it easy to remove the specimens, then pour a heated and mixed asphalt mixture into the mold. The mold is divided into three sections; pour the asphalt up to the surface of each section all at one time. Pile up the asphalt mixture slightly higher than the surface of the mold and let it cool to some extent. Next, use a metal spatula or other similar tool to shave off some of the asphalt and smooth it out. Remove the specimens from the mold and immediately put them into the water, which has been set to the test temperature. Cure them for approximately one hour. The water temperature should be 20 ± 1°C for an asphalt mat and 10 ± 1°C for sand mastic.

Measure the weight in air and weight in water for each specimen to a unit of 0.1 g to measure the specific gravity. Use the **equation (5.4.1)** to calculate the specific gravity and average down to the second decimal place for each specimen. When measuring the weight in air, use a dry cloth or other similar item to remove moisture from the surface after taking out the specimens which have been cured in water.

$$\text{Specific gravity} = \frac{\text{Weight in air}}{\text{Weight in air - weight in water}} = \frac{\text{Weight in air}}{\text{Water density (1)} \times \text{volume}} \qquad \textbf{(5.4.1)}$$

(3) Bending test

① **Test instruments**

A universal testing instrument, a mold for forming mortar specimens, an attachment for bending tests and a thermometer are required.

For the attachment for bending tests, a bending strength test instrument conforming to **JIS R 5201** should be used (**Fig. 5.4.2**). The interval (100 mm) between the supports in the bending test and other conditions should be the same as those for the mortar test. Use supporters that are tall enough so that the attachment will not come into contact with the bottom part after the specimen deforms. A universal testing instrument with a displacement control function should be used.

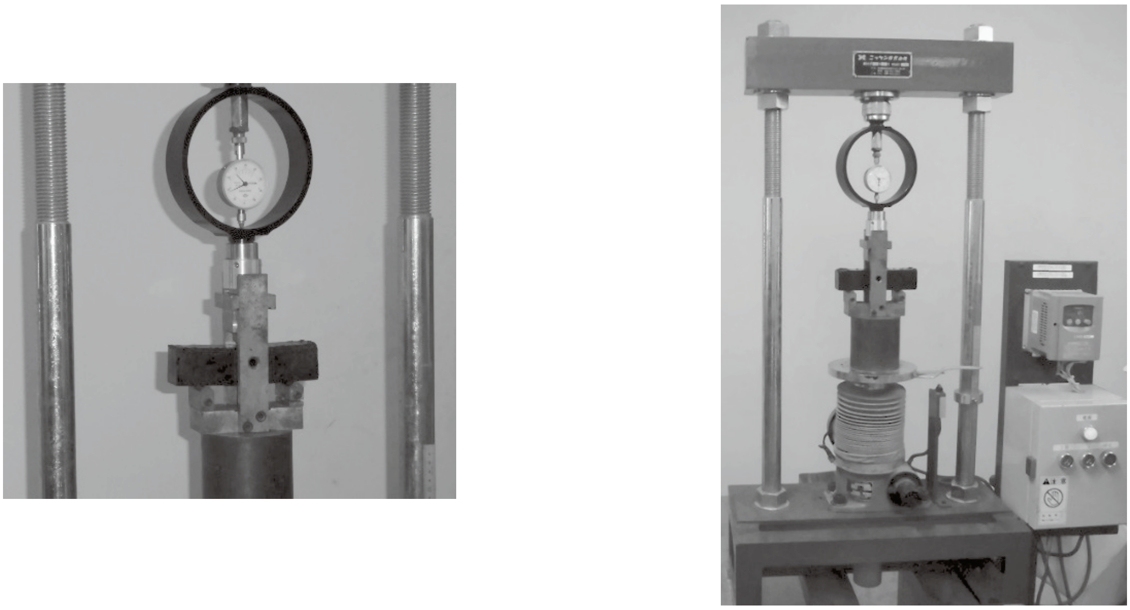

Fig. 5.4.2 Example Bending Test

② **Test procedures**

Use the specimens used for the specific gravity test (**Fig. 5.4.3**) and perform the test in accordance with **JIS R 5201**. For the test temperature and loading rate, see **Table 5.4.2**.

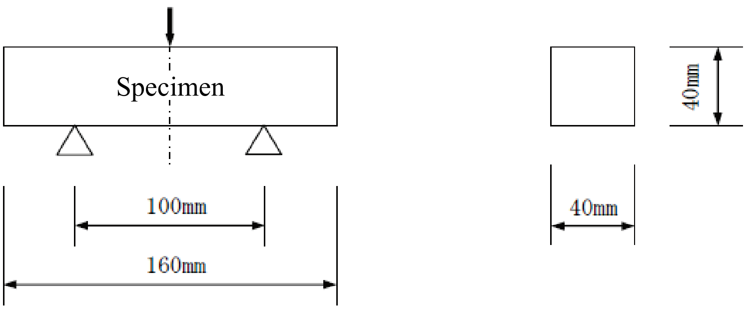

Fig. 5.4.3 Outline of Bending Test (JIS R 5201)

(4) Compression test

① **Test instruments**

A universal testing instrument, a mold for forming mortar specimens, an attachment for compression tests and a thermometer are required. For the attachment for compression tests, a compressive strength test instrument conforming to **JIS R 5201** should be used. The area to be pressurized (40 × 40 mm) is the same as that in the mortar test. A universal testing instrument with a displacement control function should be used.

② **Test procedures**

Use the three half pieces of the specimens used in the bending test and perform the test in accordance with **JIS R 5201**. For the test temperature and loading rate, see **Table 5.4.2**.

For a friction enhancement mat specimen, when Class 5 broken stones (grain sizes of 13 to 20 mm) pile up in the compressional axis direction, as shown in **Fig. 5.4.4**, an excessive load is measured. In this case, the test results should be regarded as unacceptable and the other half pieces should be tested.

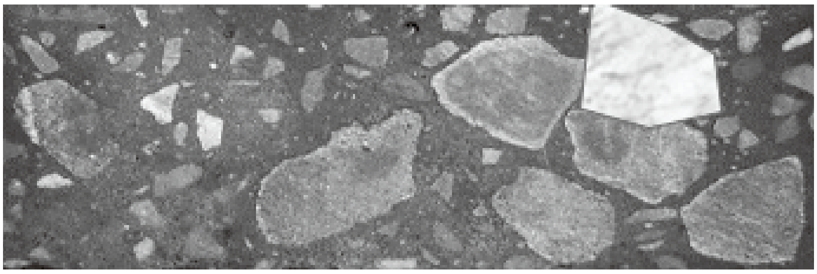

Fig. 5.4.4 Example for When Broken Stones Pile Up

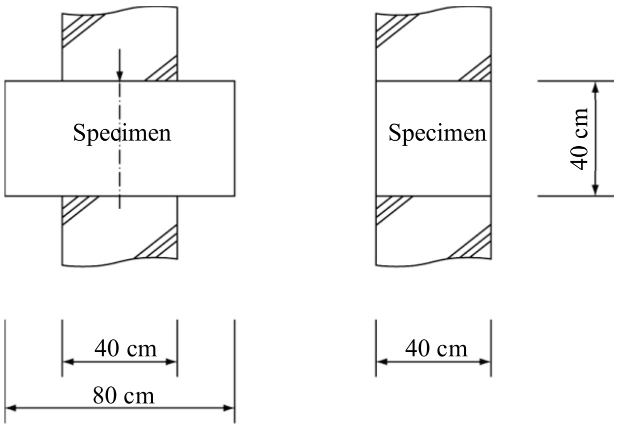

Fig. 5.4.5 Outline of Compression Test (JIS R 5201)

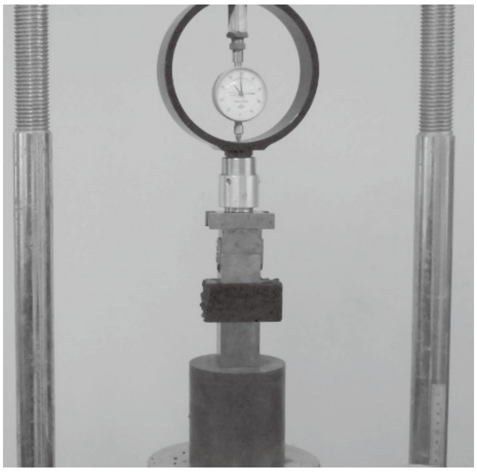

Fig. 5.4.6 Example Compression Test

(5) Push-off test

① **Test instruments**

A universal testing instrument, a push-off tester (**Fig. 5.4.7**), a steel mold for specimens (**Fig. 5.4.8**) and a thermometer are required.

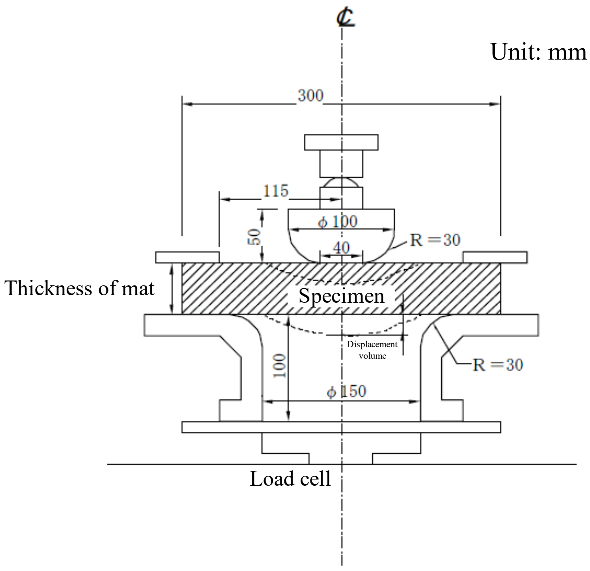

Fig. 5.4.7 Example Push-Off Tester

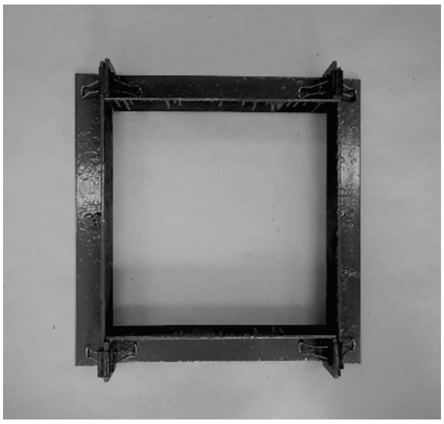

Fig. 5.4.8 Steel Mold

② **Test procedures**

Use the steel mold (50 × 300 × 300 mm) to produce three specimens. Pour a heated and mixed asphalt mixture to the middle of the steel mold from the bottom (mold height: 50 mm). Next, smooth out the surface and install a reinforcement, then pour the heated and mixed asphalt mixture to the upper end of the steel mold. Let it cool to some extent, smooth out the surface for forming, and let it cool again. Remove the specimens from the mold and immediately put them into the water, which has been set to the test temperature (20 ± 1°C). Cure them for approximately two hours.

Place one specimen on the attachment (a funnel-shaped disk plate with an inner diameter of 150 mm, R of 30 mm and height of 100 mm). Use the special fitting jig and bolt of the doughnut-shaped disk to secure the specimen (**Fig. 5.4.9**). Use a loading head with an outer diameter of 100 mm and R of 30 mm to perform a test at 20°C and loading rate of 50 mm/min. Read the maximum load (kN) for each of the three specimens to the first decimal place. Measure the displacement volume (mm) at the maximum load as a whole number and calculate the averages (**Fig. 5.4.10**).

The thickness of the specimens used for the push-off test should be 50 mm regardless of the thickness of the mat to be constructed.

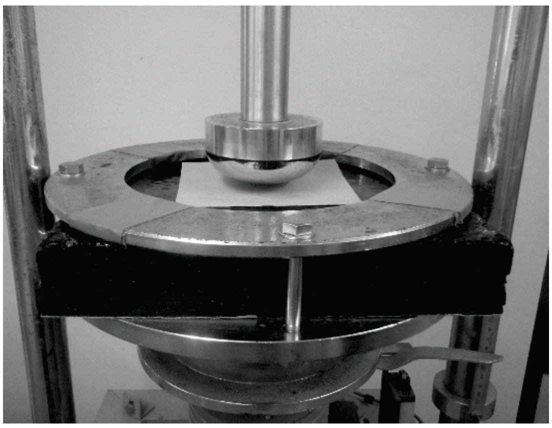

Fig. 5.4.9 Secured Specimen

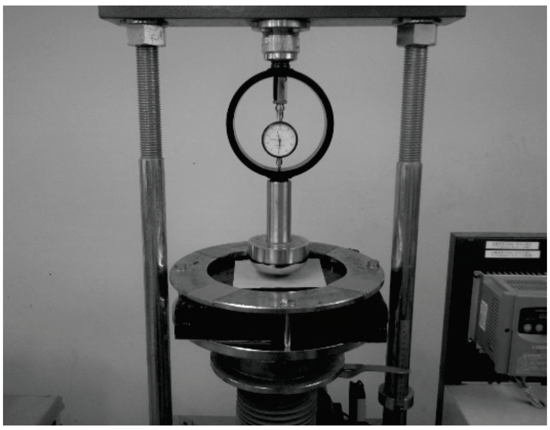

Fig. 5.4.10 Example Push-Off Test

(6) Fluidity test

① **Test instruments**

A funnel for fluidity tests (for the shape and dimensions, see **Fig. 5.4.11**) and a thermometer are required. A stopwatch should be used to measure the time.

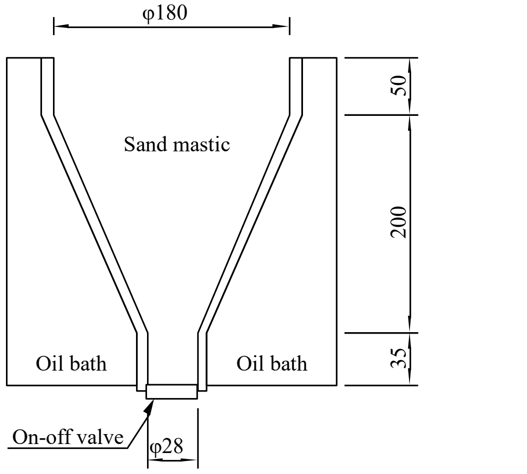

Fig. 5.4.11 Shape and Dimensions of a Funnel for Fluidity Tests

Note: In place of an oil bath, another type of instrument that can maintain the temperature of the sand mastic at 160 to 180°C can be used.

② **Test procedures**

Set the temperature of the oil bath, etc., outside the funnel to 160 to 180°C. Mix and stir the asphalt, filler and sand sufficiently. Heat the sand mastic to 190 to 200°C and stop the heating, then let it cool to 180°C while stirring. Maintain the temperature.

Apply a silicone release agent or other type of agent to the inside of the funnel. Pour the sand mastic to 50 mm from the upper end of the funnel and open the on-off valve under the funnel. Use the stopwatch to measure the time until the continuous flow of the sand mastic from the discharge port first breaks (**Fig. 5.4.12**).

Fig. 5.4.12 Sand Mastic Flowing Down

5.4.3 Test Procedures for Asphaltic Pavement Materials

For the standards and tests for paving asphaltic materials regarding **Part III Facilities, Chapter 5, 9.18 Aprons** and **Part III Facilities, Chapter 6, 2.7 Performance Verification of Pavement**, refer to the **Standard Specifications for Port & Harbor Works**,[13] the **Standards for Quality Control Testing** and **Acceptance Criteria for Port Construction**,[14] the **Guideline for Pavement Design and Construction**[15] and the **Pavement Construction Handbook**.[16] When recycled materials are used, refer to the **Handbook for Recycling of asphalt pavement**.[17]

For details of the main test procedures, refer to the **Pavement Investigation and Test method Handbook.**[18]

5.5 Stones

5.5.1 Stone Testing

Stone materials for port construction work are used mainly as rubble mounds, armor stones, foot protection stones, backfilling stones, base course materials and aggregates for concrete. The quality and other requirements demanded for each material type are listed below.

① **Rubble mounds, armor stones and foot protection stones**: The shape should not be flat and elongated. They should be hard and fine, have high durability, and have no risks of weathering or frost damage.

② **Backfilling stones**: They should be stones where a sufficient angle of internal friction can be expected.

③ **Base course materials**: The required bearing power should be obtainable, foot protection should be easy, and the materials should have high durability.

④ **Aggregates for concrete**: They should be strong and have high durability, the grain sizes should be appropriate, and they should not contain thin stone pieces.

(1) Testing Objectives

The necessary inspections and tests should be performed to ensure the quality of the stones. This section describes the necessary inspection and test procedures targeting stones when they are used as rubble mounds, armor stones, foot protection stones and backfilling stones. Not all the items in **Tables 5.5.1** and **5.5.2** need to be tested. The test items should be determined by sufficiently considering the conditions at the actual sites, the application purposes and other factors.

Table 5.5.1 Classification of Inspection Items[19]

Test	Test item	Inspection target	Test frequency
Geological survey	–	Understanding of general conditions	○
Material quality judgment test	Apparent specific gravity and water absorption rate test	Apparent specific gravity and water absorption rate	◎
	Grain size distribution measurement	Grain size distribution	○
	Dissolution test of heavy metals and other substances	Degree of dissolution of heavy metals and other substances	○
Strength test	Unconfined compression test	Unconfined compression strength	◎
	Radial compression and tension test	Radial compressive and tensile strength	○
	Ultrasonic nondestructive strength test	P-wave propagation velocity	○
Durability test	Stability test	Weather resistance	○
	Abrasion resistant test	Abrasion resistance	○

Notes: ◎: To be performed in many cases
○: Omitted in some cases

Table 5.5.2 Stone Testing Objectives and Test Results[19)]

Stone test system		Test objective	Test result
Geological reconnaissance		• A geological reconnaissance should be performed to understand the ground configuration, geographical features and bedrock at the quarrying site and around it in advance to make the quarrying reasonable and economical.	• Prepare a ground plan and sectional view of the geographical features of the quarrying site and around it to understand an overview of the bedrock. Use these as basic data to plan reasonable and economical quarrying.
Material quality judgment test	Apparent specific gravity and water absorption rate test	• To be performed to make a rough judgment of the physical properties of the stones.	• Check if the specific gravity satisfies the requirements.
	Grain size distribution test	• Grain size distribution should be measured to understand if the grain size and oblateness satisfy the requirements as materials for rubble mounds, backfilling stones and other stones.	• Use the grain size distribution as basic data to understand the properties of the stones. • Stones with large oblateness are not desirable to use as stone materials.
	Dissolution test of heavy metals and other substances	• Dissolution of heavy metals and other substances contained in stones should be tested to understand to what extent they will flow out to seawater in order to prevent marine pollution.	• Stones that do not satisfy the criteria for dissolution tests of heavy metals and other substances must not be used.
Strength test	Unconfined compression test	• To be performed to understand the compressive strength of the stones.	• Classify the rocks into soft rock, semi-hard rock, hard rock and other types. Use them as basic data for consideration based on the purpose of use.
	Radial compression and tension test	• To be performed to understand the tensile strength of the stones.	• Estimate the strength of the bedrock at the actual site from the radial compressive and tensile strength values. Use it as basic data to dig the bedrock or for other purposes.
	Ultrasonic nondestructive strength test	• By measuring the propagation velocity of ultrasonic waves through rocks, the unconfined compression strength, radial compressive and tensile strength and other physical properties can be indirectly understood without breaking the rocks.	• Conform to the unconfined compression tests and radial compression and tension tests. • Although the accuracy of nondestructive strength tests is lower than that of the unconfined compression tests and radial compression and tension tests, the test procedures are simple, so perform this type of test for a preliminary inspection or when an unconfined compression test or a radial compression and tension test are impossible.
Durability test	Stability test	• To be performed to judge the stability of the stones to weathering (weather resistance).	• Stones with a large loss mass percentage are not desirable to use as stone materials. • The Standard Concrete Specifications[20)] specify that the loss mass percentage should be 10% or less for fine aggregates and 12% or less for coarse aggregates.
	Abrasion resistant test	• To be performed to understand the wear resistance of the stones.	• Stones for which the amount of abrasion loss is large are not desirable to use as stone materials. • The Standard Paving Specifications[21)] specify that the amount of abrasion loss on coarse aggregates should be 35% or less for paving concrete.

1) Among sandstones, the specific gravity and compressive strength of those produced from areas other than Fukuoka Prefecture are in a saturated surface-dry condition.
2) The *SPT-N value* of hardpan is 50 or more.

(2) Stone Types

Part II, Chapter 11, 5 Stones, Table 5.1.1 shows the main stone types related to port construction work along with their physical properties. However, attention should be paid to the fact that the physical properties largely vary depending on the material producing area and quarrying location even for the same type of rock. **Table 5.5.3** and **Figure 5.5.1** show examples of differences between the producing areas.

Table 5.5.3 Physical Properties of Main Stone Types[19)]

Type	Stone name		Specific gravity		Water absorp-tion rate (%)	Porosity (%)	Strength (kgf/cm^2)		
	Popular name	Producing area	True	Apparent			Com-pressive	Tensile	Bending
Granite	Inada	Ibaraki	2.64	2.63	0.34	1.01	1,452	47	123
	Sanshu	Aichi	2.69	–	0.22	–	1,670	83	116
	Hon Mikage	Hyogo	2.65	–	0.43	0.98	1,570	58	163
	Mannari	Okayama	2.63	2.62	0.43	1.02	1,868	53	122
	Kitagi	Okayama	2.64	2.61	0.10	1.22	1,678	44	145
Andesite	Shirochoba	Kanagawa	2.57	2.30	2.44	10.91	1,034	56	78
	Shin-komatsu	Kanagawa	2.56	–	1.83	3.82	1,165	36	86
	Yokonezawa-ishi	Shizuoka	2.47	–	3.09	6.10	910	70	94
	Tsukide-ishi	Shizuoka	2.50	–	2.99	–	1,038	61	121
Tuff	Oya-ishi	Tochigi	1.99	1.40	19.04	39.46	87	8	23
	Boshu-ishi	Chiba	2.70	1.47	30.93	45.54	61	13	–
	–	Fukuoka	–	2.75	0.23	–	1,118	–	–
Sandstone	Myoutai	Chiba	–	1.74	3.7	–	76	–	–
	Choshi-ishi	Chiba	2.65	2.34	5.10	11.90	568	28	–
	Sawara	Chiba	–	2.10	1.78	–	116	–	–
	Tatebo-ishi	Shizuoka	2.48	–	13.22	–	365	26	–
	–	Fukuoka	–	2.76	0.07	–	1,045	–	–
Serpentinite	–	Saitama	2.76	–	0.37	–	978	59	–
Limestone	Motobu limestone	Okinawa	–	2.68	0.81	0.55	701	–	–
	Ryukyu limestone	Okinawa	–	2.42	2.59	1.24	297	–	–
Hardpan	–	Keihin port, Honmoku	–	1.8	6.5	–	5.7 to 10.7	Shear 2.2 to 4.5	–

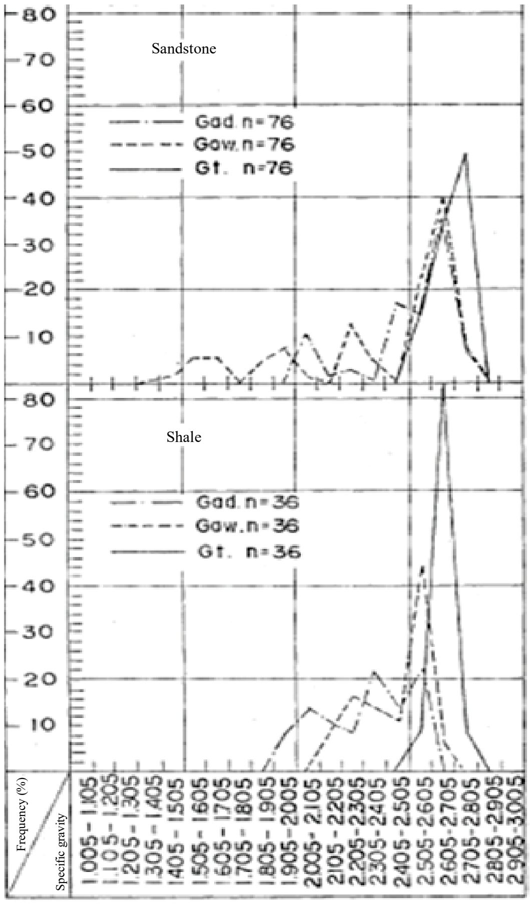

Fig. 5.5.1 Example of the Specific Gravity Distribution of Rocks[21)]

(3) Systematic Organization of Test Procedures

The stone test procedures are systematically organized and shown below. When stones are used as base course materials for roads, refer to the **Paving Design and Construction Guidelines**.[15)] When stones are used as aggregates for concrete, refer to the **Standard Concrete Specifications**.[20)]

① **Material quality tests for stones**

In a material quality test for stones, the apparent specific gravity, water absorption rate, grain size distribution and dissolution of heavy metals and other substances should be tested.

(a) Apparent density and water absorption rate tests

The tests should mainly follow JIS A 1110 Methods of test for density and water absorption of coarse aggregates, while JIS A 5003 Stones and JIS Z 8807 Methods of measuring density and specific gravity of solid should be used as references.

Check if the specific gravity satisfies the requirements as a result of the test. When the names of rocks and the producing areas are known, whether they are acceptable can be understood by comparing them to the standard specific gravity and water absorption rate.

Generally, rocks with a low apparent specific gravity and high water absorption rate are soft rocks, and rocks with a high apparent specific gravity and low water absorption rate are hard rocks. For the specific gravity, in addition to the apparent specific gravity, there is bulk specific gravity and true specific gravity. Although the true specific gravity is greater than the other two types, the differences can be ignored for rocks with few voids. The apparent specific gravity value of common rocks is 2.0 to 3.0. The values of igneous and metamorphic rocks are usually larger than those of sedimentary rocks.

(b) Grain size distribution tests

The grain size distribution should be measured to understand if the rocks satisfy the required grain size and oblateness as materials for rubble mounds, backfilling stones and other stones. No procedures for measuring the grain size distribution of stones have been established so far. Tests should follow **JIS A 1204 Test method for particle size distribution of soils** and **JIS A 1102 Method of test for sieve analysis of aggregates**.

In a grain size distribution test, a cloth measure should be used to measure the grain size in place of a sieve, and a platform scale should be used to measure the mass. Rocks with large oblateness are not desirable to use as stone materials.

The data on grain size distribution and oblateness can be organized using the format in **Table 5.5.4** as a reference.

Table 5.5.4 Data List[21)]

Mass classifi-cation	Number of samples	Mass	Ratio	Major axis			Minor axis			Average oblateness	Average grain size
				Max.	Min.	Ave.	Max.	Min.	Ave.		

(c) Dissolution tests of heavy metals and other substances

The dissolution of heavy metals and other substances contained in stones should be tested to know to what extent they will flow out to seawater in order to prevent marine pollution (**Table 5.5.5**). Generally, when normal stones are used, this type of test is not required in most cases. When there is a concern that heavy metals and other substances may dissolve into the seawater, the necessary tests should be performed.

For the test items and test procedures, refer to the environmental standards for water pollution based on the provisions in Article 9 of **Public Notice No. 59 by Japan's Ministry of the Environment**, **the Basic Act on Pollution Control** (Law No. 132 of 1967).

Table 5.5.5 Procedures for Testing the Dissolution of Heavy Metals and Other Substances

Item	Criterion	Measurement procedures
Cadmium	0. 003 mg/L or less	Procedures specified in 55.2, 55.3 or 55.4 in JIS K 0102
Total cyanide	Not detected	Procedures specified in 38.1.2 and 38.2 in JIS K 0102, those in 38.1.2 and 38.3 in JIS K 0102, or those in 38.1.2 and 38.5 in JIS K 0102
Lead	0.01 mg / L or less	Procedures specified in 54 in JIS K 0102
Hexavalent chromium	0.05 mg/L or less	Procedures specified in 65.2 in JIS K 0102 (when brackish water or seawater is measured in accordance with the procedures in 65.2.6 in JIS K 0102, the operations in 7 a) or b) in JIS K 0170-7 should be carried out.)
Arsenic	0.01 mg/L or less	Procedures specified in 61.2, 61.3 or 61.4 in JIS K 0102
Total mercury	0.0005 mg/ L or less	Procedures specified in Appendix 1
Alkylmercury	Not detected	Procedures specified in Appendix 2
PCB	Not detected	Procedures specified in Appendix 3
Dichloromethane	0.02 mg/L or less	Procedures specified in 5.1, 5.2 or 5.3.2 in JIS K 0125
Carbon tetrachloride	0.002 mg/L or less	Procedures specified in 5.1, 5.2, 5.3.1, 5.4.1 or 5.5 in JIS K 0125
1,2-dichloroethane	0.004 mg/L or less	Procedures specified in 5.1, 5.2, 5.3.1 or 5.3.2 in JIS K 0125
1,1-dichloroethylene	0.1 mg/L or less	Procedures specified in 5.1, 5.2 or 5.3.2 in JIS K 0125
Cis-1,2-dichloroethylene	0.04 mg/L or less	Procedures specified in 5.1, 5.2 or 5.3.2 in JIS K 0125
1,1,1-trichloroethane	1 mg/L or less	Procedures specified in 5.1, 5.2, 5.3.1, 5.4.1 or 5.5 in JIS K 0125

Item	Criterion	Measurement procedures
1,1,2-trichloroethane	0.006 mg/L or less	Procedures specified in 5.1, 5.2, 5.3.1, 5.4.1 or 5.5 in JIS K 0125
Trichloroethylene	0.01 mg/L or less	Procedures specified in 5.1, 5.2, 5.3.1, 5.4.1 or 5.5 in JIS K 0125
Tetrachloroethylene	0.01 mg/L or less	Procedures specified in 5.1, 5.2, 5.3.1, 5.4.1 or 5.5 in JIS K 0125
1,3-dichloropropane	0.002 mg/L or less	Procedures specified in 5.1, 5.2 or 5.3.1 in JIS K 0125
Thiuram	0.006 mg/L or less	Procedures specified in Appendix 4
Simazine	0.003 mg/L or less	Procedures specified in 1 or 2 in Appendix 5
Thiobencarb	0.02 mg/L or less	Procedures specified in 1 or 2 in Appendix 5
Benzene	0.01 mg/L or less	Procedures specified in 5.1, 5.2 or 5.3.2 in JIS K 0125
Selenium	0.01 mg/L or less	Procedures specified in 67.2, 67.3 or 67.4 in JIS K 0102
Nitrate nitrogen and nitrite nitrogen	10 mg/L or less	For nitrate nitrogen, the procedures specified in 43.2.1, 43.2.3, 43.2.5 or 43.2.6 in JIS K 0102. For nitrite nitrogen, those in 43.1 in JIS K 0102.
Fluorine	0.8 mg/L or less	Procedures specified in 34.1 or 34.4 in JIS K 0102 or 34.1 c) in JIS K 0102 (note: the third sentence in [6] is excluded). (When materials that constitute an obstacle in a suspended solid and ion chromatography do not coexist, they can be omitted.) Also, procedures specified in Appendix 6.
Boron	1 mg/L or less	Procedures specified in 47.1, 47.3 or 47.4 in JIS K 0102
1,4-dioxane	0.05 mg/L or less	Procedures specified in Appendix 7

Notes: Appendixes 1 to 7 in the table are listed in the environmental standards for water pollution based on the provisions in Article 9 of the Basic Act on Pollution Control (Public Notice No. 59 by Japan's Ministry of the Environment, Law No. 132, from 1967).

Rocks should be regarded as part of the earth and sand to be discharged into seawater; therefore, they should satisfy the criteria on the earth and sand on water bottoms specified in **Public Notice No. 19 by Japan's Ministry of the Environment: Ministerial Ordinance Specifying the Criteria for Waste Containing Metals and Other Substances to be Discharged to Landfill Sites Stipulated in Article 5-1 of the Order for Enforcement of the Act on Prevention of Marine Pollution and Maritime Disaster** (February 17, 1973), shown in **Table 5.5.6**.

Table 5.5.6 Criteria for Heavy Metals and Other Substances

Item	Criterion
Alkylmercury compound	Not detected
Mercury or its compound	Mercury: 0.005 mg or less per 1 L of test liquid
Cadmium or its compound	Cadmium: 0.1 mg or less per 1 L of test liquid
Lead or its compound	Lead: 0.1 mg or less per 1 L of test liquid
Organophosphorus compound	Organophosphorus compound: 1 mg or less per 1 L of test liquid
Hexavalent chromium compound	Hexavalent chromium: 0.5 mg or less per 1 L of test liquid
Arsenic or its compound	Arsenic: 0.1 mg or less per 1 L of test liquid
Dicyan compound	Dicyan: 1 mg or less per 1 L of test liquid
Polychlorinated biphenyl	Polychlorinated biphenyl: 0.003 mg or less per 1 L of test liquid
Copper or its compound	Copper: 3 mg or less per 1 L of test liquid
Zinc or its compound	Zinc: 2 mg or less per 1 L of test liquid
Fluoride	Fluorine: 15 mg or less per 1 L of test liquid
Trichloroethylene	Trichloroethylene: 0.3 mg or less per 1 L of test liquid
Tetrachloroethylene	Tetrachloroethylene: 0.1 mg or less per 1 L of test liquid
Beryllium or its compound	Beryllium: 2.5 mg or less per 1 L of test liquid
Chromium or its compound	Chromium: 2 mg or less per 1 L of test liquid
Nickel or its compound	Nickel: 1.2 mg or less per 1 L of test liquid
Vanadium or its compound	Vanadium: 1.5 mg or less per 1 L of test liquid
Organochlorine compound listed in 2-24 in Appendix 3-3 of the waste treatment rules	Chlorine: 40 mg or less per 1 kg of sample

Item	Criterion
Dichloromethane	Dichloromethane: 0.2 mg or less per 1 L of test liquid
Tetrachloromethane	Tetrachloromethane: 0.02 mg or less per 1 L of test liquid
1,2-dichloroethane	1,2-dichloroethane: 0.04 mg or less per 1 L of test liquid
1,1-dichloroethylene	1,1-dichloroethylene: 1 mg or less per 1 L of test liquid
Cis-1,2-dichloroethylene	Cis-1,2-dichloroethylene: 0.4 mg or less per 1 L of test liquid
1,1,1-trichloroethane	1,1,1-trichloroethane: 3 mg or less per 1 L of test liquid
1,1,2-trichloroethane	1,1,2-trichloroethane: 0.06 mg or less per 1 L of test liquid
1,3-dichloropropane	1,3-dichloropropane: 0.02 mg or less per 1 L of test liquid
Tetramethyl thiuram disulfide (thiuram)	Thiuram: 0.06 mg or less per 1 L of test liquid
2-chloro-4,6-bis (ethylamino)-s-triazine (simazine)	Simazine: 0.03 mg or less per 1 L of test liquid
S-4-chlorobenzyl-N,N-diethylthiocarbamate (thiobencarb)	Thiobencarb: 0.2 mg or less per 1 L of test liquid
Benzene	Benzene: 0.1 mg or less per 1 L of test liquid
Selenium or its compound	Selenium: 0.1 mg or less per 1 L of test liquid
1,4-dioxane	1,4-dioxane: 0.5 mg or less per 1 L of test liquid

② **Strength tests**

Strength tests should be performed while referring to the instructions below.

(a) Unconfined compression tests

An unconfined compression test should be performed to understand the compressive strength of the stones in accordance with **JIS M 0302 Method of Test for Compressive Strength of Rock**. The stones should be classified as soft rock, semi-hard rock, hard rock or other types of rock based on the unconfined compression strength value, and this should be used as basic data for consideration based on the purpose of use.

(b) Radial compression and tension tests

A radial compression and tension test should be performed to understand the tensile strength of the stones in accordance with **JIS M 0303 Method of Test for Tensile Strength of Rock**. In this method, a compression line load is applied to the diameter direction of the section with two lines facing each other on the peripheral surface of a cylindrical specimen to calculate the tensile strength of the rocks indirectly from the load at which the specimen cracks due to pressure. The radial compressive and tensile strength value can be used to estimate the strength of the bedrock at the actual site, and that can be used as the basic data for digging the bedrock and other purposes.

(c) Ultrasonic nondestructive strength tests

Measuring the propagation velocity of ultrasonic waves through rock can show the unconfined compression strength, radial compressive and tensile strength, and other physical properties indirectly without destroying the rock. **Fig. 5.5.2** shows an example relationship between the propagation velocity and unconfined compression strength in a test performed in accordance with **JGS 2110 Method for Laboratory Measurement of Ultrasonic Wave Velocity of Rock by Pulse Test**. The accuracy of nondestructive strength tests is lower compared to unconfined compression tests and radial compression and tension tests, but the test procedures are simple. Therefore, a nondestructive strength test is desirably to be performed for a preliminary inspection that does not need to be highly accurate or when an unconfined compression test or radial compression and tension test are impossible.

For this type of test, specimens to be subject to an unconfined compression test and radial compression and tension test can be used, so it is desirable to perform a nondestructive strength test at the same time with other types of strength tests.

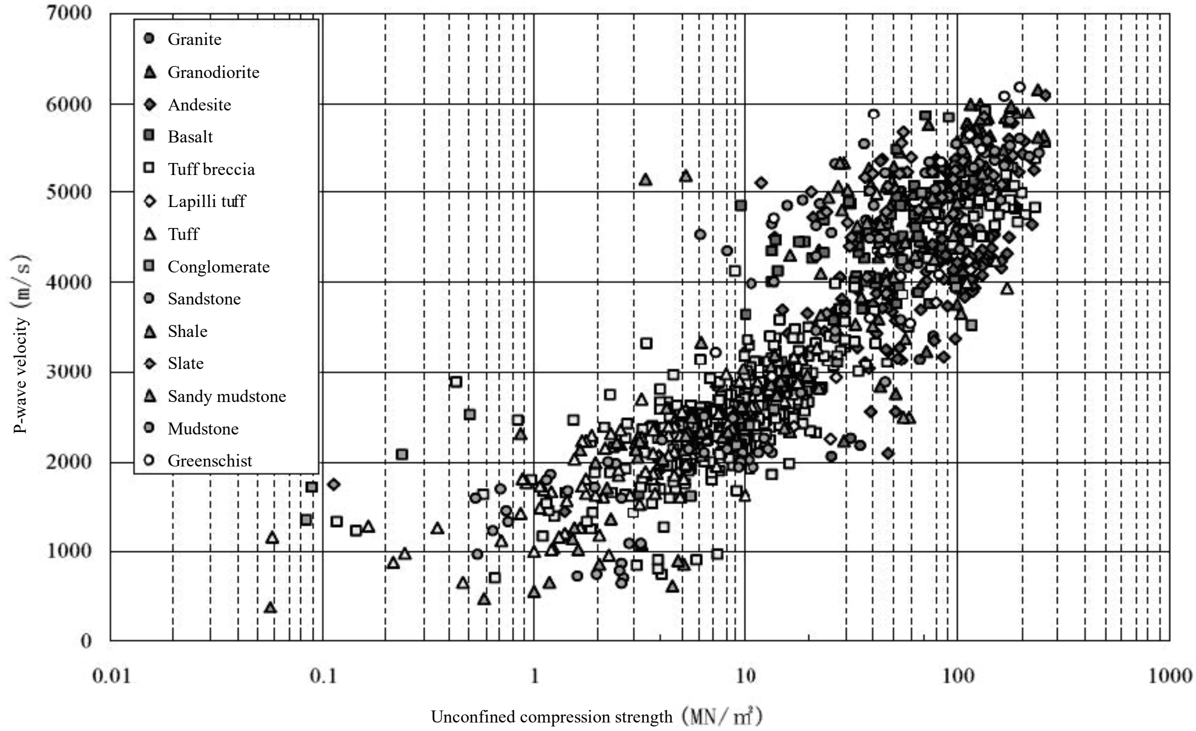

Figure 5.5.2 Relationship between the Propagation Velocity and Unconfined Compression Strength[22)]

③ **Durability tests**

Durability tests for stones judge the stability (weather resistance) of the stones against weathering. **JIS A 1122 Method of Test for Soundness of Aggregates by Use of Sodium Sulfate** is informative regarding stability tests, and **JIS A 1121 Method of Test for Resistance to Abrasion of Coarse Aggregate by Use of the Los Angeles Machine** is informative for abrasion resistant tests. Stones with a large loss mass percentage are not desirable to use as stone materials. The **Standard Concrete Specifications**[20)] specify that the loss mass percentage should be 10% or less for fine aggregates and 12% or less for coarse aggregates. Therefore, these values can be used as reference to judge the stability of stones.

To examine the abrasiveness of rocks, DRI tests are performed, and for coarse aggregates, Deval tests and Los Angeles tests are performed; of these, Los Angeles tests are the most common. Therefore, for abrasion resistant tests, **JIS A 1121 Method of Test for Resistance to Abrasion of Coarse Aggregate by Use of the Los Angeles Machine** is helpful.

Stones for which the amount of wearing-off is large are not desirable to use as stone materials. The **Standard Paving Specifications**[21)] specify that the amount of wearing-off on coarse aggregates should be 35% or less for paving concrete. Therefore, this value can be used as reference to judge the resistance of the stones to wearing-off.

5.5.2 Inspections and Tests at Quarrying Sites

(1) Geological Reconnaissance

A geological reconnaissance should be performed to understand the ground configuration, geographical features and bedrock at the quarrying site and around it in advance to make the quarrying reasonable and economical. Geologic knowledge is required for this type of test, so it is desirable to have a specialist participate or ask a specialist to carry out an inspection. The **Introduction to Geology for Civil Engineers**[22)] or other similar documents may be referred to.

① **Inspection procedures**

The inspection procedures are shown below.

(a) Survey the target area to understand an overview of the rocks and geologic structures. Use the data to check the general strike of the strata and the extension of the igneous complexes.

(b) For an inspection route, select a direction perpendicular to the direction of the general strike and igneous complexes as much as possible. At this time, include a cliff, quarry or cut slope that could be optimal to observe the outcrop into the route.

(c) When there is a bedrock outcrop, observe the rock type, rock quality, degree of weathering, strike of sedimentary rocks, inclination and conditions of cracks (e.g., faults and joints). Observe the continuity in the vertical direction or to the side and changes in the conditions for faults, joints and other cracks, in particular.

(d) Examine the relationship between the conformity and unconformity, whether the strata are new or old, the degree of metamorphic rock, the shapes of igneous complexes and other necessary items.

(e) When there is no bedrock outcrop, observe the shapes and sizes of strata and boulders covering the bedrock, rock types, ground configurations (e.g., flat or steep slopes), and other necessary items.

(f) At a planned quarrying site, the thickness of the topsoil and weathered layer could cause issues. Observe what types of tree clusters are growing on the cut slope and at the actual site and their conditions (e.g., whether the plants are deep rooted or shallow rooted).

(g) Examine whether a landslide or collapse has occurred.

(h) When the sea floor needs to be inspected, it is desirable to use an underwater camera or other similar tool to make observations such as the land in the vicinity. When it is impossible to directly observe the sea floor, have a licensed diver observe and record the conditions.

(i) In addition to the actual observation, use a chart and lead to check the sea floor configuration. When existing data on the geographical features by boring are available, use them as reference to estimate the ground configuration and geographical features.

(j) Be sure to record the items inspected at the site to a field book or map immediately at the site along with the outcrop location and serial numbers for organizations. Sketch and photograph important items.

(k) In addition to the observation, it is desirable to sample an approximately 30-cm square block from the bedrock for the typical ground as a specimen.

② **Organization of the inspection results**

Prepare a ground plan and sectional view of the geographical features of the quarrying site and around it to understand the overview of the bedrock. Use them as basic data to plan reasonable and economical quarrying. The inspection results should be organized as shown below.

(a) Prepare a route map by recording the data observed in the reconnaissance along the reconnaissance route.

(b) For the geographical features of the areas other than the reconnaissance route on the route map, draw figures of boundaries of the rocks and the intersection of the flat plane of the fracture zone, etc., and the ground surface based on the relationship between the planar structure in the geographical features and ground surface to prepare a ground plan and sectional view of the geographical features.

(c) The contraction scale should desirably be approximately 1/500 to 1/1000.

(2) Material Quality Judgment Tests

① For the strength and durability of rocks, refer to **Reference [Part II], Chapter 1, 5.5.1 Stone Testing**.

② When a dissolution test of heavy metals and other substances is performed, for testing each item, refer to **Reference [Part II], Chapter 1, 5.5.1 Stone Testing**.

③ When stones are used in the sea, refer to **Reference [Part II], Chapter 1, 5.5.1 Stone Testing**.

5.6 Wood

5.6.1 Wood Testing

(1) Wood Strength Tests

For testing the strength of wood, **references 23)** and **24)** may be referred to.

(2) Wood Durability Tests

For testing the durability of wood against decay fungi and termites, **reference 25)** may be referred to.

For testing the durability of wood against marine borers (e.g., Teredinidae and Limnoria lignorum), **reference 26)** may be referred to.

For evaluating the soundness of wood that is in use, **references 27)** to **29)** may be referred to.

5.7 Fenders

5.7.1 Fender Testing

This sub-section describes the testing of rubber fenders used for mooring facilities. Rubber fenders are installed onto mooring facilities to absorb the berthing energy of ships. They can be divided into two types: solid rubber fenders and pneumatic rubber fenders. Solid rubber fenders absorb the berthing energy of ships by elastic deformation and buckling of the fender rubber itself. On the other hand, pneumatic rubber fenders absorb the berthing energy of ships by compression of the air inside when the fenders deform. For testing pneumatic rubber fenders, **ISO 17357-1: 2014** can be referred to. This sub-section describes the testing of solid rubber fenders (hereinafter, rubber fenders), many of which are commonly used.

5.7.2 Test Types

Rubber fenders should have physical properties such as aging resistance and ozone resistance, so their performance shall be verified by physical tests. In addition, the compression properties of rubber fenders are generally expressed as the characteristic curves of absorbed energy and reaction force to the compression displacement, so their characteristics shall be verified by static compression tests. Furthermore, rubber fenders should be able to resist repeated loads, so the performance shall be verified by durability tests by repeated compression. However, it is difficult to test the durability of rubber fender products themselves, so tests should be carried out in advance and a certification of the durability should be obtained from a third party. This sub-section describes the physical, static compression and durability tests for fenders.

5.7.3 Physical Tests

(1) Sampling

One set of samples taken from the kneaded rubber used for the target lot in the production of rubber fenders shall be used as specimens for physical tests. Samples shall be taken from a rubber plate manufactured such that the composition and vulcanization conditions are the same as those of the products.

(2) Test Procedures

Since rubber fenders largely deform when ships berth, the quality of rubber materials is extremely important. Basically, rubber materials should be natural or synthetic rubber including carbon black or white carbon or vulcanized substances containing mixtures of these materials. They should resist aging, seawater, ozone, abrasions and the others. They also should be homogeneous, allowing no foreign matter to get in, with no air bubbles, flaws, cracks or other detrimental defects.[13)] In addition, the criteria of the physical properties of rubber used for rubber fenders are provided.[13)] Physical tests shall be carried out based on these criteria. There are several physical tests: an accelerated aging test to measure aging resistance, etc. (**JIS K 6257 Rubber, vulcanized or thermoplastic – Determination of heat aging properties**), a tensile test (**JIS K 6251 Rubber, vulcanized or thermoplastics – Determination of tensile stress-strain properties**), a durometer hardness test (**JIS K 6253-3 Rubber, vulcanized or thermoplastic – Determination of hardness – Part 3: Durometer method**), and an ozone resistance test to measure static ozone resistance **(JIS K 6259-1 Rubber, vulcanized or thermoplastic – Determination of ozone resistance – Part 1: Static and dynamic strain testing**).

(3) Handling of Test Results

The physical test results of rubber fenders should satisfy the material quality standards for rubber fenders specified in the **Standard Specifications for Port & Harbor Works**.[13] **Table 5.7.1** lists the criteria of the physical properties of rubber fenders.

Table 5.7.1 Physical Properties of Rubber[13]

Test item		Criterion	Test standard
Accelerated aging test	Tensile strength	80% or more of the value before heating	JIS K 6251
	Elongation	80% or more of the value before heating	JIS K 6251
	Hardness	Should not exceed +8 of the value before heating	JIS K 6253-3
Ozone resistance test	Static ozone degradation	No cracks should be found in a visual inspection after 72 hours	JIS K 6259-1

(4) Handling of Rejected Samples

If a test sample did not satisfy the physical test criteria, another two sets from the target lot shall be sampled for retesting. When all the samples in the two sets satisfy the criteria, the whole target lot is regarded as acceptable.

(5) Storage of Samples

The material of the samples used for a physical test should be that used for the rubber fender product lot. However, proving this is sometimes difficult due to certain reasons related to the rubber fender manufacturing process. Therefore, a sample of the same rubber as used for the physical test shall be stored.

(6) Properties in Physical Tests

Rubber exhibits its physical properties through a chemical reaction called vulcanization by heat and pressure. It takes time for heat to reach the inside of thick rubber such as in rubber fenders, so the ways of applying pressure and temperature vary between fender products. In addition, even when a sample made from the rubber material used for the product lot is vulcanized, it is impossible to match the physical properties to those on any section of the product. Therefore, the criteria that should be satisfied in physical tests are the levels required for the material so that it can exhibit the required performance without fail when the material is used for rubber fenders. It should be noted that the criteria do not guarantee that the material will always show the physical properties to be the same as those of various sections of the product.

5.7.4 Static Compression Tests

(1) Outline

Static compression tests are basic tests in which rubber fenders in commercially available sizes are actually compressed. This type of test is carried out for rubber fenders in a wide range of sizes, from actual products to scale models. In general, whether a static compression test of a large actual product size can be performed depends on the scale of the compression test apparatus. To understand the compressive performance (basic properties) of a rubber fender, a static compression test should be carried out for all product types and sizes to be manufactured and for the material quality.

As a rule, a static compression test shall be carried out by compression perpendicular to the side of the rubber fender specimen that will receive impacts. However, if there is a possibility that ships berth diagonally to the fender's impact receiving side, angular compression should desirably be carried out. When an angular compression test using an actual product is impossible because of restrictions due to the performance of the compression test apparatus, a scale model can be used instead.

(2) Sampling

A specimen for a static compression test shall be taken from one of ten rubber fender products (when the number of pieces is less than ten, one sample can be taken).

(3) Test Procedures

As a rule, a static compression test shall be carried out in accordance with the procedure shown below.[30]

① **Constant temperature environment**

The performance of rubber fenders is affected by the temperature, so the temperature of the specimens and test environments should desirably be constant at the target temperature, which is referred to as a set temperature. The set temperature shall be 23°C (standard temperature) when the standard static compressive performance is measured. A static compression test should desirably be carried out in an environment where the temperature is within ±15°C of the set temperature.

The temperature of the rubber fender can be made appropriate by storing the specimen for a certain period of time in a constant temperature environment within ±5°C of the set temperature. In addition, in an environment where the temperature is not constant, for example, in the case of a large rubber fender for which it is difficult to make the temperature of the specimen constant within ±5°C of the set temperature, use the average of the recorded environment temperatures to correct the temperature.

② **Preliminary compression**

Compress the rubber fender specimen to the design strain or more at least three times. In a compression test, for open-leg-type V-fenders, secure the fender to the face plate to prevent the legs from opening.

③ **Main compression**

Leave the rubber fender specimen within ±5°C of the set temperature for at least one hour after the preliminary compression to recover the fender's compressive performance from the influence of the preliminary compression. Then, compress the specimen once to more than the design strain. Use the compressive performance at that time as the static compressive performance.

As a rule, the compression rate shall be within the static compression rate range (0.01 to 0.3%/s). If it is impossible to store the specimen within ±5°C of the set temperature, use the average of the recorded environment temperatures to correct the temperature.

(4) Expression of the Compressive Performance

The compression test results of the rubber fender shall be expressed as a reaction force-displacement curve and absorbed energy-displacement curve. The compressive performance shall be expressed as the value of the absorbed energy while the fender was compressed to the specified compression displacement and the value of the maximum reaction force generated during that time. The specified compression displacement is the compression displacement at which the ratio of the absorbed energy value to the reaction force value calculated from the fender's compressive performance curves becomes the maximum.

(5) Handling of Test Results

In evaluations of the compressive performance values of rubber fenders, it shall be confirmed that the maximum reaction force value is equal to or less than the specified performance value and the absorbed energy value is equal to or higher than the specified performance value based on the **Standard Specifications for Port & Harbor Works**.[13)] The specified performance values refer to the characteristic values shown in the specifications or other similar documents. However, the performance values of rubber fenders vary within ±10% of the standard performance values of the applicable fenders listed in the catalogs issued by the fender manufacturers. Therefore, in general, the absorbed energy value shall be 90% of that to the rated deformation on the standard performance curve and the maximum reaction force value shall be 110% of that on the standard performance curve.

(6) Handling of Rejected Samples

If a test sample did not satisfy the specifications in the static compression test, a sample shall be taken from every five pieces that have not been tested from the target lot (when the number of pieces is less than five, one sample can be taken) for retesting. When all of these samples have passed the retest, all of them (except for those rejected) are regarded as acceptable. If another specimen was judged unacceptable in the retest, all the remaining pieces shall be tested and it shall be judged whether each individual one can be accepted.

5.7.5 Durability Tests

(1) Outline

In a durability test, a commercially available rubber fender in the minimum size or larger is actually repeatedly compressed a few thousand times. This type of test verifies if a rubber fender is safe enough to stand repeated compression when it is compressed without sparing enough time before recovering to the displacement and reaction force prior to the compressive loading.

Regarding the fatigue deterioration characteristics in durability tests, a scale model is not similar to an actual rubber fender, so a specimen that is of the same size as a commercially available product shall be used. The specimen can be the minimum size of commercially available products. Due to differences in the phenomena between the heat generation and heat dissipation, as the size of a fender continuously compressed becomes larger, the temperature of the rubber tends to increase easily. However, when ships actually berth, fenders are not continuously compressed, so this type of test is a method that can most directly evaluate the basic durability of rubber fenders.

(2) Test Procedures

As a rule, a durability test shall be carried out in accordance with the procedure shown below.[30)]

① **Constant temperature environment**

The set temperature for a durability test shall be 23°C (standard temperature). Make the temperature of the rubber fender specimen uniform targeting ±5° C of the set temperature.

② **Preliminary compression**

Compress the rubber fender specimen to the design strain at least three times. At this time, keep the compression rate constant and use the standard static compression rate.

③ **Static compression (before repeated compression)**

Leave the rubber fender specimen within ±5° C of the set temperature for approximately one hour after the preliminary compression, then compress the specimen once to more than the design strain. Use the compressive performance at that time as the standard static compressive performance.

④ **Repeated compression**

Compress the rubber fender specimen to the standard deflection specified by the fender manufacturer at a constant rate or while decelerating continuously 3,000 times. At that time, one compression to the design strain shall be completed within 150 seconds. For deceleration compression, the velocity waveform is not specified, but the locations of the compression start and unloading start strain shall be fixed. During repeated compression, the specimen shall not be artificially cooled.

⑤ **Visual inspection**

Visually check the rubber fender specimen after repeated compression for cracks and defects.

⑥ **Static compression (after repeated compression)**

Leave the rubber fender specimen within ±5°C of the set temperature, then compress the specimen once again to more than the design strain within 24 hours after the repeated compression. Use the compressive performance at that time as the static compressive performance after the repeated compression.

(3) Handling of Test Results

For the durability of rubber fenders, it shall be confirmed that no cracks or defects are found on the rubber fender specimen in a durability test by repeated compression based on the **Standard Specifications for Port & Harbor Works**.[13)] In addition, the durability test results of the rubber fender shall be used to obtain a certificate from an institution (third party) showing that the rubber fender has enough durability.

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6 Observation and Survey of Environments

6.1 Outline of Environmental Surveys

6.1.1 Objectives

(1) General

Environmental surveys are carried out to clarify the environmental properties and characteristics of the target area (e.g., water quality, sediment, living organisms and currents). Prior to the survey, the purpose of the survey needs to be understood, and survey items, survey periods, survey methods, accuracy and other necessary elements required to achieve the objectives should be properly selected.

Environmental surveys in port projects are broadly divided into environmental surveys to assess the impacts of projects on the environment and environmental surveys in marine environment improvement projects. The objectives vary between the two types of environmental surveys, and therefore, the approach for an effective survey differs between them. Both types of survey are separately described below.

(2) Environmental Surveys to Assess the Impacts of Projects on the Environment

For port infrastructure development projects, serious environmental impacts must be avoided or reduced. It is therefore necessary to make projections of the environmental impacts before starting projects (hereinafter, "pre-construction"), conduct monitoring during the construction works, and continue the monitoring after the facilities have begun to be used (hereinafter, "operation") based on the scale of the projects in accordance with the Japanese Environmental Impact Assessment Act and other relevant laws and regulations.

To properly assess the impacts of projects during the construction works and during operation, it is important to understand the environmental conditions in advance. When the impacts of the projects need to be predicted in advance, data should be collected for the predictions. For the data collection, existing survey results can be referred to in addition to carrying out new surveys.

During the construction works, the target items should be monitored to check if the impacts of the works are within the ranges assumed in the planning.

During operation, the target items should be monitored to check if the impacts caused by the existence of the facilities are within the predicted ranges. In addition, the target items need to be monitored at control points in which the impact of the projects is non-existent, to extract the impacts of the projects from the environmental data including other variations such as seasonal variations and yearly fluctuations.

(3) Environmental Surveys in Marine Environment Improvement Projects

Marine environment improvement projects are carried out to improve water quality, sediment and marine life among others. Therefore, for environmental surveys in marine environment improvement projects, assessment of the efficiency of the environment improvement methods should be carried out and the environment improvement effects properly assessed.

To carry out environment improvement projects effectively, the environmental characteristics of the target sea areas should be well understood in advance, effective environment improvement methods should be selected, and improvement goals determined. In addition, to accurately assess the environment improvement effects, it is important to compare the conditions before and after the projects. To that end, the environmental conditions in the target sea areas should also be properly understood in advance.

During the construction works, the monitoring items should be monitored to check if the impacts of the works are within the ranges assumed in the planning, similar to the case with environmental surveys to assess the impacts of the projects on the environments described in the previous item.

During operation, the assessment items should be monitored to check if the environments for which the improvement measures were implemented in the projects (hereinafter, "improved environments") have met the improvement goals determined in the planning. In addition, the assessment items at the control points unaffected by the projects need to be monitored during operation to extract the environment improvement effects from the environmental data including seasonal and yearly fluctuations among others.

6.1.2 Survey Flow

Key aspects of the survey process from the beginning of the survey to the interpretation of results are organized below.

(1) Understanding the Survey Objectives

Understand the background of the project for assessing the environmental impacts of the marine environment improvement project. When considering the survey results, collect the necessary information and discuss the survey items, survey and analysis techniques, and other necessary factors.

(2) Collecting and Organizing Existing Documents

In addition to the survey results from past years and information on the surrounding environment, information that matches the objectives of the project should be collected and organized. Examples of this information include relevant assessment standards regarding environmental impact assessments and indexes to check the effects regarding marine environment improvement projects.

To comprehensively organize background information on marine environments, there are materials available in the Tokyo Bay Environmental Information Center (http://www.tbeic.go.jp/), managed by the Yokohama Port and Airport Technology Survey Office under the Kanto Regional Development Bureau; marine cadasters (http://www.kaiyoudaichou.go.jp/) provided by the Hydrographic and Oceanographic Department of the Japan Coast Guard; and Setouchi Net (https://www.env.go.jp/water/heisa/heisa_net/setouchiNet/seto/), managed by the Environment Management Bureau under the Japanese Ministry of the Environment.

(3) Field Survey

Adequately understand the situation of the target areas and the information required to formulate a specific survey plan.

(4) Formulating a Survey Plan

Determine the survey points, survey periods, survey and analysis techniques, methods for summarizing results, safety management measures and other necessary elements. For planning, understand the background and characteristics of the project and consider the characteristics of the project assessment items.

(5) Procedure

Make the necessary applications for approval and authorization and notify stakeholders in the area about the work based on the details of the operations.

(6) Survey Preparations

Inspect the devices to be used in the survey, confirm that they function properly, prepare spare parts and containers for storing and transporting samples, clean the equipment and materials, and make other necessary preparations.

(7) Actual Survey

Before starting the survey, recheck the survey period, survey time, location and water depth of the target area, number of sampling times and other necessary aspects. The storage method for the samples should be considered based on the analysis items. In addition, it is important to implement and manage adequate safety measures based on the characteristics of the survey.

(8) Summarizing Results

When the results are organized, it is desirable to unify the units and other elements to make it easier to compare the results with past surveys. The survey results for each location often reflect the impacts of the surrounding environment. It is useful to consider changes in the topographic features, water quality, sediments, ecosystem and other elements in the vicinity. For consideration and examination of the results, attention should be paid to whether any changes originate within the project or the surrounding environment.

6.1.3 Field Surveys

A field survey should desirably be carried out regardless of the scale of the survey. Carrying out a field survey makes it possible to understand the survey in more detail, which is useful to formulate the survey plan. In addition, field surveys make it possible to prepare the best course of action for unexpected incidents at the actual sites.

When conducting a field survey, check the users in the area, survey vessel equipment, movement and evacuation routes and other necessary items while keeping in mind that the collected information will contribute to the survey plan (e.g., survey method, survey procedures, survey equipment and materials, safety management and target audience).

6.1.4 Formulating a Survey Plan

(1) Environmental Surveys to Assess the Impacts of Projects on the Environment

① Selecting survey items

Survey items and environmental impact factors should be selected based on the target sites of the projects and their details. During the selection, pay attention to the project types, laws and regulations for the target sea area, characteristics of the sea area, seriousness of possible impacts on the environment, and other necessary matters. In addition, the survey items should be selected based on each particular phase in the project (pre-construction, during construction works and operation). To select an adequate amount of survey items, it is desirable to understand the characteristics of the project well and use a matrix in which the environmental factors are linked to the phases in the project for examination.

To select the survey items (environmental impact assessment items) in projects requiring an environmental impact assessment, refer to the **Environmental Impact Assessment Guidebook for Port Development (2013)** [1].

In addition, for projects for which no environmental impact assessment is required, the survey items should be selected based on the impacts of the projects. For an example of the selection process, **Table 6.1.1** shows a matrix for selecting the survey items for a marine environment in a breakwater development project.

Table 6.1.1 Example Matrix for Selecting Survey Items (Breakwater Development Project)

Impact factor / Environmental factor			During construction works	Operation (Existence of breakwater)	Survey item
Environmental factors to be surveyed, predicted and assessed to keep the natural components of the environment in a healthy state	Aquatic environment	Water pollution		○	Water quality
		Suspended sediments	○		Turbidity
		Bottom sediments		○	Sediments
		Other environmental factors related to an aquatic environment		○	Current direction and speed
	Environment related to soil Other environment type	Topographic and geological features		○	Bathymetry
Environmental factors to be surveyed, predicted and assessed to secure the diversity of living organisms and preserve the natural environment	Animals	Important species and habitats	○	○	Zooplankton, larva and juvenile fish, benthic organisms, fish, shellfish and birds
	Plants	Important groups and communities	○	○	Phytoplankton, seaweed, and seagrass
	Ecosystem	Ecosystem that characterizes the region	○	○	Understand the ecological characteristics

② **Selecting survey points**

Survey points should be selected in the project area and its surroundings in consideration of the environmental conditions and the purpose of the collected data. As described in **6.2 Water Quality Surveys**, **Reference [Part II], Chapter 1, 6.3 Sediment Surveys** and **Reference [Part II], Chapter 1, 6.4 Living Thing Surveys**, the concepts regarding the arrangement of the survey points vary based on the characteristics of the survey items. The basic concepts regarding the arrangement of the survey points in each phase of the project are described below.

In the pre-construction phase, the survey points should be arranged so that it can be understood whether there is an impact during the construction work and during operation. Therefore, it is important to arrange the survey points within a range that the project may affect, and to select control points in environments similar to those of the survey points in other areas where the project will not have an impact. In addition, for the arrangement of the survey points, it is desirable to select an area for which no environment improvement is planned other than the project so that monitoring can be continued in the future.

During construction works, the survey points should be arranged to make it possible to understand whether there is any impact from the construction works. Therefore, the survey points should be arranged in an area which the construction possibly affects and at boundaries, etc., where there is a concern about an impact on other users in the area. It is also desirable to arrange control points in similar environments in areas where the project will not have an impact.

During operation, the survey points should be arranged to make it possible to understand whether the existence of the facility will have an impact. Therefore, it is desirable to arrange the survey points in an area for which some effects due to the project are predicted pre-construction, and to select control points identical to those in the pre-construction survey.

③ **Selecting a survey method**

In many cases, multiple survey methods are available for certain survey items. If the best survey method cannot be selected, the survey results will not satisfy the objectives and another survey may be required, which may affect the project schedule. The details of each survey method will be described in **Reference [Part II], Chapter 1, 6.2 Water Quality Surveys**, **6.3 Sediment Surveys** and **6.4 Living Thing Surveys**. The basic concepts regarding the selection of survey method are shown below.

To ensure that the survey data is objectively reliable, with the exception of special surveys, ordinal and highly reliable survey and analysis methods that have been certified by academic societies and public institutions should desirably be selected.

When the survey data over time is compared, the survey method and analysis accuracy (e.g., minimum determination limit and number of significant digits) should desirably be the same as those in past surveys to avoid disparities due to differences in the survey methods and analysis accuracy.

The classification standards of marine life change as studies develop, and judgments regarding the classifications may vary from researcher to researcher. To compare living organisms that appear over time, the classification standards should be agreed upon with those in past surveys, or how the revised classification standards correspond to those in past surveys should be understood.

In surveys of the water quality and sediments, the survey results are often compared to assessment standards for quantitative assessments. To compare them to the assessment standards, the lower limits to be reported in the analysis results should be set to the accuracy that the assessment standards demand. For information in the lower limits to be reported and minimum determination limits in the environmental standards that are often used as water quality assessment standards refer to **Kansuiki No. 92** (May 2001; revised in March 2013).[2)]

If the quantity of a target substance to be analyzed in a sample is too small, the result may not reach the minimum determination limit even when all of the sample is analyzed. It is important to contrast the concentration and quantity of the target substance in advance from existing documents and determine the required quantity to be sampled.

④ **Selecting survey period**

In projects requiring an environmental impact assessment, seasonal surveys are often demanded considering seasonal variations.

Generally, the water quality, currents and factors in sea areas have various types of periodicity, and those differentiate the response and emergence conditions of living organisms. For example, there are fluctuations during the day (diurnal), high and low tides from a half-day to approximately one day, lunar cycles (spring and neap tides) and seasonal variations. When environmental characteristics largely change depending on the tide, the timing of the surveys should be adjusted as much as possible. The response of living organisms also varies, so in some cases, the survey date and time should be determined considering the impacts of high and low tides and life history.

(2) Environmental Surveys in Marine Environment Improvement Projects

① Selecting survey items

The survey items are selected mainly to collect information required to formulate marine environment improvement project plans, to check whether there is any impact during the construction, and whether the environment during operation has been improved as planned. To select an adequate amount of survey items, the characteristics of the environments to be improved should be well understood and a matrix in which the environmental factors are linked to the project phases should desirably be used. As an example of the survey item selection, **Table 6.1.2** shows a matrix for selecting the survey items in a tidal flat development project.

When formulating plans, existing documents should be used, if available, and any necessary field surveys should be included. For the operation phase, survey items should be selected depending on the characteristics of the target environments, the ecological characteristics of the living organisms that are expected to inhabit the areas and the target functions that the improved environments should have among other factors.

Survey items largely vary between marine environment improvement project types, so survey plans unique to the project should be formulated. For concepts regarding the selection of survey items in typical marine environment improvement projects (tidal flats, seaweed beds and coral reef development) refer to the **Marine Environment Regeneration Handbook: Planning, Engineering and Implementation**.[3), 4), 5), 6)]

Table 6.1.2 Example Matrix for Selecting Survey Items (Tidal Flat Development Project)

<table>
<tr><th colspan="3">Project phase
Environmental factor</th><th>Planning (*1)</th><th>During construction</th><th>Operation (effect of developed tidal flats)</th><th>Survey items
(objectives and key points)</th></tr>
<tr><td rowspan="2">Environmental factors to be surveyed and assessed to determine the specifications of the tidal flat to be developed</td><td rowspan="2">Environment related to the design of the tidal flat</td><td>Topographic features</td><td>○</td><td></td><td></td><td>Depth and beach line measurement
(Calculate the necessary sediment volume and consider construction conditions.)</td></tr>
<tr><td>Wave and current conditions</td><td>○</td><td></td><td></td><td>Waves, current direction and current speed
(Consider the movement of the sediment after development)</td></tr>
<tr><td rowspan="7">Environmental factors related to the survival of the target organisms in the tidal flat to be developed</td><td rowspan="3">Aquatic environment</td><td>Water quality</td><td>○</td><td></td><td></td><td rowspan="7">Existing documents
(Understand the conditions of the living organisms that are expected to inhabit the tidal flat to be developed, their ecological characteristics and other necessary matters using the existing documents)</td></tr>
<tr><td>Sediments</td><td>○</td><td></td><td></td></tr>
<tr><td>Water depth</td><td>○</td><td></td><td></td></tr>
<tr><td rowspan="4">Biological environment</td><td>Biota that is expected to inhabit the area</td><td>○</td><td></td><td></td></tr>
<tr><td>Target organisms (*2)</td><td>○</td><td></td><td></td></tr>
<tr><td>Competing organisms</td><td>○</td><td></td><td></td></tr>
<tr><td>Feeding resources</td><td>○</td><td></td><td></td></tr>
</table>

<table>
<tr><th colspan="3">Project phase / Environmental factor</th><th>Planning (*1)</th><th>During construction</th><th>Operation (effect of developed tidal flats)</th><th>Survey items
(objectives and key points)</th></tr>
<tr><td>Environmental factors to be surveyed, predicted and assessed regarding whether good conditions are kept during the construction</td><td>Aquatic environment</td><td>Suspended sediments</td><td></td><td>○</td><td></td><td>Turbidity</td></tr>
<tr><td rowspan="6">Environmental factors to be surveyed and assessed regarding whether the developed tidal flat exerts the intended effects</td><td>Environment related to changes in the shape of the developed tidal flat</td><td>Topographic features</td><td></td><td></td><td>○</td><td>Depth and beach line measurement
(Understand changes in shape)</td></tr>
<tr><td rowspan="2">Aquatic environment</td><td>Water quality</td><td></td><td></td><td>○</td><td>Water quality</td></tr>
<tr><td>Sediments</td><td></td><td></td><td>○</td><td>Sediments</td></tr>
<tr><td rowspan="2">Biological environment</td><td>Biota that is expected to inhabit the area</td><td></td><td></td><td>○</td><td>Benthic organisms, seaweed and seagrass, plants, larval and juvenile fishes, fishes, shellfishes, and birds,
(Understand the living organisms that are expected to inhabit the developed tidal flat and the conditions of the target, feeding sources and competition)</td></tr>
<tr><td>Target organisms (*2)</td><td></td><td></td><td>○</td><td>Floating larvae, settled larvae and maturation value of littleneck clams
(Understand the living situations of the target organisms in detail)</td></tr>
<tr><td>Floating debris</td><td>Waste that washes up on shore, seaweed and shells</td><td></td><td></td><td>○</td><td>Floating debris
(Understand whether there is a factor that hinders the functions of the developed tidal flat)</td></tr>
</table>

*1: Surveys at the planning stage will be conducted mainly by gathering existing information, and field surveys will be conducted to collect the missing information.

*2: This table was formulated considering littleneck clams as the main species (target organism) that is expected to live in the developed tidal flat.

② **Selecting survey points**

The survey items required in marine environment improvement projects vary between projects, as well as their contents. Survey points should be selected focusing in the project areas and surroundings considering the purpose of the data obtained in each survey. As described in **Reference [Part II], Chapter 1, 6.2 Water Quality Survey**s, **6.3 Sediment Surveys** and **6.4 Living Thing Surveys**, the concepts of how to arrange the survey points differ depending on the survey items. The basic concepts of how to arrange the survey points in each project phase are described below.

When a plan is formulated, the survey area should be determined so that the characteristics of the environment to be improved and improvement goals can be discussed. Therefore, the survey points should be arranged in the project area, and if another environment similar to the project area exists in the surroundings, control points should be arranged in that area. This similar environment will serve as a reference to understand and evaluate the effects after completion.

During construction works, the survey points should be arranged so as to understand whether the construction has an impact. Therefore, the survey points should be arranged in an area which the construction possibly affects and at boundary zones, etc., where there is a concern about an impact on other users. It is also desirable to arrange control points in areas with environments similar where the project will not have an impact.

During operation, the survey points should be arranged such that the effects of the improved environment can be understood. Therefore, the survey points should desirably be arranged in the project area, and if another environment similar to the project area exists, control points should also be arranged in that area.

③ **Selecting a survey method**

The survey items required for marine environment improvement projects vary between projects and in their contents. In addition, for the same survey items, multiple survey and analysis methods are available. Each survey item is discussed in **Reference [Part II], Chapter 1, 6.2 Water Quality Surveys**, **6.3 Sediment Surveys** and **6.4 Living Thing Surveys**. The basic concepts for selecting a survey method are described below.

To ensure that the survey data is objectively reliable, with the exception of special surveys, ordinal and highly reliable survey and analysis methods that have been certified by academic societies and public institutions should desirably be selected (refer to **6.1.4 [1]** ③).

When survey data over time is compared, the survey method and analysis accuracy (e.g., minimum determination limit and number of significant digits) should desirably be the same as those in past surveys to avoid disparities due to differences in the survey methods and analysis accuracy (refer to **6.1.4 [1]** ③).

The classification standards of marine life change as studies develop, and judgments regarding the classifications may vary from researcher to researcher. To compare living organisms that appear over time, the classification standards should agree with those in past surveys, or how the revised classification standards correspond to those in past surveys should be understood (refer to **6.1.4 [1]** ③).

For surveys to understand the effects of the improved environments, a special method is often used for each effect. To make it possible to compare the survey results to past data, past knowledge and other information should desirably be referred to when a special survey method is selected. In addition, when a special survey method is selected, it is important to record the specific survey procedures so that the surveys that follow can be made using the same methods.

④ **Selecting survey period**

The concept for determining the survey period is the same as that for environmental surveys to assess the impacts of projects on the environments described in the previous section. Although seasonal variations are also important in marine environment improvement projects, surveys are more frequently carried out immediately after the project start rather than to monitor seasonal variations in order to understand changes in the water quality and sediments, as well as the introduction of living organisms, at an early stage.

6.1.5 Interpreting and Assessing Survey Data

(1) Understanding Conditions When Survey Data is Collected

For both types of environmental surveys (surveys to assess the impacts of projects on the environment and environmental surveys in marine environment improvement projects), when survey data is used, it is important to understand the environmental conditions at the time when the data was collected. For example, when data on the pollution (water quality) collected during construction work monitoring is interpreted, it should be determined whether the pollution was caused by the construction, rains before the survey, or by another load source in the vicinity. In addition, for surveys targeting living organisms, the captured amounts and varieties, etc., may largely change due to weather and hydrographic conditions instead of environmental changes over time.

Making these judgments becomes difficult if the environmental conditions at the time of the survey are not known. Some example items that should be understood at the actual site in addition to the survey items are listed below.

- Time, weather, degree of cloudiness, temperature, wind direction, wind velocity, tidal level, differences in tidal level, wave direction and wave height at the time of the survey

- Particular conditions observed at and around the survey points at the time of the survey (e.g., construction work conditions, operation conditions, effluents, fronts, red and blue tides)
- Weather and hydrographic conditions during a certain period before the survey

(2) Assessments for Environmental Surveys to Evaluate the Impacts of Projects on the Environment

There are three types of assessments for environmental surveys to assess the impacts of projects on the environment: assessments by comparisons to reference values, assessments by comparisons to predicted results and assessments based on changes in trends over time.

An assessment by comparisons to reference values is carried out based on to what degree the survey results match the applicable reference values. For a description of the typical standards used for the assessments, refer to the **Marine Survey Engineering Manual: Water Quality and Sediment Surveys**.[7)]

An assessment by comparisons to predicted results is carried out by comparing the results predicted for after the completion obtained by numerical simulations, etc., to survey the results during operation.

An assessment based on changes in trends over time is carried out by clarifying the trends (e.g., worsened, no change, or improved) by linking the monitoring survey results over time starting from pre-construction to operation.

Regardless of the assessment method, if the survey results deviate from the predicted results, the factors should be considered and measures should be taken to eliminate such deviations, or other accommodative management is required.

Typical reference values used for the assessments are listed below, and **Reference [Part II], Chapter 1, 6.1.6 Reference Value Lists** organizes the reference values.

① **Environmental standards**

These standards have been determined based on the stipulation that the standards should desirably be maintained to protect human health and preserve living environments specified in the Japanese Basic Environment Act, and the Environmental Quality Standards for Water Pollution for marine environments. For the details regarding the standards, refer to **Notification No. 59 issued by the Japanese Environment Agency** on December 28, 1971 (final revision: H-28 Environment Notice No. 37 [http://www.env.go.jp/kijun/mizu.html]). In addition, the environmental standards for dioxins have been determined based on the **Japanese Act on Special Measures against Dioxins** (Act No. 105 of 1999). For the details regarding the standards, refer to **Notification No. 68 issued by the Japanese Environment Agency** on December 27, 1999 (final revision: H-21 Environment Notice No. 11 [http://www.env.go.jp/kijun/dioxin.html]).

② **Water quality standards for fisheries**

These standards have been determined based on the following: If the quantity of a substance existing in a water area exceeds the limit under natural conditions, or a substance that does not exist in the natural environment has accumulated, it may hinder the production of common living organisms in the water area and thus damage fisheries. Therefore, to prevent the water quality in a natural water area from being adversely affected, the environmental (natural) conditions should be fully studied and environmental water quality standards for protecting aquatic life should be established. These standards are called Water Quality Standards for Fisheries. For the details regarding the standards, refer to the **seventh edition of the Water Quality Standards for Fisheries (2012)**.[8)]

③ **Water quality criteria for bathing beaches**

These criteria have been determined to judge and assess how safe is the water quality of bathing beaches in regard to human health and comfort. For the details regarding the criteria, refer to the **Revision of the Water Quality Criteria for Bathing Beaches** and the **Establishment of the Guidelines for Comfortable Bathing Beaches** (https://www.env.go.jp/en/water/wq/wbcbbeach.html),originally published by the Japanese Ministry of the Environment in a press release on March 28, 1997.

④ **Provisional removal standards for sediments**

These standards have been determined regarding the removal, etc., of contaminated sediments that may deteriorate the water quality of public water areas and pollute fish and shellfish. Reference values are provided for sediments of marine environments; for the details regarding these values refer to **Kansuikan No. 119** issued on October 28, 1975 (final revision: S-63 Kansuikan No. 127 [https://www.env.go.jp/hourei/05/000179.html]).

⑤ **Bottom sediment and soil criteria**

These criteria have been specified in the **Ministerial Order for Determining the Judgment Criteria for Waste Including Metals to Be Discharged to Reclaimed Land** stipulated in article 5-1 of the **Enforcement Order of the Act on Prevention of Marine Pollution and Maritime Disaster** (Ordinance of the Prime Minister's Office No. 6 issued in 1973). For the details regarding the criteria, refer to e-Gov, the official web portal for the Government of Japan (http://elaws.e-gov.go.jp/search/elawsSearch/elaws_search/lsg0500/detail?lawId= 348 M 50000002006 &openerCode=1).

⑥ **Bottom mud criteria determined by the Bureau of Port and Harbor, Tokyo Metropolitan Government**

These standards are applied by the Bureau of Port and Harbor, Tokyo Metropolitan Government to assess the bottom mud at canal sections that are to be dredged. For the details regarding the standards, refer to the **Survey Outline of Bottom Sediments in Tokyo Port** as determined by the director of the Bureau of Port and Harbor, Tokyo Metropolitan Government on February 18, 1977, and partially revised on October 31, 2014 (http://www.kouwan.metro.tokyo.jp/business/shiyosho/suiteichousayoukou_itibukaisei261031.pdf). For local governments, information on the project area should be collected pre-construction to ascertain whether the local government has its own standards.

(3) Assessments for Environmental Surveys in Marine Environment Improvement Projects

There are four types of assessments for environmental surveys in marine environment improvement projects: assessments by comparisons to target values, assessments by comparisons to control points, assessments based on changes in trends over time, and assessments by predicting improvement effects.

An assessment by comparisons to target values is carried out by comparing the monitoring survey results during operation to the water quality, sediments, condition of living organisms and other elements that were determined in the improvement goals. The environmental conditions that favor living organisms expected to inhabit the area are determined in the improvement goals. Regarding the environmental conditions for each species, the following documents are available: **Reference Book for Designing Fishing Ports and Fishing Grounds (2015) Volume 1**[9] to review the available knowledge on fisheries production (animals: 59 varieties, plants: 12 varieties); **Aquatic Life Ecology Data (March 1981)**[10], a document that comprehensively organizes the environmental conditions, etc., suitable for 84 varieties of aquatic life according to their developmental stage; and **Aquatic Life Ecology Data (continuation) (March 1983)**[11] targeting 50 new varieties of aquatic life following the previous edition.

An assessment by comparisons to control points is carried out by comparing the monitoring survey results during operation to those in a similar environment outside and around the project area.

An assessment based on changes in trends over time is carried out by clarifying the trends (e.g., worsened, no change, and improved) by linking the monitoring survey results over time from pre-construction to operation.

An assessment based on projecting improvement effects is carried out by projecting the improvement effects of a marine environment improvement project quantitatively and comparing the details of the improvement effects and degrees to the present conditions and environmental standard values, or by assessing whether the environment is suitable for the survival of living organisms.

Regardless of the assessment method, if the survey results deviate from the predicted results, the factors should be considered and measures should be taken to eliminate such deviations, or other accommodative management is required.

For marine environment improvement projects, assessments after understanding the importance of the multi-faceted functions and values brought by the improved environments are essential. For example, a developed tidal flat has various functions for cultivating living organisms, water and bottom sediment purification, decreasing waves, amenity and fishing grounds having a wide range of values in regard to the environment, as well as disaster prevention, recreation and industries. The assessment items for environmental surveys in marine environment improvement projects should be selected so that the functions and values of the improved environments can be accurately judged. It is important to assess the functions and values in accordance with the aforementioned four policies and promote the preservation, regeneration, creation and maintenance of the improved environments while the conditions of the assessment items are understood through monitoring.

For the details of the monitoring survey result assessments and accommodative management of typical marine environment improvement projects (preservation, regeneration and creation of tidal flats, seaweed beds and coral reefs), refer to the **Marine Environment Regeneration Handbook: Planning, Engineering and Implementation, Volume 2: Tidal Flats**,[4] **Volume 3: Seaweed Beds**[5] and **Volume 4: Coral Reefs**.[6]

6.1.6 Reference Value Lists

The reference values included in the standards listed in **6.1.5 (2)** as of December 2017 are listed and organized below.

Table 6.1.3 Quality Standards for Water Pollution (Environmental Standards for Protecting Human Health)

(outlined in **6.1.5 [2]** ①)

Item	Reference value	Item	Reference value
Cadmium	0.003 mg/L or less	1,1,2-trichloroethane	0.006 mg/L or less
Total cyanide	Not present	Trichloroethylene	0.01 mg/L or less
Lead	0.01 mg/L or less	Tetrachloroethylene	0.01 mg/L or less
Hexavalent chrome	0.05 mg/L or less	1,3-dichloropropene	0.002 mg/L or less
Arsenic	0.01 mg/L or less	Thiuram	0.006 mg/L or less
Total mercury	0.0005 mg/L or less	Simazine	0.003 mg/L or less
Alkylmercury	Not present	Thiobencarb	0.02 mg/L or less
PCB	Not present	Benzene	0.01 mg/L or less
Dichloromethane	0.02 mg/L or less	Selenium	0.01 mg/L or less
Tetrachloromethane	0.002 mg/L or less	Nitrate nitrogen and nitrite nitrogen	10 mg/L or less
1,2-dichloroethane	0.004 mg/L or less	Fluorine	0.8 mg/L or less
1,1-dichloroethylene	0.1 mg/L or less	Boron	1 mg/L or less
Cis-1,2-dichloroethylene	0.04 mg/L or less	1,4-dioxane	0.05 mg/L or less
1,1,1-trichloroethane	1 mg/L or less		

Note 1: The reference values are annual averages. However, the reference value for total cyanide corresponds to the maximum value.

Note 2: The reference values for fluorine and boron are not applied to marine areas.

Table 6.1.4 Quality Standards for Water Pollution (Items to Be Monitored and Guideline Values for the Protection of Human Health for Public Water Areas)

(outlined in **6.1.5 [2]** ①)

Item to be monitored	Guideline value	Item to be monitored	Guideline value
Chloroform	0.06 mg/L or less	Fenobcarb (BPMC)	0.03 mg/L or less
Trans-1,2-dichloroethylene	0.04 mg/L or less	Iprobenfos (IBP)	0.008 mg/L or less
1,2-dichloropropane	0.06 mg/L or less	Chloronitrophen (CNP)	—
p-dichlorobenzene	0.2 mg/L or less	Toluene	0.6 mg/L or less
Isoxathion	0.008 mg/L or less	Xylene	0.4 mg/L or less
Diazinon	0.005 mg/L or less	Diethylhexyl phthalate	0.06 mg/L or less
Fenitrothion (MEP)	0.003 mg/L or less	Nickel	—
Isoprothiolane	0.04 mg/L or less	Molybdenum	0.07 mg/L or less
Oxine‐copper (organocopper)	0.04 mg/L or less	Antimony	0.02 mg/L or less
Chlorothalonil (TPN)	0.05 mg/L or less	Vinyl chloride monomer	0.002 mg/L or less
Propyzamide	0.008 mg/L or less	Epichlorohydrin	0.0004 mg/L or less
EPN	0.006 mg/L or less	Total manganese	0.2 mg/L or less
Dichlorvos (DDVP)	0.008 mg/L or less	Uranium	0.002 mg/L or less

Table 6.1.5 Quality Standards for Water Pollution
(Environmental Standards for the Preservation of Living Environments for Sea Areas I)

(outlined in **6.1.5 [2]** ①)

Type	Item / Intended use or purpose	Hydrogen ion concentration (pH)	Chemical oxygen demand (COD)	Dissolved oxygen (DO)	Coliform count	n-hexane extract (e.g., oil content)
		—	mg/L	mg/L	MPN/100 mL	mg/L
A	Fishery class 1, bathing, environmental conservation and items listed in field B or lower	7.8 or more 8.3 or less	2 or less	7.5 or more	1,000 or less	Not present
B	Fishery class 2, industrial water and items listed in field C	7.8 or more 8.3 or less	3 or less	5 or more	—	Not present
C	Environmental conservation	7.0 or more 8.3 or less	8 or less	2 or more	—	—

Note 1: Regarding fishery class 1, for sampling points in areas of aquaculture of oysters that can be consumed raw, the coliform count should be 70 MPN/100 mL or less.

Note 2: Regarding fishery class 2, the method to measure COD at sampling points in the aquaculture of seaweed should be the alkaline process.

Table 6.1.6 Quality Standards for Water Pollution

(Environmental Standards for the Preservation of Living Environments for Sea Areas II)

(outlined in **6.1.5 [2]** ①)

Type	Item / Intended use or purpose	Total nitrogen (T-N)	Total phosphorus (T-P)
		mg/L	mg/L
I	Environmental conservation and items listed in field II or lower (Except fishery classes 2 and 3)	0.2 or less	0.02 or less
II	Fishery class 1, bathing and items listed in field III or lower (Except fishery classes 2 and 3)	0.3 or less	0.03 or less
III	Fishery class 2 and items listed in field IV (Except fishery class 3)	0.6 or less	0.05 or less
IV	Fishery class 3, industrial water and biological habitat conservation	1 or less	0.09 or less

Note 1: The reference values are annual averages.

Note 2: The type of water area should be specified for marine areas where marine phytoplankton may significantly proliferate.

Table 6.1.7 Quality Standards for Water Pollution
(Environmental Standards for the Preservation of Living Environments for Sea Areas III)

(outlined in **6.1.5 [2]** ①)

Type	Item / Aquatic habitat	Total zinc	Nonylphenol	Linear alkylbenzene sulfonate and its salt
		mg/L	mg/L	mg/L
Habitat A	Water area inhabited by aquatic organisms	0.02 or less	0.001 or less	0.01 or less
Habitat SA	Among the water areas applicable to habitat A, water areas that should be particularly preserved as spawning grounds (breeding areas) of aquatic life or as habitats for fish larva and juveniles	0.01 or less	0.0007 or less	0.006 or less

Table 6.1.8 Quality Standards for Water Pollution
(Environmental Standards for the Preservation of Living Environments for Sea Areas IV)

(outlined in **6.1.5 [2]** ①)

Type	Item / Residence and spawning habitats	DO on bottom layer mg/L
Habitat 1	Water areas that preserve and regenerate places where aquatic life with low anoxia tolerance can live, or water areas that preserve and regenerate places where aquatic life with low anoxia tolerance can reproduce	4.0 or more
Habitat 2	Water areas that preserve and regenerate places where aquatic life can live, except those with low anoxia tolerance, or water areas that preserve and regenerate places where aquatic life can reproduce, except those with low anoxia tolerance	3.0 or more
Habitat 3	Water areas that preserve and regenerate places where aquatic life with high anoxia tolerance can live, water areas that preserve and regenerate places where aquatic life with high anoxia tolerance can reproduce, or water areas that eliminate places without living organisms	2.0 or more

Note 1: The reference values are daily averages.
Note 2: In sampling points where DO significantly changes near the bottom surface, use a Van Dorn horizontal type water sampling bottle.

Table 6.1.9 Quality Standards for Water Pollution
(Items to Be Monitored and Guideline Values for the Preservation of Aquatic Life for Marine Water Areas)

(outlined in **6.1.5 [2]** ①)

Item	Type	Guideline value
Chloroform	Habitat A	0.8 mg/L or less
	Habitat SA	0.8 mg/L or less
Phenol	Habitat A	2 mg/L or less
	Habitat SA	0.2 mg/L or less
Formaldehyde	Habitat A	0.3 mg/L or less
	Habitat SA	0.03 mg/L or less
4-t-octyl phenol	Habitat A	0.0009 mg/L or less
	Habitat SA	0.0004 mg/L or less
Aniline	Habitat A	0.1 mg/L or less
	Habitat SA	0.1 mg/L or less
2,4-dichlorophenol	Habitat A	0.02 mg/L or less
	Habitat SA	0.01 mg/L or less

Table 6.1.10 Standards Based on the Japanese Act on Special Measures against Dioxins

(outlined in **6.1.5 [2]** ①)

Medium	Environmental standard	Note
Air	0.6 pg-TEQ/m^3 or less	To be assessed using the annual average Determined to protect human health
Water quality	1 pg-TEQ/L or less	To be assessed using the annual average To be applied to public water areas and groundwater
Sediment	150 pg-TEQ/g or less	To be applied to sediments in public water areas
Soil	1000 pg-TEQ/g or less	Except soil that is properly differentiated (e.g., managed reclamation areas) (survey required if value exceeds 250 pg-TEQ/g)
Effluent	10 pg-TEQ/L or less	Discharge standards

Table 6.1.11 Water Quality Standards for Fisheries (e.g., Water Quality and Organic Matter)

(outlined in **6.1.5 [2]** ②)

Sea area			
Organic matter	General sea areas Seaweed farms and closed bays	COD_{OH}*	1 mg/L or less 2 mg/L or less
Nutritive salts	Fishery class 1 specified in the environmental standards	Total nitrogen Total phosphorus	0.3 mg/L or less 0.03 mg/L or less
	Fishery class 2 specified in the environmental standards	Total nitrogen Total phosphorus	0.6 mg/L or less 0.05 mg/L or less
	Fishery class 3 specified in the environmental standards	Total nitrogen Total phosphorus	1.0 mg/L or less 0.09 mg/L or less
	Minimum concentration required for cultivating seaweed	Inorganic nitrogen Inorganic phosphorus	0.07 to 0.1 mg/L 0.007 to 0.014 mg/L
DO	Sea areas Bottom layers of fishing grounds in closed bay in summer periods	6 mg/L or more 4.3 mg/L or more	
pH	7.8 to 8.4 for sea areas The pH should not drastically change as it may adversely affect living organisms.		
Suspended solids (SS)	Suspended solids added artificially should be 2 mg/L or less. The necessary light intensity is maintained at a water depth suitable for the propagation of seaweed with no impact its reproduction and growth.		
Coloration	The penetration of light required for photosynthesis should not be hindered. Should not cause avoidance.		
Water temperature	The water temperature should not change at levels that adversely affect aquatic life.		
Coliform bacilli	Should be 1,000 MPN/100 mL or less. In the case of cultivation of oysters that can be consumed raw the value should be 70 MPN/100 mL or less.		
Oil content	No oil content should be detected underwater. No oil film should be seen on the water surface.		

*: COD should be measured by the alkaline process.

Table 6.1.12 Water Quality Standards for Fisheries (Water Quality in Sea Areas: Hazardous Substances Specified in the Environmental Standards for the Protection of Human Health)

(outlined in **6.1.5 [2]** ②)

Item	Reference value
Cadmium	0.003 mg/L
Total cyanide	0.001 mg/L
Lead	0.003 mg/L
Hexavalent chrome	0.01 mg/L
Arsenic	0.01 mg/L
Total mercury	0.0001 mg/L
Alkylmercury	0.001 mg/L
PCB	Not present
Dichloromethane	0.02 mg/L
Tetrachloromethane	0.002 mg/L
1,2-dichloroethane	0.004 mg/L
Cis-1,2-dichloroethylene	0.04 mg/L
1,1-dichloroethylene	0.02 mg/L
1,1,1-trichloroethane	0.5 mg/L
1,1,2-trichloroethane	0.006 mg/L
Trichloroethylene	0.03 mg/L
Tetrachloroethylene	0.002 mg/L
1,3-dichloropropene	0.002 mg/L
Thiuram	-
Simazine	-
Thiobencarb	0.02 mg/L
Benzene	0.01 mg/L
Selenium	0.01 mg/L
Nitrate nitrogen	7 mg/L
Nitrite nitrogen	0.06 mg/L
Fluorine	1.4 mg/L
Boron	4.5 mg/L

Table 6.1.13 Water Quality Standards for Fisheries (Water Quality in Sea Areas: Hazardous Substances Specified in the Environmental Standards for the Preservation of Living Environments)

(outlined in **6.1.5 [2]** ②)

Item	Reference value
Zinc	Not present

Table 6.1.14 Water Quality Standards for Fisheries
(Water Quality in Sea Areas: Hazardous Substances Specified as Items to Be Monitored)

(outlined in **6.1.5 [2]** ②)

Item	Reference value
Chloroform	0.06 mg/L
Trans-1,2-dichloroethylene	0.04 mg/L
1,2-dichloropropane	0.06 mg/L
p-dichlorobenzene	0.07 mg/L
Isoxathion	0.008 mg/L
Diazinon	Not present
Fenitrothion (MEP)	Not present
Isoprothiolane	0.04 mg/L
Oxine‐copper	-
Chlorothalonil (TPN)	0.002 mg/L
Propyzamide	-
EPN	Not present
Dichlorvos (DDVP)	Not present
Fenobcarb (BPMC)	0.003 mg/L
Iprobenfos (IBP)	0.008 mg/L
Chloronitrophen (CNP)	0.08 mg/L
Toluene	0.3 mg/L
Xylene	-
Diethylhexyl phthalate	0.06 mg/L
Nickel	0.007 mg/L
Molybdenum	0.07 mg/L
Antimony	0.4 mg/L
Manganese	0.2 mg/L

Table 6.1.15 Water Quality Standards for Fisheries
(Water Quality in Sea Areas: Quality Standards for Water Pollution Due to Dioxins)

(outlined in **6.1.5 [2]** ②)

Item	Reference value
Dioxins	1 pg-TEQ/L

Table 6.1.16 Water Quality Standards for Fisheries (Water Quality in Sea Areas: Reference Values for Hazardous Substances for Which No Reference Values and Guideline Values Have Been Determined)

(outlined in **6.1.5 [2]** ②)

Item	Reference value
Ammonia nitrogen	0.03 mg/L
Residual chlorine (oxidant residual)	Not present
Hydrogen sulfide	Not present
Copper	Not present
Aluminum	0.1 mg/L
Iron	0.2 mg/L
Anionic surfactant	Not present
Nonionic surfactant	Not present
Benzo(a)pyrene	0.00001 mg/L
Tributyltin compound	0.000002 mg/L
Triphenyltin compound	Not present
Phenolic compound	0.2 mg/L
Formaldehyde	0.04 mg/L

Table 6.1.17 Water Quality Standards for Fisheries (Sediments in Sea Areas)

(outlined in **6.1.5 [2]** ②)

For dried mud in sea areas, the COD_{OH} (alkaline process) should be 20 mg per 1 g of dried mud or less, the sulfide should be 0.2 mg per 1 g of dried mud or less, and the normal-hexane extract should be 0.1% or less.
The settling, development and growth of seeds and seedlings should not be hindered by fine suspended solids on rock surfaces, stones and gravel.
Among the hazardous substances obtained in the dissolution tests carried out in accordance with the Japanese Act on Prevention of Marine Pollution and Marine Disaster (Notice No. 14 issued by the Japanese Environment Agency on February 17, 1973), for substances for which the reference values have been determined in the Water Quality Standards for Fisheries, the values should be lower than 10 times the reference values. For cadmium and PCB, the concentration in the dissolution tests should be lower than the lower detection limit of each compound.
The concentration of dioxins should be lower than 150 pg-TEQ/g.

Table 6.1.18 Quality Criteria for Bathing Beaches

(outlined in **6.1.5 [2]** ③)

Type \ Item		Number of colon bacillus	Presence of oil film	Chemical oxygen demand (COD)	Transparency
		Number/100 mL	—	mg/L	m
Good	AA	Not found (Detection limit: 2)	Not found	2 or less (Lakes: 3 or less)	Clear (or more than 1)
	A	100 or less	Not found	2 or less (Lakes: 3 or less)	Clear (or more than 1)
Satisfactory	B	400 or less	Found at times	5 or less	0.5 to 1
	C	1,000 or less	Found at times	8 or less	0.5 to 1
Unsatisfactory		More than 1,000	Found consistently	Over 8	0.5 or less*

Note: Classification of each bathing beach is based on the average of the values obtained at that beach during the survey

For transparency indicated with an asterisk, clouding caused by stirred-up sand is excluded from assessment.

Table 6.1.19 Provisional removal standards for sediments

(outlined in **6.1.5 [2]** ④)

Item	Provisional removal standards for sediments	
	Sea areas	Rivers and lakes
Mercury	$C=0.18\times\frac{\Delta H}{J}\times\frac{1}{S}$ C: Provisional removal standards for sediments (ppm) ΔH: Average tide range in the sea area (m) J: Elution rate of mercury from sediments S: Safety factor*	25 ppm Applicable to sea areas with strong longshore currents
PCB	10 ppm	

* The S (safety factor) in the table is any of 10, 50 or 100 based on actual fishing in the water area and around it.

Table 6.1.20 Bottom sediment and soil criteria

(outlined in **6.1.5 [2]** ⑤)

Item	Reference value	Item	Reference value
Alkyl mercury compound	Not present	Vanadium or its compound	1.5 mg/L or less
Mercury or its compound	0.005 mg/L or less	Organochlorine compound	40 mg/kg or less
Cadmium or its compound	0.1 mg/L or less	Dichloromethane	0.2 mg/L or less
Lead or its compound	0.1 mg/L or less	Tetrachloromethane	0.02 mg/L or less
Organophosphorus compound	1 mg/L or less	1,2-dichloroethane	0.04 mg/L or less
Chromate compound	0.5 mg/L or less	1,1-dichloroethylene	1 mg/L or less
Arsenic or its compound	0.1 mg/L or less	Cis-1,2-dichloroethylene	0.4 mg/L or less
Cyanogen compound	1 mg/L or less	1,1,1-trichloroethane	3 mg/L or less
Polychlorobiphenyl	0.003 mg/L or less	1,1,2-trichloroethane	0.06 mg/L or less
Copper or its compound	3 mg/L or less	1,3-dichloropropene	0.02 mg/L or less
Zinc or its compound	2 mg/L or less	Thiuram	0.06 mg/L or less
Fluoride	15 mg/L or less	Simazine	0.03 mg/L or less
Trichloroethylene	0.3 mg/L or less	Thiobencarb	0.2 mg/L or less
Tetrachloroethylene	0.1 mg/L or less	Benzene	0.1 mg/L or less
Beryllium or its compound	2.5 mg/L or less	Selenium or its compound	0.1 mg/L or less
Chromium or its compound	2 mg/L or less	1,4-dioxane	0.5 mg/L or less
Nickel or its compound	1.2 mg/L or less	Dioxins	10 pg-TEQ/L or less

6.2 Water Quality Investigation

6.2.1 Properties of Water Quality Environment

(1) Variation Factors of Water Quality

The substances contained in seawater move and spread due to variation factors originated from the outside (e.g., tides, weather conditions and the inflow of rivers). Therefore, even when investigation are carried out at identical locations, their concentrations change temporally, which means the properties may change day by day or hour by hour. In addition, in some cases, the locality is shown in the spatial distribution, i.e., the water quality significantly changes temporally and spatially. Considering the investigating objectives, it is important to determine the temporal and spatial scales to understand the phenomena.

Fig. 6.2.1 illustrates the flow spectra of a sea area. The x-axis and y-axis are the period and wavelength of the flow, respectively, and, the z-axis is the relative energy. In port areas, the energy of the waves (1 in the figure) and swells (2 in the figure) which change over a period of several seconds in the temporal scale, with a wavelength of several meters in the spatial scale, is high. The energy of the tides (8 and 9 in the figure) which change over a period of half a day to one day in the temporal scale, with a wavelength of several kilometers in the spatial scale, is high, too.

Between them, there are variations over a period of tens of minutes in the temporal scale, with a wavelength of dozens to hundreds of meters in the spatial scale (3 and 5 in the figure), orographic eddies (6 in the figure) based on the nonlinearity of the tidal current, and over-tide currents (7 in the figure). There are also wind-driven currents (11 in the figure), tidal residual currents corresponding to the periods of the spring tide and neap tide (12 in the figure), and density currents (13 in the figure). The water quality may change due to these influences with the temporal and spatial scales, to achieve the intended results, it is important to determine the times, locations and other necessary matters considering the aforementioned variation factors.

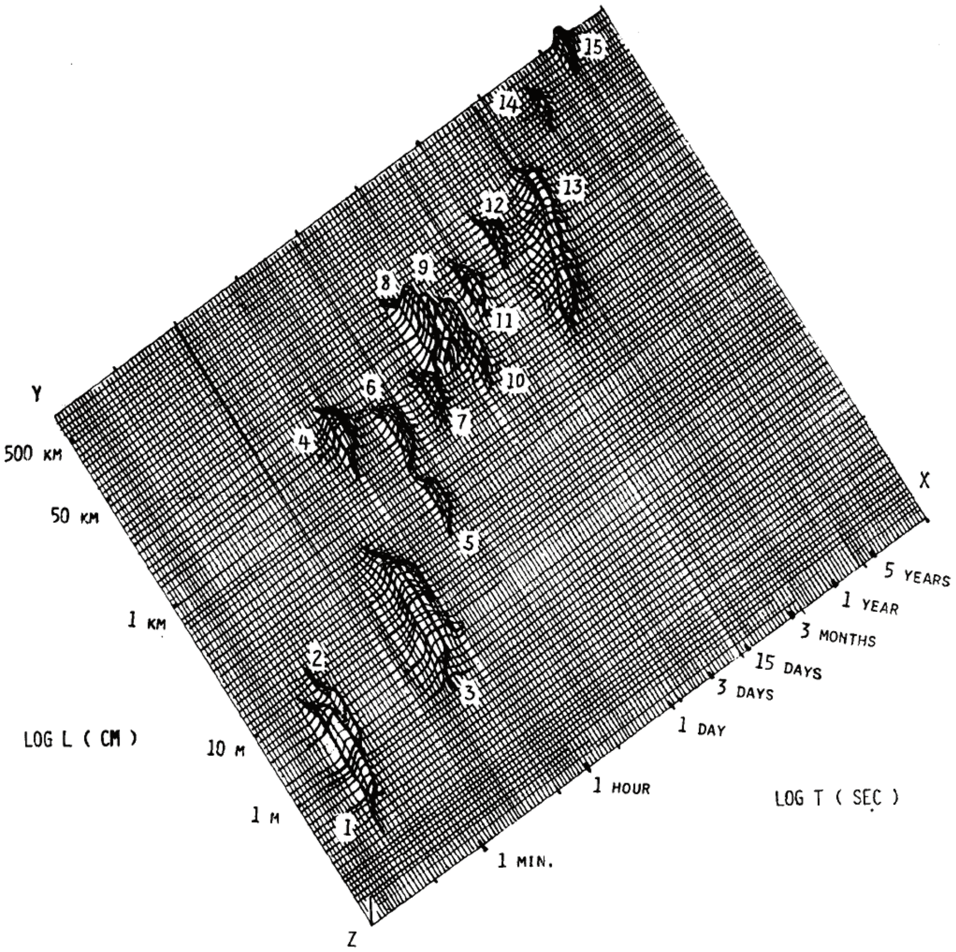

Fig. 6.2.1 Spectra of Variations in Ocean Conditions[12)]

In addition, different water masses (seawater) mix together in port areas due to the influence of the river water from the land and the adjacent ocean current from the sea. Therefore, current junctions are sometimes seen in the horizontal direction, and it is known that the water qualities are completely different between the sides of these junctions. On the other hand, in the vertical direction, the salinity in the river water is different from that in the seawater, and the water temperature of the surface-layer is different from that of the lower layer due to sunlight. Because of the density differences, thermocline would be developed. Furthermore, the water qualities are different above and below the thermocline.

While going to the site planned for investigation, it is important at all times to pay attention to the color of the sea surface and notice how much waste has gathered, in order not to overlook the characteristics of the sea area (e.g., current junctions). When the water temperature and salinity in the vertical direction can be checked at the actual site using a direct reading type water quality meter, or sediments are checked by divers combinationally, differences in the water temperature, salinity, muddiness and other items between the upper and lower layers can be clarified. Therefore, the differences in the water masses need to be understood in advance. For sea areas where water masses are locally different in the horizontal and vertical directions, it is desirable that the number of investigation sites for sampling be increased beforehand. Understanding the characteristics of these water masses is useful when interpreting and evaluating the investigation results.

6.2.2 Analysis Items and Methods

(1) Analysis Items

The items for analyzing water quality vary depending on the objectives of "the environmental investigation to assess the impacts of business projects on the environment" and "environmental investigation in marine environment improvement projects", as described in **Reference [Part II], Chapter 1, 6.1.1 Objectives**.

For "the environmental investigation to assess the impacts of business projects on the environment", the items applied to check and assess whether the construction has an impact should be considered and selected. For "environmental investigation in marine environment improvement projects", the items applied to check and evaluate the habitats of living things should be considered and selected.

Examples of analysis items necessary to check and assess whether the construction has an impact include the chemical oxygen demand, total nitrogen, total phosphorus and the quantities of suspended solids, for which the reference values are specified in the environmental standards and other standards. For the habitats of living things, once the organisms that live in an improved environment start inhabiting the area and breed, it can be said for the first time that the final effects have been demonstrated. For the breeding of organisms, the appropriate nutrient salts should circulate among the water quality, sediments and organisms. Therefore, the analysis items should also be discussed considering the circulation of substances.

In addition, when numerical simulations are designed to predict the spread of muddiness due to construction and environmental changes in the habitats of living things, and, when the impacts of business projects or effects of the environment improvement projects are to be discussed beforehand, the items which are necessary to the construction of these numerical simulations should be investigated.

According to the opinions narrated above, **Table 6.2.1** lists the water quality analysis items to be considered and determined based on the investigation objectives, and items to be measured at the actual site. For the selection of the analysis items and other matters, refer to "**Marine Investigation Technical Manual: Water Quality and Sediment Investigation**".[7)]

For example, water quality pollution is generally divided into two types: organic contamination originating in domestic waste water (impacts on nature conservation, habitats for aquatic life, people's daily life, water for factories, etc.) and pollution due to hazardous substances originating in industrial wastewater (impacts on human health and aquatic life). Examples of water quality investigation items include physical characteristics (water temperature, salinity, transparency, turbidity and suspended solids [SS]), chemical characteristics (pH, nutrient salts [nitrogen and phosphorus], dissolved oxygen [DO] and chemical oxygen demand [COD], etc.) and biological characteristics (chlorophyll a and pheo-pigments, etc.).

Table 6.2.1 Types of Analysis Items in Water Quality Investigation

Type	Viewpoint for selection		Analysis and measurement items for water quality
Water quality analysis items	Environmental investigation to assess the impacts of projects on the environment	Items for which standards are provided in laws and regulations	Substances for which environmental standards are specified, in general
	Environmental investigation in marine environment improvement projects	Indexes for the circulation of substances	Nitrogen, phosphorus, chlorophyll a, etc.
		Environmental indexes for environmental preservation and nature restoration plans, etc.	Transparency for the growth of seaweed and sea grass, DO of the bottom layer for the inhabitation of living things, etc.
	Common	Conditions of numerical simulations	Water temperature, salinity, COD, nitrogen, phosphorus, chlorophyll a, etc. (information of flow conditions, topographic features, sediments, etc., is required, in addition to water quality)
Items to be measured at actual sites	Common	Usage of field measuring instruments	The basic items showing the water properties (e.g., water temperature, salinity, transparency and turbidity) should always desirably be measured in water quality investigation.

(2) Analysis Methods

① **General analysis**

The analysis item types are as shown in **Table 6.2.1**. As a rule, when analyzing these items, the official methods specified in the JIS standards and various manuals should desirably be used. For the main analysis methods, refer to the **Common Specifications of Port Designs, Measurements and Investigation**[13] and the environmental standards, laws and regulations on the website of the Japanese Ministry of the Environment (https://www.env.go.jp/law/index.html).

② **Usage of field measuring instruments**

In water quality investigation, a main investigation and analysis method is the water sampling analysis (water sampling at the actual site and analysis in a laboratory), which excludes items measured at the actual sites (e.g., water temperature and salinity). Meanwhile, using a field measuring instrument makes it possible to measure the vertical distribution of multiple water quality items at one time, allowing the vertical distribution characteristics of the water quality to be understood in detail. In addition, obtaining data with fine temporal and spatial resolutions makes it possible to understand the present environmental situations in a more appropriate way, and is helpful for verifying the appropriateness of the calculations which reproduce the present conditions in the numerical simulation. Therefore, to carry out a water quality investigation effectively and efficiently, it is important to determine the items to be measured at the actual site using field measuring instruments and the items to be analyzed by water sampling analysis. **Table 6.2.2** lists typical field measuring instruments. It is necessary to understand the equipment characteristics, their measurement accuracy, points to note for use, whether the equipment needs to be calibrated and other necessary matters before use. To find more about the field measuring instruments, refer to the **Marine Investigation Technical Manual: Water Quality and Sediment Investigation**.[7]

Table 6.2.2 Typical Field Measuring Instruments for Water Quality Investigation

Measuring instrument type	Investigation items	Outline
Water temperature, salinity and depth gauge (CSTD and CTD)	Water temperature, salinity and depth	Can measure the salinity (electric conductivity), water temperature and water depth at the same time.
Hydrogen ion densitometer (pH meter)	pH	Two types are available: one that makes measurements by putting a sensor into sampled water, and one named hanging down style using a long cable.
Dissolved oxygen analyzer (DO meter)	Dissolved oxygen (DO)	The hanging down type allows the vertical distribution to be understood in real-time, thus it is useful to determine if there exists a deficient oxygen state and take quick action.
Turbidimeter	Turbidity	An instrument which measures suspended solids in seawater using an optical method (transparent spectrophotometry or scattering spectrophotometry). There are stationary and hanging down types. Muddiness is an important factor for which countermeasures should be taken in port construction works and other similar projects. Measurement with a turbidimeter is often performed as a part of construction management. The standard for the muddiness of water quality is shown as SS. Understanding the relationship between the turbidity and SS in advance using suspended solids at the actual site makes it easier to compare the measured values to the criteria.
Chlorophyll meter	Chlorophyll	An instrument which measures the intensity of fluorescence based on the concentration of chlorophyll contained in the phytoplankton using a blue monochrome light-emitting diode.
Multi-item water quality meter	Combined aforementioned investigation items	Typical water quality meters can measure water temperature, salinity, turbidity, hydrogen ion concentration (pH), dissolved oxygen (DO), chlorophyll a and depth.

6.2.3 Assessments

(1) Comparison to the Standards

The data obtained through field investigation can be assessed by comparing them to the environmental standards and other standards. Various standards and concepts for assessments are described in **Reference [Part II], Chapter 1, 6.1.5 Interpreting and Assessing Investigation Data**. In addition to comparisons to various standards, a statistical analysis can be used to check the variations over the years and calculate significant differences.

(2) Numerical Simulations

As a technique to predict and assess the impacts and effects on water quality, numerical simulation is used, in addition to comparison to the standards. The impacts of a business project on water quality and living things can be predicted and assessed by predicting the water quality during the period of construction work and the time of water-service, using numerical simulations and comparing the results to the environmental standards. An outline of numerical simulation is shown below.

① **General**

At the time numerical simulations were first introduced to water quality predictions, conservative models were more commonly used. However, if the prediction targets were non-conservative substances (e.g., nitrogen and phosphorus), these models were unable to reproduce the state of eutrophic sea areas. In 1993, nitrogen and phosphorus were added as items to which environmental standards were set, the necessity to predict the nitrogen and phosphorus arises up to that point, COD was the main factor in water quality predictions for port projects. Furthermore, as intensive observations and experiments conducted in eutrophic sea areas (e.g., Tokyo Bay, Mikawa Bay and Osaka Bay) clarified the ecosystem mechanisms, non-conservative ecosystem models that consider internal production began to be used.

Resource renewal models and ecosystem models that consider water quality, sediments and the movement of substances between living things are used for predictions in environmental impact assessments for port projects. They are also used to plan marine environment improvement projects (e.g., the reproduction of seaweed beds and tidal flats) and assess their effects.

② **Outline of numerical simulations**

There are several types of numerical simulations: a simple model in which the sea area is regarded as an assembly of simple boxes, where the balance of the flow rate and water quality between the boxes is viewed; a flow model in which the sea area is divided into small grids, where equations of motion and continuity which express the movement of the seawater between the grids are solved; and a model in which the flow model is combined with the water quality model, where advection and the spread of water quality are calculated. In recent years, lower-order ecosystem models that consider the growth and death of microorganisms (e.g., plankton and bacteria), benthic organisms have been developed and used for environmental impact assessments including predictions.

The advantages of numerical simulations are that they can make predictions for any future topographic or environmental condition and can quantitatively express water quality that changes in a complicated way in temporal and spatial scales. However, various simulation conditions to be used for the simulation are determined from the field survey results, sometimes it is difficult to collect all the conditions that are required for the simulation. It should be noted that the simulation results do not completely reflect the natural phenomena at the actual site. In addition, numerical simulations cannot predict all of the natural phenomena, each model has its advantages and disadvantages. **Table 6.2.3** lists the advantages and disadvantages of various types of models. For an outline of the numerical simulation to be used for environmental impact assessments, refer to the **Environmental Impact Assessment Guidebook in the Port Sector 2013**.[1] The **Guidebook for Predicting the Impact of Muddiness in Port Construction**[14] describes predictions of impacts of muddiness in port construction in detail.

Table 6.2.3 (1) Outline of Numerical Simulations
(Revised based on Technologies for Hydrologic and Water Quality Management in Lakes and Marshes[17)])

Model type	Characteristics	Applicable conditions (or suitable conditions)	Simulation targets	Advantages	Disadvantages
Box model	• The water area is divided into multiple boxes in the longitudinal direction, and changes in the water quality in each box due to inflow and outflow are simulated. • Balance is the only item for hydraulic quantity • The water quality is shown as the average of each box.	• The water quality distribution in one box can be regarded as uniform. • Impacts of temporal variations of the low can be ignored to a certain extent.	• Average water quality in each box • Heat exchange on water surface • Material balance (inflow and outflow + sedimentation) • When multiple boxes are used, advection and spreading in the longitudinal direction can be taken into account. • Loads from sediments can be considered.	• Simulations do not take much time. • The long-term water quality can be predicted.	• Not suitable for stratified sea areas since this type of model assumes a mixing of all layers. • Cannot show the water quality distribution in one box. • Cannot take impacts of variations of the flow into account.
Vertical one-dimensional model	• The water area is divided into layers, and the vertical distribution of the hydraulic and water quantities are simulated. • The hydraulic and water quantities are averages of the layers.	In addition to the sea areas to which the box model can be applied: • In relatively small sea areas, the flow and horizontal distribution of water quality can be regarded as uniform • Sea areas with simple shapes	In addition to the targets of the box model: • Vertical distribution of the hydraulic quantity and water quality	• Simulations do not take much time. • The long-term hydraulic quantity and water quality can be predicted.	• Horizontal variations in the water quality cannot be understood. • Difficult to express local phenomena.
Horizontal two-dimensional model	• The water area is divided into a mesh pattern in the horizontal direction, and the distribution of the hydraulic quantity and water quality are simulated. • The hydraulic and water quantities are simulated for each mesh, but the distribution in the vertical direction is regarded as uniform.	In addition to the sea areas to which the box model can be applied: • Sea areas for which the water quality distribution in the vertical direction can be regarded as uniform (e.g., wide and shallow sea areas) • Sea areas with relatively complex shapes (e.g., with an inlet)	In addition to the targets of the box model: • Horizontal distribution of the hydraulic and water quantities	• Simulations are faster than 3-D simulations. • Mid-term (one to tens of years) hydraulic and water quantities can be predicted.	• Not suitable for stratified sea areas because changes in the water quality in the vertical direction cannot be expressed.

Table 6.2.3 (2) Outline of Numerical Simulations
(Revised based on Technologies for Hydrologic and Water Quality Management in Lakes and Marshes[17)])

Model type	Characteristics	Applicable conditions (or suitable conditions)	Simulation targets	Advantages	Disadvantages
Vertical two-dimensional model	• The water area is divided into a mesh pattern in the longitudinal and vertical directions. • The hydraulic quantity and water quality are simulated for each mesh, but the distribution in the transverse direction is regarded as uniform.	In addition to the sea areas to which the vertical one-dimensional model can be applied: • Sea areas with long and narrow shapes, such as a river, for which the water quality distribution in the transverse direction can be regarded as uniform	• Distribution of hydraulic quantity and water quality in the longitudinal and vertical directions	• Simulations are faster than 3-D simulations. • Mid-term (one to tens of years) hydrology • The water quality can be predicted. • Countermeasures to control stratification can be considered.	• Cannot express changes in the water quality in the transverse direction. • Cannot express the flow for which distribution is seen in the horizontal direction (e.g., wind-driven currents).
Three-dimensional model	• The water area is divided into a mesh pattern in the longitudinal, transverse and vertical directions, and the 3-D distribution of hydraulic quantity and water quality are simulated. • 3-D distribution of the hydraulic quantity and water quality can be simulated.	In addition to the sea areas to which the vertical two-dimensional model can be applied: • Sea areas where water quality distributes in horizontal and vertical directions (e.g., sea areas where there is a density current or deep sea areas) • Sea areas with complex planar shapes	• 3-D distribution of the hydraulic quantity and water quality	• Phenomena can be understood in 3-D. • Can express local hydraulic and water quality characteristics. • Can take into account the flows, such as a density current or a wind-driven current. • More complicated countermeasures for the sea areas can be considered.	• Simulations take a significant amount of time since a 3-D mesh division is performed. • Not suitable for mid- or long-term simulations.

③ **Various tests required for numerical simulations**

For numerical simulations of water quality, sediments, ecosystems, bottom mud elution tests, oxygen consumption tests, nitration and denitrification rate tests are sometimes carried out to determine the simulation conditions, in addition to the results of the investigation of present situations. For various tests regarding water quality numerical simulations and the determination of the simulation conditions, refer to **references** 7), 15), 16) and 17). An outline of the bottom mud elution tests and oxygen consumption tests is described in **Reference [Part II], Chapter 1, 6.3 Sediment Surveys**.

④ **Examples of numerical simulations**

Numerical simulations have been performed in many projects, including the development of port facilities and marine environment improvement projects. Some projects have released environmental impact assessment reports or documents of review board concerning the marine environment improvement projects. These documents can be referred to when a numerical simulation is performed. For impact assessments for projects including the development of port facilities, refer to **references** 18), 19) and 20), and for marine environment improvement projects, refer to **references** 21), 22), 23), 24) and 25).

6.3 Sediment Surveys

6.3.1 Characteristics of Sediment Environments

(1) Causes of Changes in Sediments

Characteristics of sediments are that they reflect a long-term environmental history. For example, water quality changes hourly due to various factors such as the tide, weather conditions, hydrographic conditions and river water, and therefore, its properties change hour by hour or day by day in some cases. On the other hand, the properties of sediments are often characterized by the long-term history of environmental characteristics at the site, except for major events (e.g., floods). Specific examples include how a large amount of sediments originating from living things can be found in closed water areas where a large number of plankton is produced; how a large amount of sand and gravel particles are found in areas with strong tidal currents and waves, even though there are few organic sediments; and how there are many sediments, such as those originating in minerals and plant fragments from rivers, which have accumulated in areas near river mouths. Based on these observations, long-term changes in the environment or changes due to events can be estimated by dividing sediments into pillar shapes and analyzing them.

Sediments are affected by the surrounding environment, and inversely affect water quality at the same time. For example, microorganisms promote oxidative decomposition on the sea floor in closed water areas that have a large amount of organic sediments, which causes a lack of oxygen in the bottom water and the elution of nutritive salts, etc., from the bottom mud. Therefore, stratification continues, and in the summer, when nutritive salts flow out from the bottom mud due to a lack of oxygen in the bottom water, the sediments deteriorate. Conversely, in winter, when the top and bottom seawater mixes and there is an abundant amount of dissolved oxygen in the bottom water, the sediments tend to improve. Therefore, it is preferable to measure the water quality (especially the water quality of the bottom layers) in sediment surveys at the same time.

When sediments and an environment where living things live and grow are regarded as an ecosystem, the sediments need to be assessed as a base for the survival and growth of the living things inhabiting that environment. Tidal flats, seaweed beds and shoals are typical ecosystems for the circulation of substances consisting of an inflow of various substances (e.g., nutritive salts and organic substances) from rivers, circulations inside bays and exchanges with offshore areas. In these ecosystems, the habitats of living things are hierarchically associated with the sediments. **Table 6.3.1** lists sediment types in sea areas along with examples of living things that inhabit and grow there. For example, for the growth of Zostera (a type of seagrass bed) and the settlement of Bivalvia represented by littleneck clams, silt and sand are suitable, and changes in the sediments cause significant effects. Therefore, changes in the sediments should be understood in surveys related to benthic organisms, seaweed and sea grasses.

Table 6.3.1 Sediment Types in Sea Areas and Examples of Living Things (Created in Reference to the Natural Environment Assessment Techniques[26])

Type	Deep bays and sublittoral zones (sea areas)				
Basic material (sediment)	Mud	Silt	Sand	Gravel	Reef
Living things that inhabit and grow there	Nereid, bivalves, snails, echinoderms, etc.			Large snails (abalones and Turban shells), Echinoidea, etc.	
	Shellfish beds (Musculista senhousia)	Zostera beds	Zostera beds	Sargassum beds, Laminaria beds, Eisenia beds, etc.	

(2) Spatial Characteristics of Sediments

Sediments have a planar locality, and differences may be great vertically due to changes in the accumulated amounts and long-term quality changes. Example factors that characterize the planar distribution characteristics of sediments are their positional relationships with inflow (load) sources (e.g., rivers and industrial wastewater), environmental gradients (e.g., water depth, tide level, waves and flows), and artificial modifications (e.g., dredging work).

When arranging survey locations, the aforementioned spatial characteristics should be understood first, and grid-like arrangements, priority arrangements, arrangements that consider environmental gradients and other types of arrangements should be considered. For specific examples of arrangements of survey locations, refer to the **Marine Survey Engineering Manual: Water Quality and Sediment Surveys**.[7]

6.3.2 Analysis Items and Methods

(1) General Analysis

Table 6.3.2 lists broadly categorized sediment analysis items. As a rule, it is preferable to use the official methods specified in the JIS standards and various manuals to analyze these items. The **Specifications Common to Port Designs, Measurements and Surveys**[13] organizes the main analysis items and methods.

Table 6.3.2 Analysis Item Types for Sediment Surveys

Type	Analysis item	
Physical properties	Grainsize pattern, density (specific gravity), moisture content, etc.	
General items	Ignition loss, chemical oxygen demand (CODsed), sulfides (T-S), total nitrogen (T-N), total phosphorus (T-P), total organic carbon (TOC), etc.	
Hazardous substances	Metals	Alkyl mercury compounds, cadmium or its compound, lead or its compound, zinc or its compound, etc.
	Organic compounds	Trichloroethylene, dichloromethane, tetrachloromethane, PCB, dioxins, etc.
Other special items	Stable isotopes (e.g., ^{13}C and ^{15}N): Analysis of the sediment origins and clarification of the circulation of the substances by food chain	
	pH: Analysis for projects for improving sediments using recycled materials (materials for improving sediments using steel slag, coal ash and other materials)	
Items to be measured at actual sites	Before conducting a field survey, the actual site should be observed to understand the present situation, in addition to a sediment survey. Examples of general observation items at actual sites are the mud color, appearance, impurities (foreign substances), odor, mud temperature and oxidation-reduction potential (ORP).	

The characteristics and notes for each survey item in the sediment surveys are organized below. For the characteristics and notes (e.g., portioning out, treating, storing and transporting samples) regarding the tools and materials listed in **Table 6.3.3**, refer to the **Marine Survey Engineering Manual: Water Quality and Sediment Surveys**.[7)]

Table 6.3.3 Characteristics and Notes on Tools and Materials for Each Sediment Survey Item

Survey item	Tools and materials	Characteristics	Notes
Mud on the outermost layer	[Onboard work] • Grab sampler • Dredge sampler [Diving work] • Manual grab sampler, trowel, etc.	• For understanding the general environment. • Floating mud on the surface of the sampled mud has been washed away, so it is not in an undisturbed state. • When a dredge sampler is used, the size of the mud area cannot be determined.	• The layer sampled using a grab sampler often varies depending on the sediment (sand or mud), so the sampling method should preferably be adjusted based on the sediment at the survey locations.
Any layer of mud	[Onboard work] • Gravity core-sampler [Diving work] • Acrylic cores, and drivers and vibro-corers to drive cores	• For understanding the history of accumulation and to consider the impacts of dredging. • Cylindrical samples of undisturbed bottom mud are taken. • Any layers from the Gravity core-sampler bottom mud are portioned out for analysis.	• The layers that can be sampled differ between survey methods. Generally, these layers are approximately one meter deep in the case of bare-handed driving, and approximately a few meters when using a vibro-corer. • Sampling from sediment with a lot of gravel, stones and shells is rather difficult.

Survey item	Tools and materials	Characteristics	Notes
Mud from undisturbed outermost layers	[Onboard work] • Gravity core-sampler [Diving work] • Acrylic cores, and drivers and vibro-corers to drive cores [Sample transportation] • Large heatproof containers that can keep samples for each core cool and dark.	• For evaluating the mass transfer between the seabed and bottom water. • Cylindrical samples of undisturbed bottom mud are taken. • Cylindrically-sampled bottom mud is subject to laboratory tests, such as oxygen demand and dissolution tests, without treatment.	• Sampling from sediment with a lot of gravel, stones and shells is rather difficult. • The bottom water and substances that accumulate on the surface of the sediments are important, so samples are transported by keeping them still inside the cores so as not to disturb them. • The bottom water immediately above the sediments is sometimes sampled for use in laboratory tests.
Mud from the surface	[Land work, etc.] • Acrylic cores • Spoons or other similar tools are used to directly sample mud at areas where tidal flats are exposed at low tide	• For surveying fine algae growing on the seabed, sediments, etc. • Undisturbed bottom mud is sampled, and the surfaces are portioned out.	• It is important to block sunlight for samples concerning fine algae to prevent photosynthesis. • When an analysis cannot be performed quickly, a fixative is sometimes used to secure and store the fine algae.

(2) Various Tests

For numerical simulations of water quality, sediments and ecosystems, dissolution and oxygen demand tests may be carried out to determine the simulation conditions. For the specific procedures for tests ① and ② below, refer to the **Marine Survey Engineering Manual: Water Quality and Sediment Surveys**.[7)] For the specific procedures for test ③, **reference 27)** can be referred. For planning, conducting and assessing these types of tests, it is preferable to consult with specialists, when necessary.

① **Bottom mud dissolution tests**

It is understood that the elution of nutritive salts (nitrogen and phosphorus) from sea bottom mud significantly contributes to the water quality pollution in closed water areas (ports, in particular) and shallow sea areas. There are two methods to measure the amount (rate) of elution from bottom mud: an in-situ method, in which a tester is directly installed in the sea bottom mud at the actual site, and a pseudo in-situ method (laboratory test), in which undisturbed cylindrically-sampled mud (generally, with an inner diameter of 200 mm and height of 1,000 mm, and a bottom mud thickness of 30 cm) is carried back to a laboratory for testing. In recent years, the pseudo in-situ method is often used because field operations and management during test periods are easier with this method and technologies have advanced; for example, thermostatic chambers with highly accurate temperature control functions have been developed. Test conditions to be noted in the pseudo in-situ method are the temperature, DO concentration of the water right above the mud, and light. The test conditions should be coordinated with the conditions at the actual site as much as possible based on the survey objectives. The test period is often 10 to 20 days, and the concentration of nutritive salts is analyzed at intervals of a few days to assess the elution amount (rate) based on changes in the concentration over time.

② **Oxygen demand tests**

Organic substances which have accumulated on the sea floor are decomposed by bacteria, and at that time, the dissolved oxygen in the water right above the bottom mud is consumed. Therefore, in water areas where the organic contamination of the bottom mud has progressed, the dissolved oxygen concentration in the bottom layer becomes low (causing a lack of oxygen) and eutrophication is accelerated due to the elution of nutritive salts as a result of the lack of oxygen.

Because of the same reasons as with the bottom mud dissolution tests, oxygen demand tests are often performed in laboratories (laboratory tests) by bringing back cylindrically-sampled undisturbed mud (the general specifications are the same as those for the bottom mud dissolution tests). Test conditions to be noted in the laboratory tests are the water temperature directly above the mud, light and flow. The water temperature directly above the mud (condition setting) should be coordinated with that of the actual site environment as much as possible. Furthermore, as a rule, the light conditions should be dark. This is because the amount of light near the bottom mud at the actual site is minor, and to suppress changes in the water quality due to increases in phytoplankton and other materials in the water directly above the mud. The test period often takes ten days. The dissolved oxygen is measured at intervals of a few days (at intervals of a few hours when organic contamination is severe) and the oxygen demand (rate) is assessed based on the changes over time. During the test, another control test using only the water directly above the mud should be preferably performed for the assessment to understand the oxygen demands from the plankton and bacteria in that water, in addition to that of the bottom mud.

③ **Tidal flat purification function tests**

Various factors are related to the purification function of tidal flats and shallow sea areas. In testing the purification function, the conditions at the actual site should be well understood and reproduced in an experimental system. For experimental systems, there are systems that reproduce actual situations at a laboratory, and systems in which equipment is directly installed at the actual site.

6.3.3 Special Surveys

Special sediment surveys are shown below. For more details, refer to the **Marine Survey Engineering Manual: Water Quality and Sediment Surveys**.[7)] For planning, conducting and assessing these types of tests, it is preferable to consult with specialists when necessary.

(1) Analysis of Chlorophyll in Tidal Flats

Many benthic animals inhabit tidal flats and shallow sea areas, which contributes to the purification function due to the food chain and tilling (burrowing). In addition to these visible organisms, many micro-algae exist on the surfaces of tidal flats and shoals, which are rich in sunlight. These algae become feed for benthic organisms and contribute to the purification function. Analyzing the amount of chlorophyll on the surface of bottom mud makes it possible to understand the functions of these micro- algae.

(2) Surveys of Precipitates from Suspended Solids

In this type of test, substances floating in the water and precipitating on the bottom mud are collected with sediment traps and other similar tools and analyzed. Understanding the properties and sedimentation of suspended solids allows the origins of the suspended solids and sedimentation rates to be estimated, making it is possible to obtain information that is useful to clarify the circulation of substances and pollution mechanisms. Although the analysis items vary depending on the objectives, analyses focusing on carbon and nutritive salts (nitrogen and phosphorus) are often performed in deep bay areas where there are many new sediments in order to clarify the circulation of substances and internal production mechanisms.

(3) Distinguishing Sediments Using Acoustic Exploration

When selecting a location suitable for installing a structure, the basic materials (e.g., mud, sand, stone and bedrock) of the bottom mud need to be understood over a wide area. Thus, distinguishing sediments using acoustic exploration is useful. Surveys using a side-scan sonar, which is one type of acoustic exploration, are similar to photographing the sea floor using sound waves. Recently, high-resolution side-scan sonars were put to practical use, and their application range has been expanding in order to check and understand underwater port structures and the distribution of seaweed beds, in addition to distinguishing the basic materials of the bottom mud. These sonars can also be applied to distinguish the basic materials of the bottom mud efficiently over a wide range in deep-water zones.

6.3.4 Assessments

(1) Comparisons to Standards

Data obtained at actual sites can be assessed by comparing it to the environmental standards and other standards. The various standards and concepts for each assessment are shown in **Reference [Part II], Chapter 1, 6.1.5 Interpreting and Assessing Survey Data**. In addition to comparisons to the various standards, statistical analyses can be used to see changes over time and calculate significant differences.

(2) Use of Analysis Models

An analysis model for sediments is rarely assessed alone. Assessments are performed for combinations of water quality and ecosystems, such as ecosystem modeling, in many cases (refer to **Reference [Part II], Chapter 1, 6.2.3 Assessments** and **6.4.3 Assessments**).

① **General**

Refer to **Reference [Part II], Chapter 1, 6.2.3 Assessments** and **6.4.3 Assessments**.

② **Outline of models**

Refer to **Reference [Part II], Chapter 1, 6.2.3 Assessments** and **Reference [Part II], Chapter 1, 6.4.3 Assessments**.

(3) Examples of Analyses

For actual cases using typical analysis models in environmental surveys to assess the impacts of projects on the environment and environmental surveys in marine environment improvement projects, refer to **Reference [Part II], Chapter 1, 6.2.3 Assessments** and **Reference [Part II], Chapter 1, 6.4.3 Assessments**.

6.4 Surveys of Living Things

6.4.1 Characteristics of Coastal Organisms

(1) Composition of Coastal Organisms

Organisms that live in shallow sea areas include plankton (organisms that float in the water), benthic organisms (periphyton and organisms that survive by attaching onto or burrowing into rocks, bottom mud and other sediments), and nekton (organisms with strong swimming capabilities that live underwater). There are many types of organisms that have floating, benthic and swimming stages in their life history, from birth to adulthood. The quantity and composition of such underwater life demonstrate the environmental characteristics of the sea areas. Certain types of living things have a temporary floating larval stage in their life history; e.g., many types of shellfish. The movement and spread of larvae in the floating stage greatly affects the habitat distribution of their populations, so floating larvae have been actively observed in recent years. These larvae have been getting attention in regard to tidal flat networks, in particular, and the spread of floating larvae has been actively simulated.

Coastal organisms have many temporal and spatial variations, so surveys and analyses should be carried out after understanding their characteristics. **Table 6.4.1** lists typical types of living things that are surveyed and their characteristics.

Table 6.4.1 Characteristics of Growth and Survival of Species in Typical Surveys of Living Things[28), 29), 30)]

Survey type	Survey target	General characteristics of survival and growth
Plankton survey	Phytoplankton Zooplankton	• Their growth phase is from spring to fall. • Because they float, they are affected by the strength of the oceanic water, coastal water, river water and ocean currents.
Benthic organism survey	Megalobenthos Macrobenthos Meiobenthos	• The number of varieties increases in spring. • Because they do not move around much, their biotas rather easily reflect the characteristics of the sea areas and the differences in sediments and submarine topography. • There are large numbers of benthic organisms to a depth of approximately 20 meters.
Periphyton survey		• The types and quantities are larger in summer and smaller in winter. • Their quantities are large from the intertidal zone to sublittoral zone.
Roe, fry and larval fish survey		• Many types lay eggs in two seasons (from spring to summer and fall). • For floating roe, many types lay eggs at night. • Fry and larval fish have motility, so they move to the surface at night due to diurnal vertical migration. • Some fish species produce adhesive eggs and demersal eggs. Some fry and larval fish inhabit the intermediate and bottom layers. • There are fish species that inhabit only certain locations (e.g., coastal surf zones).

Survey type	Survey target	General characteristics of survival and growth
Seaweed bed survey	Seaweed and sea grasses	• The life history (e.g., growing season) varies depending on the types of seaweed and sea grasses and the area. • Many types of seaweed and sea grasses grow from spring to early summer. • The growing season tends to come early in southern Japan and late in northern Japan.
Fish and shellfish survey	Pelagic fish Benthic fish and shellfish	• The swimming ability of pelagic fish is generally strong, and many species have large migration ranges. However, it is said that bottom fish do not move around very wide ranges and they have strong sedentary characteristics.
Seaside bird survey	Scolopacidae, Charadriidae, Anatidae, etc.	• The seasonal behavior of these birds basically involves migration in spring from March to May, a breeding season from May to July, migration in fall from August to October, and hibernation from December to February. • The time period in which there is the greatest number of birds in flight is at early morning and at low tide.
Seaside plant survey		• Most varieties grow the most in early summer. • The habitat range is a zonal distribution based on the elevation. • Some seaside plants on the Pacific Ocean-side wither due to massive waves from typhoons.

(2) Changes in Coastal Organisms

Many coastal organisms that are observation targets have growth and death phases and quantities that vary by season. Plankton move around due to the flow of the water, so the population density changes depending on the hydraulic conditions. The quantities of observed organisms often return to the same amount in the same season every year, although there is some seasonal fluctuation. A state where short fluctuations occur but remain in a steady state over the long term is called dynamic equilibrium.

The accumulation of phenomena that affect the survival of living things controls their state in that area. As with sediments, coastal biotas reflect the long-term environmental history, which includes actions from other living things.

In closed sea areas, the higher the concentration of nutritive salts, the greater the number of organisms and species. However, if the concentration of nutritive salts becomes too high (eutrophication), the amount of diversity will decrease and the number of particular organisms will increase. If the amount of nutritive salts further increases, along with the water temperature and sunshine increase in summer, phytoplankton will have an explosive increase, causing a red tide. Then, the phytoplankton die and settle, having been subjected to the decomposition actions of microorganisms on the sea floor. At that time, the dissolved oxygen in the bottom layer is consumed, causing a lack of oxygen, which decreases the number of benthic organisms.

(3) Distribution of Coastal Organisms

There is an environmental gradient along the coast from the waterside to the offing, forming an ecotone. Coastal organisms are distributed according to the environmental gradient. For environmental gradients in the direction from the coast to the offing, there is a distribution of salt content, water temperature, concentration of nutritive salts and muddiness. For environmental gradients in the depth direction, there are changes in the water level due to the tide and the distribution of force by incoming waves, volume of splashing waves, intensity of sunlight and water temperature.

The most predominant types of plankton change depending on the salt content. Periphyton have a horizontal distribution in the direction from the coast to the offing and vertical distribution (lateral-striped band structure) in the water depth direction. The habitats of benthic organisms inhabiting bottom mud are divided based on factors such as the grain size of the bottom mud and the content of organic substances. The habitats of periphyton are divided depending on the areas they use as a base. In eutrophic deep bays, oxygen in the bottom layer water is poor in summer, which may significantly affect benthic organisms. Depending on the reduced level of oxygen, or the anaerobic level of the bottom mud, anoxia-tolerant organisms (pollution indicator species) become more pronounced. The distribution of highly anoxia-tolerant organisms can be used to predict the sediment environment.

(4) Objectives of Surveys of Living Things in Ports

Surveys of living things in ports are broadly divided into: 1) environmental surveys to assess the impacts of projects on the environment; and 2) environmental surveys in marine environment improvement projects. The survey plans should be formulated in consideration of the various survey objectives.

For environmental impact assessments in port facility development projects, the assessment items are animals, plants and ecosystems. Examples of items to be surveyed regarding animals and plants are the states of the faunae, flora and vegetation, distribution of important species and communities, their survival and growth, and distribution of notable habitats. The important species here are selected from the protected species of animals and plants specified in the Japanese Act on Protection of Cultural Properties and ordinances for cultural properties protection issued by local governments; rare and wild animals designated in the Japanese Act on Conservation of Endangered Species of Wild Fauna and Flora; and species listed on the **Red List by the Japanese Ministry of the Environment** (a list of endangered species of wild animals).[31] The species are also selected based on the **Data Book on Rare and Wild Aquatic Life in Japan**,[32] **Documents for the Selection of Rare Species** (Red Data Books) **Issued by Local Public Bodies**, and other standards.[1] In regard to the ecosystems, by focusing on notable species extracted based on their prominence (a higher rank in the food chain of their ecosystem), typicality (species that show typical characteristics of the ecosystem in the region), distinctiveness (species that work as indexes showing that an environment is unique), circulations of substances in the ecosystems, and purification function can be assessed.[1] For details, refer to the **Technical Guidance on Environmental Impact Assessment – Ecosystems**.[33] **Table 6.4.2** lists examples of notable species extracted based on their prominence, typicality and distinctiveness.

For predictions and measurements of the effects of environment improvement projects, in addition to selecting an increase in the diversity and quantities of living things as goals, species that can be improved can also be selected. In recent years, ecosystem services are used as an index, and a series of surveys of living things and related surveys necessary to measure this index are carried out in some cases. For details, refer to the **Marine Ecosystem Survey Manual: Outline and Implementation**.[34]

Table 6.4.2 Example Selection of Notable Species[1]

Ecosystem / Item	Tidal flats	Seaweed beds (Zostera beds, Sargassum beds and submarine forests)	Coral clusters (coral reefs)
Prominence	• Birds that feed at tidal flats (e.g., Calidris alpine and Charadrius alexandrinus) • Piscivorous fish (e.g., Lateolabrax japonicus), etc.	• Piscivorous fish (e.g., Lateolabrax japonicus, Paralichthys olivaceus and Sebastes inermis), etc.	• Piscivorous fish (e.g., Plectropomus leopardus and Blue emperor) • Other animals (e.g., Sepioteuthis lessoniana), etc.
Typicality	• Fish that settle in tidal flats (e.g., Favonigobius gymnauchen and Acanthogobius) • Fish that inhabit tidal flats when young (e.g., Kareus bicoloratus and Hexagrammos otakii) • Benthic animals that widely distribute in tidal flats and become feed for higher-ranking species (e.g., Marcophthalmus japonicus, Ruditapes philippinarum, Batillaria multiformis and Nereis), etc.	• Sea grasses that form Zostera beds (e.g., Zostera marina and Nanozostera japonica) • Seaweed that forms Sargassum beds (e.g., Sargassum and Sargassum filicinum) • Seaweed that forms submarine forests (e.g., Laminaria japonica, Eisenia bicyclis and Ecklonia cava) • Animals that inhabit seaweed beds (e.g., Sebastes inermis, Hexagrammos otakii, squids, abalones and Echinoidea), etc.	• Reef-building corals that form coral reefs (e.g., deer-horn coral and Poritidae) • Animals that inhabit coral reefs (e.g., Scaridae, Chromis notata, Echinoidea and Holothuroidea), etc.

Type / Item	Species name, etc.	Reasons, etc.
Distinc-tiveness	Phacelurus latifolius communities	Large emergent perennial plants that grow gregariously at the water near river mouths and seacoasts around Japan. Hygrophytes in brackish waters that grow in sandy coastal wetlands. The growth environment is established based on a delicate balance between the land and sea areas.

Type / Item	Species name, etc.	Reasons, etc.
	Aster tripolium	Distributed from Hokkaido to Kyushu. Winter annual plants that grow gregariously in coastal wetlands. Hygrophytes in brackish waters. The growth environment is established based on a delicate balance between the land and sea areas.
	Periophthalmus cantonensis	Survives by strongly relying on tidal flats in brackish waters. Found only in limited areas.

6.4.2 Analysis Items and Methods

(1) Analysis Items and Methods

For surveys of living things, except in cases for observing and recording the growth and survival of living things through field observations, data on the organisms can be obtained by identifying and measuring samples taken at actual sites. Although the analysis items and accuracy for the samples significantly vary depending on the survey objectives, the basic operations that are required are identifying the species, counting each classified species, and weighing, measuring and compiling each taxonomic group.

Table 6.4.3 lists the main analysis items in typical surveys of living things. The sample analysis should conform to the **Specifications Common to Port Designs, Measurements and Surveys**.[13] In addition, refer to the **Marine Survey Engineering Manual: Marine Organism Surveys**[28] and the **Manual on Oceanographic Observation**.[35]

Table 6.4.3 Main Analysis Items and Methods for Samples of Typical Surveys of Living Things[13), 28), 35)]

Survey type	Survey targets	Main analysis items	Main analysis methods
Plankton survey	Phytoplankton	Identification of collected species Counting of cells for each species	Biological microscope
		Measurement of the quantity of chlorophyll a (existing quantity of phytoplankton)	Absorption method (spectrophotometer) Fluorescence method (fluorophotometer)
	Zooplankton	Identification of collected species Counting of populations for each species	Stereo microscope and biological microscope
		Deposition measurement	Settling using sedimentation tubes
Benthic organism survey	Megalobenthos Macrobenthos Meiobenthos	Identification of collected species Counting of populations for each species Measurement of wet weight (dry weight) for each species Measurement of the bodies of useful and major living things	Stereo microscope and biological microscope Balance
Periphyton survey		Identification of collected species Counting of populations for each species Measurement of wet weight and capacity for each species Measurement of the bodies of major living things	Stereo microscope and biological microscope Balance and cylinder
Roe, fry and larval fish survey		Identification of collected species Counting of populations for each species	Stereo microscope, birth test and feeding trial (unknown specimens)

Survey type	Survey targets	Main analysis items	Main analysis methods
Seaweed bed survey	Seaweed and sea grasses	Identification of collected species Counting of populations for each species Measurement of the length of each section of leaf body and counting Measurement of wet weight (dry weight) for each species	Visual determination and stereo microscope Tape measure Balance
Fish and shellfish survey	Pelagic fish Benthic fish and shellfish	Identification of collected species Measurement of body length and weight	Visual determination Tape measure and balance

(2) Sampling Methods

Target organisms are always changing temporally and spatially depending on the species, growth stage of each species as part of its life history, growth and habitat environments, and other factors. Therefore, it is desirable to collect samples that are most typical for each survey location to the extent possible. In addition, sampling methods and equipment that suit the survey objectives and target species should be selected in the survey plans. In field surveys, the sampling equipment should be operated so that the sampling method is properly carried out and the equipment performs properly. Furthermore, the collected samples should be appropriately treated at the site (e.g., fixation using a formalin solution), and the samples should be handled so that later analyses can be properly performed.

Table 6.4.4 lists the main sampling methods for typical surveys of living things. The sampling and fixation of the specimens should conform to the **Specifications Common to Port Designs, Measurements and Surveys**.[13)] In addition, refer to the **Marine Survey Engineering Manual: Marine Organism Surveys**.[28)]

Table 6.4.4 Outline of Sampling Methods for Typical Surveys of Living Things[13), 28)]

<table>
<tr><th>Survey type</th><th>Survey targets</th><th colspan="2">Main sampling methods</th><th>Notes</th></tr>
<tr><td rowspan="2">Plankton survey</td><td rowspan="2">Phytoplankton
Zooplankton</td><td colspan="2">Method using water-quality samplers</td><td>• Pay attention to the speed at which the water-quality sampler is raised and lowered for accurate water sampling.</td></tr>
<tr><td colspan="2">Method using plankton nets
(The filtration rate and volume of filtrated water is measured with a flow meter.)</td><td>• Properly consider the net type, structure, netting type, seine method (vertical pulling, inclined pulling and horizontal pulling [stratification]), pulling distance, pulling time, pulling speed and other factors depending on the survey objectives and target organisms.</td></tr>
<tr><td rowspan="2">Benthic organism survey</td><td rowspan="2">Megalobenthos
Macrobenthos
Meiobenthos</td><td>Tidal flats and shallow sea areas</td><td>Method using quadrats for sampling by divers, etc.</td><td rowspan="2">• Select sampling equipment based on the characteristics of the target sea areas, living species and species groups.</td></tr>
<tr><td>Offing</td><td>Megalobenthos: Towed dredges, small dragnets, etc.
Macrobenthos: Grab samplers, etc.
Meiobenthos: Core samplers, grab samplers, etc.</td></tr>
</table>

Survey type	Survey targets	Main sampling methods	Notes
Periphyton survey		Method in which organisms are scraped from the basic material on which they have attached (A quadrat is placed on the material and a scraper is used to remove all the organisms that have attached to the material in the quadrat.)	• Collect carefully so as not to damage the organisms. • Before sampling, take photographs, including of the quadrat (the conditions of the periphyton can be evaluated in later analyses).
Roe, fry and larval fish survey		Method using a net for collecting roe, fry and larval fish (The filtration rate or volume of the filtrated water is measured with a flow meter.)	• Properly consider the net type, structure, netting type, seine method (e.g., vertical pulling, inclined pulling and horizontal pulling [stratification]), pulling distance, pulling time, pulling speed and other factors depending on the survey objectives, development periods of the target fry and larval fish, and the quantities to be collected.
Seaweed bed survey	Seaweed and sea grasses	Method using quadrats for sampling by divers, etc. (A quadrat is placed on the sea floor and all the seaweed and sea grasses within the quadrat are collected, including their roots.)	• Before sampling, take photographs, including of the quadrat (the conditions of the seaweed and sea grasses can be evaluated in later analyses).
Fish and shellfish survey	Pelagic fish Benthic fish and shellfish	Fish surveys at fish markets, test fishing gear operation surveys, floating seaweed surveys, surveys using fish-luring lights at night, beached fish surveys, etc.	• There is no standard or commonly used sampling equipment. Fishing gear and methods that are used in actual fishing are often used in line with the survey objectives and targets.

(3) Visual Observation

For some target types in surveys of living things, information on the organisms is collected through visual observations (field observations). **Table 6.4.5** lists survey methods by visual observation. For survey methods regarding coastal organisms, refer to the **Marine Survey Engineering Manual: Marine Organism Surveys**.[36)] For bird and plant surveys, refer to the Seacoast Surveys in **Technical Criteria for River Works: Practical Guide for Methods of Investigation**.[37)]

Typical survey methods for visual observations in surveys of living things are quadrat and transect methods. In the quadrat method, a quadrat (square frame) is placed on the habitat of the organisms, and the species that appear in the frame, number of each species, cover degree and other items in the quadrat are surveyed. The size of the quadrat varies depending on the target organisms and objectives, and ranges from tens of centimeters to several meters. In the transect method, in a range partitioned on a belt along the measuring line set in the target sea area, the species that appear in the range, number of each species, cover degree and other items are surveyed. Both methods can be combined by placing multiple quadrats on a belt.

Table 6.4.5 Survey Methods in Field Observations[36), 37)]

Survey type	Outline of survey method	
Benthic organism survey	Transect method	• For tidal flats and shallow sea areas where survey or diving staff can perform direct reconnaissance or diving surveys, the transect method is used to observe species, etc. • For sampling using quadrats (sampling tools) by divers, etc., multiple quadrats are placed on the measuring line for visual observations in the transect method, and samples within each quadrat are collected and analyzed.
Periphyton survey	Transect method	• Survey or diving staff observe the various species, cover degree (plants) and populations (animals) using the transect and quadrat methods.
Seaweed bed survey	Shipboard visual survey	• A water glass or other similar tool is used to check the existence of a seaweed bed and the outer edge of the growth zone of the species (the outer edge of the seaweed bed), and to record the position data using the GNSS, etc. • The observation and survey targets are the main species, types of vegetation cover, types of sediment patterns, water depth and other items. • If the species cannot be identified by a shipboard visual check, a sampling tool is used for sampling from the ship. An underwater camera survey and diving survey are combined in some cases.
	Transect method	• The diving staff observe and survey the main species in the seaweed bed, types of vegetation cover, types of sediment patterns, water depth and other items using the transect method. • The populations of the main species in the seaweed bed for each growth group (classifications of imagoes and larvae) are counted, and the leaf body length of several typical individual organisms is counted to understand the existing quantities, when necessary.
	Manta method	• A towing plate is pulled using a small ship, and an observer makes observations while being towed.
	Quadrat method	• A quadrat is placed on the sea floor, and seaweed bed types in the quadrat and their cover degrees are observed.
Fish and shellfish survey	Diving observation	• Diving staff observe and record the fish species, ecology and behavior of the fish and shellfish visually and by using an underwater video camera.
Seaside bird survey	Fixed point observation	• Fixed points that can overlook the seacoast and above the sea are installed on a measuring line to record bird species found in observations of approximately one hour, their populations, observation situations (e.g., visual check, bird calls, flying, resting and feeding), observation positions and locations.
Seaside plant survey	Survey of a cross section of vegetation	• For herbaceous and shrub communities, quadrats are continuously placed on a measuring line in a direction from the coast to the offing (transect survey), and species that appear in the quadrats, cover degree for each species, sociability, height and level are observed. • For arborous communities, a quadrat for which one side is the height of the community is placed in a typical community on a measuring line, and species that appear in the quadrat, the height of each level, predominant species in each level, cover degree and sociability are observed.

(4) Surveys of Habitat Functions

The magnitude of individual actions (e.g., photosynthetic rate, decomposition rate, predation rate, filtration rate of bivalves and CO_2 absorption rate) forming an ecosystem's functions is sometimes measured and compared. To measure an organism's action rates, a water tank or other similar tool is used in a laboratory to enclose the organisms while using the same existence conditions to the extent possible, and the environmental and concentration changes due to the actions of the organisms after a certain period of time has passed are measured. In tests for measuring the effects of photosynthesis by phytoplankton on the surface of fine tidal flats, a transparent cylinder is used to enclose the surface of the tidal flat along with the water directly above the bottom, and the photosynthetic rate is assessed from the oxygen concentration changes.[38)] For the measurement of CO_2 absorption rates and blue carbon, refer to the **Guideline of Blue Carbon (CO_2 Absorption and Carbon Sequestration) Measurement Methodology in Port Areas**.[39)]

For these types of special surveys, the objectives and indexes should be clarified, and the methods that are most appropriate should desirably be adopted. To measure the function rates, the original state and state after a certain amount of time has passed need to be compared; therefore, the measurement should be well planned so that significant differences before and after can be detected.

6.4.3 Assessments

In assessments of the impacts of projects on the environment and assessments of the effects of marine environment improvement projects, there are three main assessment viewpoints in surveys of living things: specific species, species diversity and ecosystems.

In regard to specific species, notable species in the target survey area are selected (**Table 6.4.2**), and the impacts on the target species are assessed using the prediction results (e.g., water quality and sediments) for the habitats, or trends in changes in the population over time are assessed. In regard to species diversity, community structures represented mainly by benthic organisms and periphyton are compared to those at comparative points for assessment. In regard to ecosystem assessments, the Habitat Evaluation Procedure (HEP) is used for comparisons to comparative points, and an ecosystem model is used to predict the environmental improvement effects from the viewpoint of the circulation of substances in the ecosystem. For the methods to assess species diversity and ecosystems, refer to the **Marine Ecosystem Survey Manual: Outline and Implementation**.[34)]

(1) Species Diversity

Species diversity is a comprehensive index that expresses the living situations of the organisms. When species are highly diverse, it means that various types of animals, plants and other organisms are living and growing. Moreover, species diversity is defined by species abundance and uniformity. This value is calculated based on data obtained in surveys of the survival of the organisms.

Species abundance refers to the number of species existing in a community (e.g., communities of benthic organisms). In general, the more species there are, the greater the species diversity in a community. In addition, even if the number of species is the same, when the state of a large population of a certain species is compared to another state where the population of each species is almost the same, the species diversity is greater for the latter case. This equality of populations in the observed species in a community is referred to as uniformity. Typical indexes that express species diversity are shown below. When comparing species diversity, it is important that the indexes are calculated from an identical sample size. For examples of measurement cases of species diversity, refer to the **Summary Report of Coastal Area Surveys (Rocky Shores, Tidal Flats, Seagrass Beds and Algal Beds) on Monitoring Sites 1000 Project in FY2008-2012**.[40)]

① **Genus**

The genera checked for the surveys of living things are a basic index that expresses species diversity. When species abundance across an entire area becomes higher through a combination of sites having communities with many characteristic species, it is called community complementarity.[41)]

② **Diversity**

(a) Shannon-Weaver diversity index

Use of the Shannon-Weaver diversity index H’ (also referred to as the “Shannon-Weaver function”) began in the 1960s as an index that expresses the species diversity in a community, and it is still widely used today. The greater the number of genera in a community, and the higher the uniformity of each species, the larger this index value is.

$$H' = -\sum_{i=1}^{S} p_i \ln p_i \ (0 \leqq \mathrm{H'}) \qquad \textbf{(6.4.1)}$$

Where,

S : genus

p_i : percentage of population n_i in the ith species to the total population N (n_i/N)

(b) Morishita index C_λ

Morishita index C_λ represents a community of organisms from the viewpoint of species composition based on similarities (similarity index). This index considers the multiplicity of species as its key characteristic. When the composition (genera and populations) of two communities from the first and second groups is completely the same, index C_λ is approximately 1, and when there are no identical families between the first and second groups, index C_λ is 0.

$$C_\lambda = \frac{2\sum_{i=1}^{S} n_{1i} \cdot n_{2i}}{(\lambda_1 + \lambda_2)N_1 \cdot N_2}, 0 \le C_\lambda \le 1 \tag{6.4.2}$$

$$\lambda_1 = \frac{\sum_{i=1}^{S} n_{1i}(n_{1i} - 1)}{N_1(N_{1-1} - 1)},\ \lambda_2 = \frac{\sum_{i=1}^{S} n_{2i}(n_{2i} - 1)}{N_2(N_{2-1} - 1)} \tag{6.4.3}$$

Where,

N_1 and N_2 : total number of samples in the first and second group

n_{1i} and $_{\mathrm{n2i}}$: number of samples in the *i*th section in each group

S : number of groups in a section

λ : unbiased estimator of the population of Simpson's simplicity index $\Sigma\Pi^2$

(2) Ecosystem Assessments

The living situations of coastal organisms are determined by the interactions of three types of organisms: producers that produce organic substances through photosynthesis, animals (predators) that prey upon such organic substances, and bacteria that decompose dead organisms and organic substances. Production of organic substances by microorganisms and the prey of such organic substances are taken into account as interactions, in some cases. These types of links between organisms and links between non-biological conditions (e.g., habitats and other physical conditions) and organisms are called ecosystems as a whole.

Common ecosystem assessment methods are environmental function assessment methods (e.g., HEP, WET and HGM), in which data obtained at the actual site are analyzed to assess the organisms' habitat, and organism function assessment methods (e.g., ecosystem modeling), in which a numerical simulation model that quantitatively analyzes the circulation of substances in an ecosystem, including the food chain, is used for assessment.

① Environmental function assessment methods

(a) HEP

The Habitat Evaluation Procedure (HEP) is a habitat assessment system developed by the U.S. Fish and Wildlife Service to show a comparative assessment between an ecosystem that will be destroyed in a mitigation plan and another compensating ecosystem, quantitatively and objectively.

In the method, the Habitat Suitability Index (HSI) for one or multiple species (target organisms) is multiplied by the area, and this quantified habitat unit (HU: HSI × area) is used for the assessment. To calculate the HSI, the Suitability Index (SI), which shows the relationship between individual environmental factors and the quantity of organisms, should be determined. The SI is the ratio of the state of an environmental factor in a habitat in a survey area to that of the environmental factors in an ideal habitat. It should be determined such that it is 1 for an optimum environment and 0 for an environment unfit for survival.

The SI should be calculated for each environmental factor that controls the survival of the organisms, and the SIs combined to calculate the HSI. The SI does not need to be a continuous function, and the SI values can be set for quantitative or qualitative assessment ranks. Methods to combine the SIs are called HSI models. Example methods are the arithmetical average method, geometric mean method and limiting factor method.

(b) WET

The Wetland Evaluation Technique (WET), developed by the U.S. Army Corps of Engineers, is a functional assessment system for wetlands using a hydrogeomorphological technique. The functions of wetlands are qualitatively assessed in three levels (high, moderate and low) in accordance with a predetermined assessment flow. Various items are used for the assessments: underground water conditions, cultural functions, flood adjustment function, purification function and organism survival.

(c) HGM

The Hydrogeomorphic Approach (HGM), developed by the U.S. Army Corps of Engineers, is a functional assessment system for wetlands using a hydrogeomorphological technique. The wetland is first subdivided into classes and sub-classes in consideration of the geomorphological location, stream source and flow. Then, the various wetland functions of each class are assessed using the Functional Capacity Index (FCI).

The FCI is defined as a ratio of the functional capacity (FC) of a wetland (the assessment target) to the FC of the reference standard (a wetland in the same class or sub-class that can be regarded as exerting its essential functions). The ratio should be determined such that it is 0 to 1. The target wetland's FCI is multiplied by its area to assess the FC. This method has the characteristics of both the HEP and WET, focusing on various functions of the wetland, as with the WET, and is capable of quantitative assessments.

② **Ecosystem modeling**

Ecosystem modeling is a technique to assess the circulation of substances in an ecosystem quantitatively by considering the organisms' inherent functions (metabolism, immune system and internal secretions), ecological functions in relationships within and between species, and interactions between the organisms and abiotic environments (circulation and transportation of substances). Lower-order ecosystem models that consider low-order organisms (e.g., plankton) at the trophic level are used to predict changes in water quality, water quality purification function, primary production amounts, etc. Models that consider higher-order consumers (e.g., fish and birds) are called higher-order ecosystem models, but systems can become complicated, and therefore, there are no generalized models.

(3) Example Ecosystem Assessments

For assessments of the impacts on projects including port facility development, refer to **reference 42)**. For marine environment improvement projects, refer to **references 43), 21)** and **44).**

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Chapter 2 Surveys and Tests after a Large Earthquake and Tsunami

1 General

1.1 Purpose of this Chapter

This chapter summarizes the surveys and tests that must be implemented after multiple ports are damaged by a large-scale earthquake and resulting tsunami (hereinafter referred to as a "large earthquake and tsunami"), for facilitating the understanding of the overall damage; determining the usability of port facilities, such as mooring facilities; executing emergency measures, such as the elimination of port obstacles; and implementing disaster relief projects based on the measures taken following the 2011 Great East Japan Earthquake.[1)] The contents of this chapter can also be referred to when responding to disasters due to earthquakes without tsunamis, as well as storm surges and high waves.

1.2 Structure of this Chapter

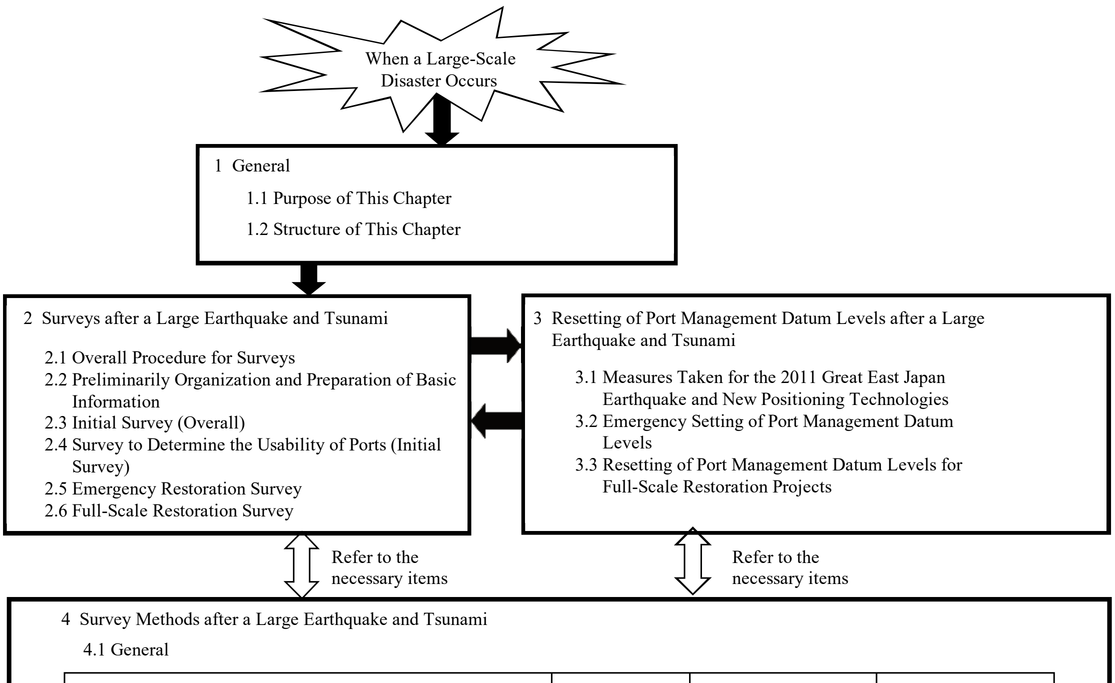
Fig. 1.2.1 shows the structure of this chapter. The contents and purposes of each section are as follows.

①**Section 1:** The purpose and structure of this chapter.

②**Section 2:** The overall procedure for surveys, outlines of the surveys to be implemented at each stage (initial, emergency and full-scale restoration stages), and surveys to determine the usability of ports.

③**Section 3:** The method for resetting Chart Datum Level (C.D.L.) after a large earthquake and tsunami.

④**Section 4:** The viewpoints of the surveys and outlines of the survey methods at each stage, obtainable information, and points to consider with respect to seven major items expected to be surveyed after a large earthquake and tsunami.



	Initial Survey	Emergency Restoration Survey	Full-Scale Restoration Survey
4.2 Understanding the Overall Damage Situation	○	○	
4.3 Understanding the Geometry of Onshore Areas	○	○	○
4.4 Understanding the Geometry of Underwater Areas	○	○	○
4.5 Understanding Broad-Based Ground Deformation		○	○
4.6 Surveys on Ground Liquefaction		○	○
4.7 Understanding the Cavities at Apron Sections		○	○
4.8 Understanding the Deformation on Underground Structures			○

Fig. 1.2.1 Overall Structure of "Surveys and Tests after a Large Earthquake and Tsunami"

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2 Surveys after a Large Earthquake and Tsunami

2.1 Overall Procedure for Surveys

In order to utilize ports as disaster relief bases after a large earthquake and tsunami, it is necessary to promptly implement several surveys on the damage. **Fig. 2.1.1** shows the overall procedure for surveys after a large earthquake and tsunami based on the measures taken for the 2011 Great East Japan Earthquake.[1] The outlines of the surveys and points to consider at each stage are described below.

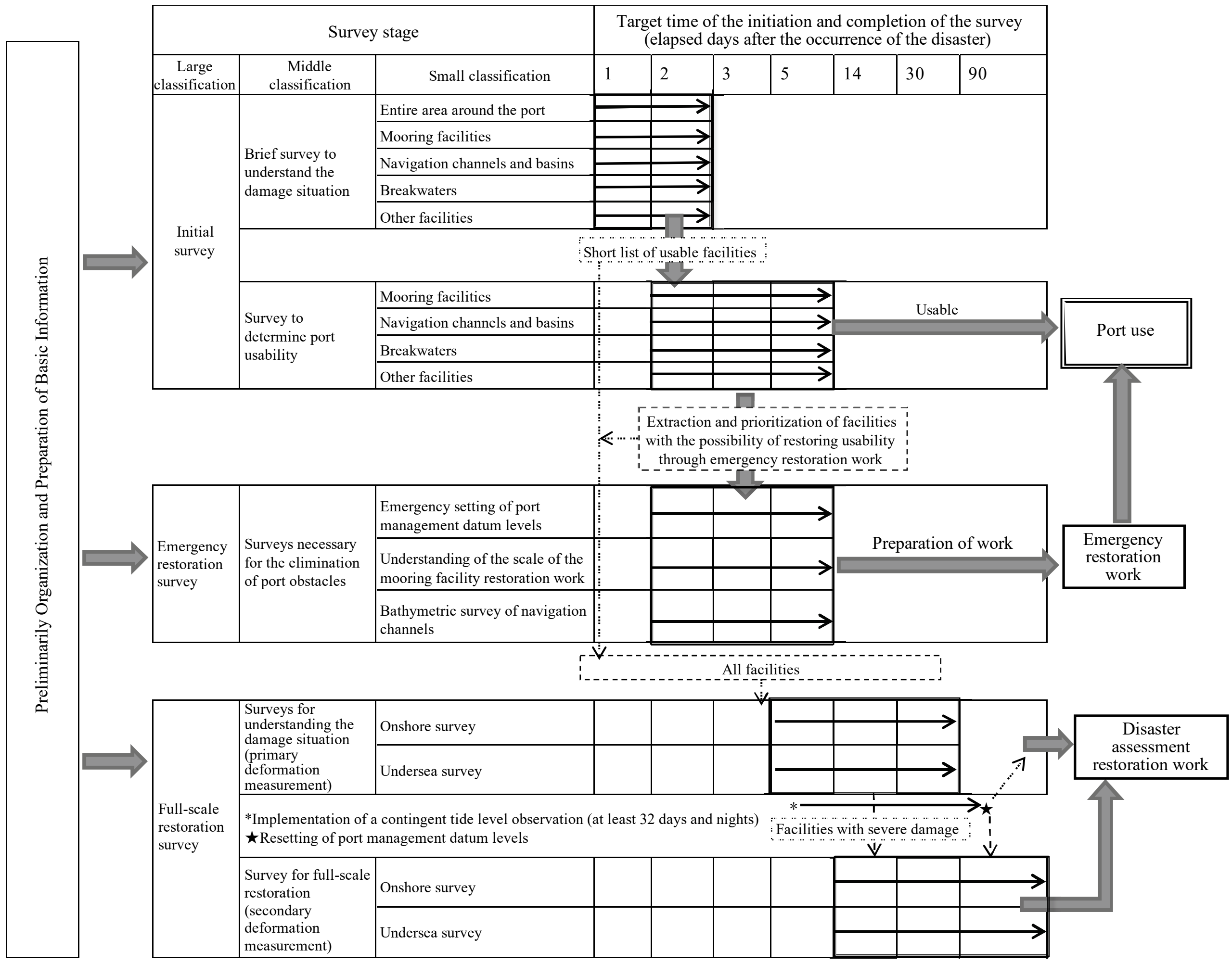


Fig. 2.1.1 Overall Procedure for Surveys on the Damage after a Large-Scale Earthquake and Tsunami

2.2 Preliminarily Organization and Preparation of Basic Information

In preparation for prompt and efficient implementation of post disaster surveys, it is necessary to appropriately organize and manage several pieces of basic information on the facilities that are to be the survey objects and control points so that the information can be readily available. It is also necessary to make the information on the business continuity plans after the occurrence of a disaster readily available for the smooth implementation of surveys in collaboration and coordination with many related organizations. However, there may be cases where the damage to office buildings, including inundation, due to a large earthquake and tsunami causes the information to become inaccessible. Thus, the information necessary for surveys at the initial stage is preferably stored at multiple locations or on several media. The basic information to be organized and prepared in advance is listed below.

- Information on the structural profiles of the object facilities (design conditions, typical cross-sectional drawings, coordinate values, maintenance plans, etc.)
- Information on the control points and chart datum level (refer to **Reference (Part II), Chapter 2, 3 Resetting of Chart Datum Level after a Large Earthquake and Tsunami**)

- Information on port business continuity plans, Technical Emergency Control Forces and disaster-relief cooperation agreements

2.3 Initial Survey (Overall)

(1) General

The initial survey is a type of survey which must be initiated on the day of the occurrence of a disaster, and completed within a few days for the purpose of identifying port facilities (including those which can be reinstated through simple restoration work) that are usable for allowing ships carrying emergency disaster relief supplies to enter and leave the ports, and that allow berthing for loading and unloading the supplies. The initial survey is classified into a “brief survey to understand the damage situation” and a “survey to determine the usability of the facilities.” For the details of the initial survey, refer to the **Reference 1)**.

(2) Brief Survey to Understand the Damage Situation

The brief survey to understand the damage situation shall be implemented immediately after the tsunami has struck in a manner that determines the damage situation of the breakwaters, mooring facilities, navigation channels, basins and seawalls through visual confirmation (including observations with binoculars, cameras and video cameras) from safe locations. In this survey, swift information collection is given priority over accurate information, and information collection shall include overall pictures of the disaster situation in the areas surrounding the ports which cover access roads at the back of the port districts by utilizing satellite and aerial photographs and videos (refer to **Reference [Part II], Chapter 2, 4.2 Understanding the Overall Damage Situation**).

Table 2.3.1 shows the methods and standpoints of a brief survey to understand the damage situation. Although the object facilities listed in the table below are limited to mooring facilities, navigation channels, basins, breakwaters and seawalls, other facilities shall also be surveyed as needed.

Table 2.3.1 Methods and Standpoints of a Brief Survey to Understand the Damage Situation

<table>
<tr><th>Object</th><th>Method</th><th>Standpoint</th></tr>
<tr><td>Areas around the port</td><td>Satellite imaging (4.2.2)
Aerial photography using aircraft (4.2.3)
Aerial photography using UAVs (4.2.4)
Aerial laser surveys (4.2.5)
Other (4.2)</td><td>• Damage situation of several port facilities
• Inundation situations of the areas around the port
• Situations of wreckage blocking roads
• Floating debris blocking navigation channels
• Other</td></tr>
<tr><td>Mooring facilities</td><td rowspan="4">Visual confirmation, photographs and video recordings</td><td>• Abnormalities in the structural bodies (swelling, inclination and settlement)
• Caving and level differences on the aprons
• Obstacles blocking access roads to the aprons</td></tr>
<tr><td>Navigation channels and basins</td><td>• Amounts and types of floating debris in waterways and basins as well as the extent of the impact</td></tr>
<tr><td>Breakwaters</td><td>• Abnormalities in the structural bodies (overturning, sliding, inclination and settlement)</td></tr>
<tr><td>Seawalls (revetments, dikes, water gates and land locks)</td><td>• Prevention of storm surges from leading to secondary disasters
(Current situation of protection line against a high tide)</td></tr>
</table>

The results of the brief survey to understand the damage situation are used for identifying those facilities which have a high possibility of being continuously used as is and reinstated through simple restoration work (hereinafter referred to as “emergency restoration work”). When identifying usable mooring facilities and breakwaters, priority shall be given to those that have received only minor damage (e.g., 0 or I) in the classification of physical damage (0 to IV) shown in **Tables 2.3.2** and **2.3.3**. When identifying usable seawalls, priority shall be given to those which shall be reinstated through emergency restoration work from the viewpoint of preventing storm surges from leading to secondary disasters.

Table 2.3.2 Classification of the Levels of Damage to Mooring Facilities[2)]

Damage level	Damage situation
0	No damage
I	No abnormalities in the structural body, except for destruction or deformation of accessories
II	Substantial deformation of the structural body
III	Externally sound but evident internal destruction of the structural body
IV	Total destruction and loss of the original shape

Table 2.3.3 Classification of the Levels of Damage to Breakwaters (Caisson Type)[1)]

Damage level	Damage situation
0	No damage
I	No abnormalities in the caisson body, except for minor deformation, such as the settlement of the levee crown, or deformation or destruction of wave-dissipating work and mounds
II	No abnormalities in the caisson body, except for deformation such as settlement, sliding or inclination, which can be restored without reinstalling the caisson
III	Remarkable settlement, sliding or inclination of the caisson, which can be reinstated through reinstallation of the caisson (including partial repair or reinforcement of the caisson members)
IV	The caisson body has slid out of the mound and has no possibility of being reinstated (including structural destruction and damage preventing the caisson from being refloated)

In addition, a system has been developed that enables damage situations to be understood through strong motion observation records.[3)] This system instantaneously acquires the information on earthquake ground motions observed through strong motion seismometers on the occurrence of an earthquake, and estimates the damage situation of the facilities based on the acquired information. The system is one of the most effective methods for conducting a brief survey to understand the damage situation in cases where field surveys cannot be implemented because a tsunami warning is in effect after a large earthquake, or the disaster occurred at night. Note, however, that this system is considered to be inferior to the in-situ direct measurement of displacement (refer to **Reference (Part II), Chapter 2, 2.4 Survey to Determine the Usability of Ports (Initial Survey)**) in terms of accuracy. The system is divided into a simplified method which uses a relationship between the levels of damage preliminarily obtained through a two-dimensional effective stress analysis (refer to **Reference (Part III), Chapter 1, 2 Basic Items Concerning Seismic Response Analysis**) and the velocity PSI values of the earthquake ground motions,[4)] as well as a detailed method which estimates the levels of damage through an automatic two-dimensional effective stress analysis directly using observed ground motion waveforms. Although the detailed method requires a long calculation time, it is considered to be more accurate than the simplified method. However, the simplified method is advantageous in that it can produce analysis results instantaneously.

(3) Survey to Determine the Usability of the Facilities

The survey to determine the usability of the facilities is a field survey implemented immediately after the **Brief Survey to Understand the Damage Situation Described in Item (2)** above. It shall be initiated about two days after and completed about five days after the occurrence of the disaster for the purpose of ensuring the transportation routes for emergency disaster relief supplies. In the survey, the usability of the port facilities is determined (refer to **Reference (Part II), Chapter 2, 2.4 Survey to Determine the Usability of Ports [Initial Survey]**) for the facilities (such as the mooring facilities, breakwaters, navigation channels and basins) extracted as a result of the brief survey to understand the damage situation as the determination objects. Those facilities determined to be usable as is shall be put back into service as a part of the transportation routes for emergency disaster relief supplies. In addition, the emergency restoration survey (refer to **Reference (Part II), Chapter 2, 2.5 Emergency Restoration Survey**) shall be implemented for the facilities subjected to emergency restoration work.

Table 2.3.4 shows the standpoints for determining the usability of the facilities (such as the mooring facilities, navigation channels, basins and breakwaters). For the details of the survey to determine the usability of the facilities, refer to **Reference (Part II), Chapter 2, 2.4 Survey to Determine the Usability of Ports (Initial Survey)**.

Table 2.3.4 Standpoints in the Determination of the Usability of Facilities
(Mooring Facilities, Navigation Channels, and Basins, Breakwaters)

Object	Method	Standpoint (refer to **2.4** for details)
Mooring facilities	(Refer to **2.4**)	• Is the structural stability secured? • Does large-scale scouring occur on the ground in front of the mooring facilities? • Can ships come alongside the mooring facilities? • Can cargo loading and unloading be conducted? • Is road accessibility maintained?
Navigation channels and basins		• Are the required water depths secured? • Is there a large amount of floating debris in the waterways and basins? • Are there obstacles protruding from the sea surface?
Breakwaters		• Can they block waves from the open sea?

2.4 Survey to Determine the Usability of Ports (Initial Survey)

In order to utilize ports as disaster relief bases after a large earthquake and tsunami, it is necessary to promptly implement multiple surveys, put safe and usable facilities back into service early, and take measures to keep people away from those facilities determined to be unsafe. Here, the outlines and points to consider necessary for determining the usability of facilities are described for mooring facilities, navigation channels and basins as the determination objects, based on past experiences with actual disasters involving earthquakes and tsunamis.

The standpoints of the survey vary facility by facility. In the case of mooring facilities, their usability shall be determined according to whether or not the structural stability is secured, whether large-scale scouring occurs on the ground in front of the mooring facilities, whether ships can come alongside them, whether cargo loading and unloading can be conducted, and whether road accessibility is maintained. In the case of navigation channels and basins, their usability shall be determined according to whether or not the required water depths are secured, and whether there is a large amount of floating debris. In any event, advance preparations are important for making an appropriate determination of the usability of facilities after the occurrence of a disaster.

2.4.1 Survey to Determine the Usability of Mooring Facilities

(1) Differences in Survey Contents Depending on Structural Types

It shall be noted that the damage patterns of mooring facilities vary depending on the types of mooring facilities (e.g., gravity-type wharves, sheet pile wharves and piled piers), and, therefore, the information required to determine the usability of mooring facilities largely differs depending on their types.

① **Gravity-type wharves**

The typical type of damage to gravity-type wharves due to earthquakes that has been observed in the past is the seaward displacement of wharf bodies, along with their settlement and inclination, which causes level differences on the ground behind the wharf bodies (refer to **Figs. 2.4.1** and **2.4.2**).[5] Wharf bodies which undergo significant seaward displacement interfere with ship berthing, and large level differences on the ground behind the wharf bodies interfere with vehicle traffic. In contrast, there have been cases where no significant damage to the wharf bodies was reported even after the actions of large external forces, as was the case with Kobe Port, which was hit by the 1995 South Hyogo Prefecture Earthquake. Thus, for gravity-type wharves, it is considered to be sufficient to determine their usability based on whether or not ships can come alongside them and whether road accessibility is maintained. Of course, there may be a risk that the wharf bodies could lose their structural stability when horizontally displaced to a level in which they come very close to the tops of mound slopes. However, the determination criteria for usability are generally much stricter than those for structural stability. Thus, the usability of gravity-type wharves can be visually determined to some extent, and, in that sense, there may be cases where visual surveys using UAVs can be a useful method to determine the usability of gravity-type wharves in the future. When determining the usability of concrete block wharves, which are also classified as gravity-type wharves, it is necessary to confirm that the concrete blocks have maintained their integrity.

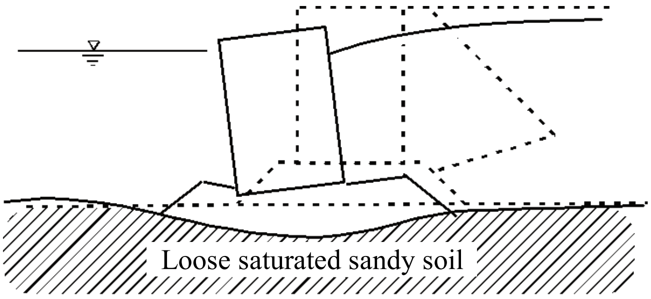


Fig. 2.4.1 Typical Damage Pattern of Gravity-Type Wharves[5)]

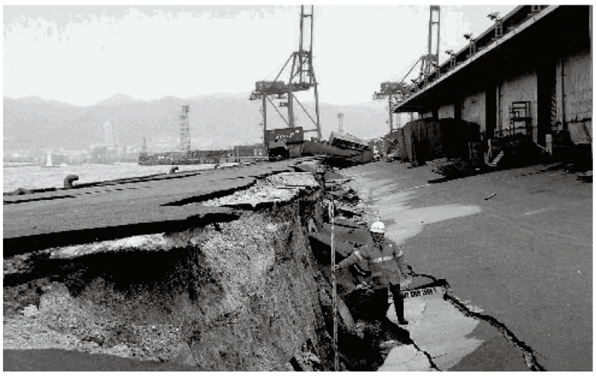


Fig. 2.4.2 Damage to Gravity-Type Wharves Due to the South Hyogo Prefecture Earthquake (The image is reversed)

② **Sheet pile wharves and piled piers**

There have been cases where mooring facilities mainly made of steel members, such as sheet pile wharves and piled piers, have lost structural stability due to damage to these structural members. These cases include, for example, the cracks on the sheet pile wharf in Akita Port due to the 1983 Japan Sea Earthquake (refer to **Fig. 2.4.3**), and the buckling of the steel pipe pile on the piled pier in Kobe Port due to the 1995 South Hyogo Prefecture Earthquake (refer to **Fig. 2.4.4**).[5)] Thus, the determination of the usability of sheet pile wharves and piled piers cannot be sufficiently made based only on the availability of ship berthing and road accessibility, and additionally requires reliable confirmation of the stress state of the structural members. Particular attention shall be given to the fact that the determination of usability may not be possible through visual surveys or submersible surveys by divers in cases where the underground members are damaged, as with previous disasters (refer to **Fig. 2.4.4**).[5)] **Fig. 2.4.4** illustrates a damage pattern of a piled pier. When a sheet pile wharf undergoes a similar deformation mode, as shown in **Fig. 2.4.5**, the bending moment on the structural members is maximized at the underground portions. The following section describes the procedure for determining the usability of mooring facilities mainly made of steel members such as sheet pile wharves and piled piers.

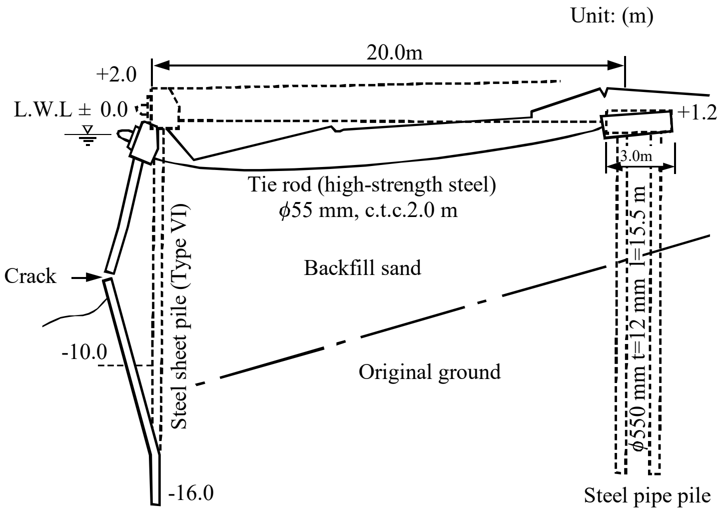


Fig. 2.4.3 Damage to a Sheet Pile Wharf Due to the Japan Sea Earthquake (Cracks on the Sheet Pile)[5)]

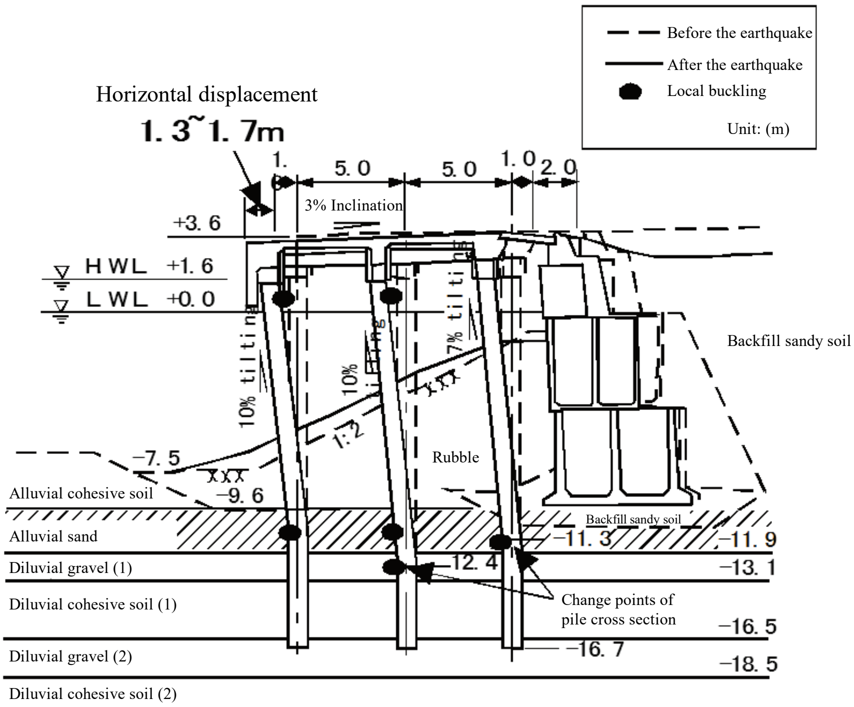


Fig. 2.4.4 Damage to a Piled Pier Due to the South Hyogo Prefecture Earthquake (with Local Buckling on Underground Piles)[5)]

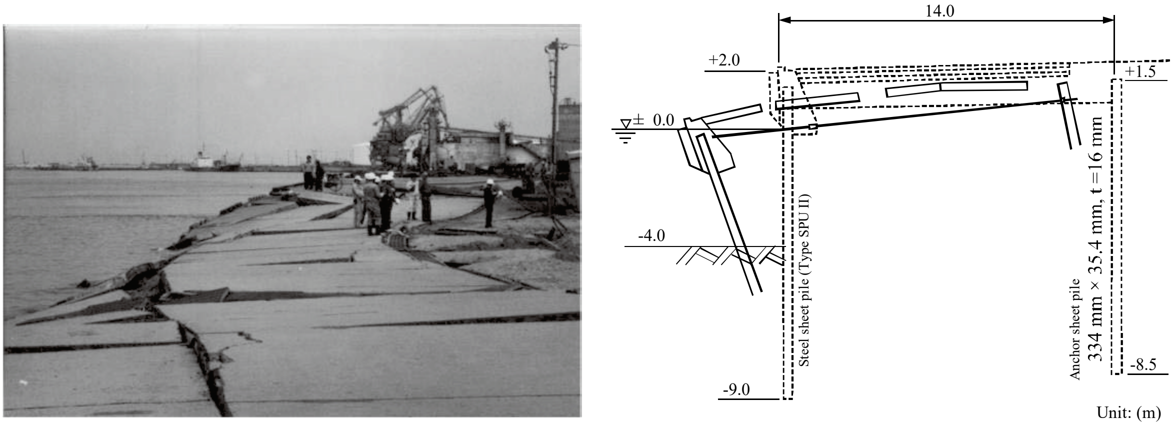


Fig. 2.4.5 Damage to a Sheet Pile Wharf Due to the Japan Sea Earthquake (Displacement of Anchorage Work)[5)]

(2) Procedure for Determining the Usability of Mooring Facilities Mainly Made of Steel Members Such As Sheet Pile Wharves and Piled Piers

As described in Item **(1)** above, it is necessary to confirm the deformation and stress state of structural members when determining the usability of mooring facilities mainly made of steel members such as sheet pile wharves and piled piers. The deformation and stress state can be confirmed through the following procedure (shown in **Fig. 2.4.6**).

① The residual horizontal displacement of the levee crowns due to the deformation of the mooring facilities shall be acquired through survey results before and after an earthquake.

② The relationships among the residual horizontal displacement of the levee crowns, deformation of the members and stress states shall be preliminarily obtained for the respective facilities. Then, the stress states of the members and the structural stability of the facilities shall be evaluated based on the residual horizontal displacement obtained through ① above.

③ The usability of the mooring facilities shall be finally determined using information on the structural stability together with the determination results based on the availability of ship berthing and road accessibility, as is the case with gravity-type wharves.

In the above procedure, the methods which can be used in ① and ② are the RTK-GNSS (refer to **Reference (Part II), Chapter 2, 4.3.3 Real-Time Kinematic Survey**) and a ground-structure system dynamic analysis, such as the FLIP (refer to **Reference (Part III), Chapter 1, 2 Basic Items concerning Seismic Response Analysis**), respectively. It shall be noted that the determination can be made only when steps ① and ② above are combined. In addition, steps ① and ② respectively require advance preparation, as described below. Furthermore, in the case of remarkable settlement of the ground behind a sheet pile wharf, even though the residual horizontal displacement of the levee crown is small, there may be a risk of damage to the sheet pile body, and, therefore, it is necessary to confirm such damage through a submersible survey.

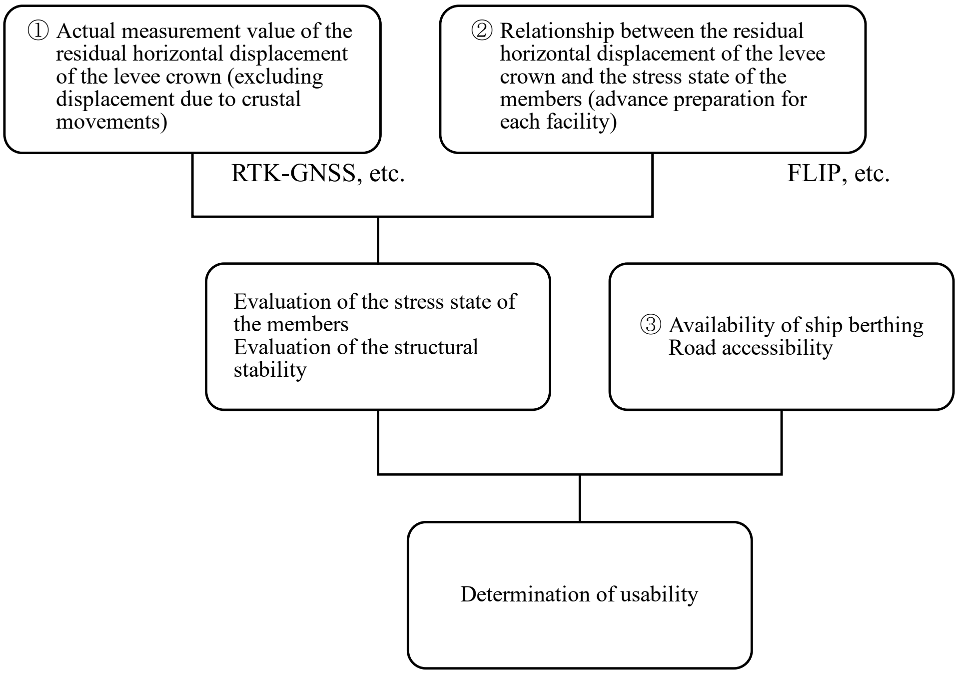


Fig. 2.4.6 Procedure for Determining the Usability of Mooring Facilities Mainly Made of Steel Members such as Sheet Pile Wharves and Piled Piers

(3) Understanding the Residual Horizontal Displacement of Levee Crowns Due to the Deformation of Mooring Facilities

The residual horizontal displacement of levee crowns due to the deformation of mooring facilities shall be acquired through the measurement of the displacement directly related to the stress states of the members with attention paid to the following points based on experiences with past disasters.

First, it is not appropriate to rely on visual confirmation for determining the presence or absence of residual horizontal displacement in consideration of a precedent where a mooring facility, which was hit by the 1995 South Hyogo Prefecture Earthquake, appeared to have almost no displacement, but turned out to have had a displacement of about 50 cm through later aerial photogrammetry[6] (for an example, refer to **Photo4.2.3.5** of Facility No. ③ at Maya Wharf in the **Reference 7)**). Overlooking a displacement of about 50 cm is a fatal error in the evaluation of the stresses in members. In addition, it is not appropriate to use a tape measure to measure the relative displacement between an object and a facility, which appears to have no displacement to the naked eye. This is because even facilities which appear to have no displacement may have it in reality.

Here, it is necessary to survey the residual horizontal displacement of the levee crowns of mooring facilities; however, it shall be noted that crustal movements (refer to **Reference (Part II), Chapter 6, 2 Crustal Movements**) due to large-scale earthquakes may put the control points of the Geospatial Information Authority of Japan out of commission for some months. Thus, for determining the usability of facilities within five days after the occurrence of a disaster, there is a necessity to establish a survey system without relying on the control points of the Geospatial Information Authority of Japan.

Generally, the displacement of mooring facilities is the sum of the displacement due to crustal movements and the displacement due to the deformation of the mooring facilities (i.e., the displacement due to the local deformation of the ground around the mooring facilities).

(a) Displacement of mooring facility = (b) Displacement due to crustal movements + (c) Displacement due to the deformation of mooring facilities

Because the displacement due to crustal movements has no relationship with the evaluation of the stresses in the members, the differences in the absolute coordinates of the mooring facilities before and after an earthquake include the displacement of (b) and (c) above, and, therefore, they are not appropriate for the evaluation of the stresses in the members.

Then, it is necessary to secure "reference points" behind the mooring facilities, which are not considered to be subjected to the local deformation of the ground around the mooring facilities (although they are subjected to crustal movements), and to measure the relative displacement between the reference points and the displaced mooring facilities (as shown in **Fig. 2.4.7**). Here, the locations of the reference points shall be appropriately selected so as to avoid the influence of the local deformation of the ground around the mooring facilities (or other local ground conditions), with attention paid to the possibility that the influence of the local deformation of the ground around the mooring facilities can be felt at a fairly far distance behind the mooring facilities, as with Maya Wharf in **Fig. 2.4.8**. The information on (c) displacement due to the deformation of the mooring facilities necessary for the evaluation of the stresses in the members can be obtained by comparing the distances between the reference points and mooring facilities before and after an earthquake.

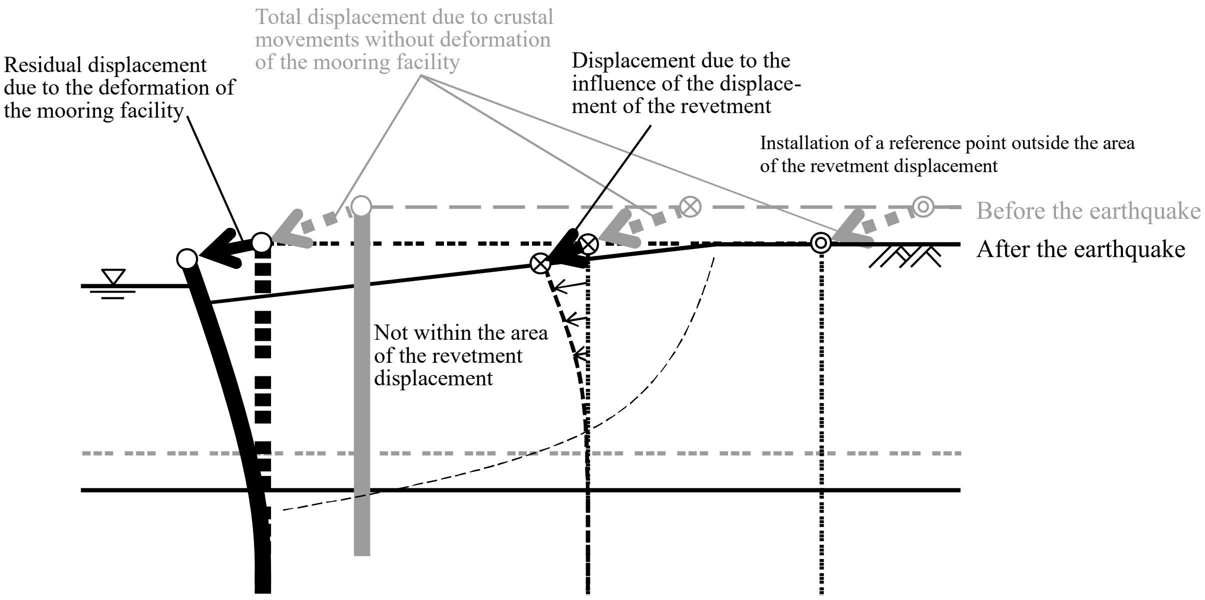
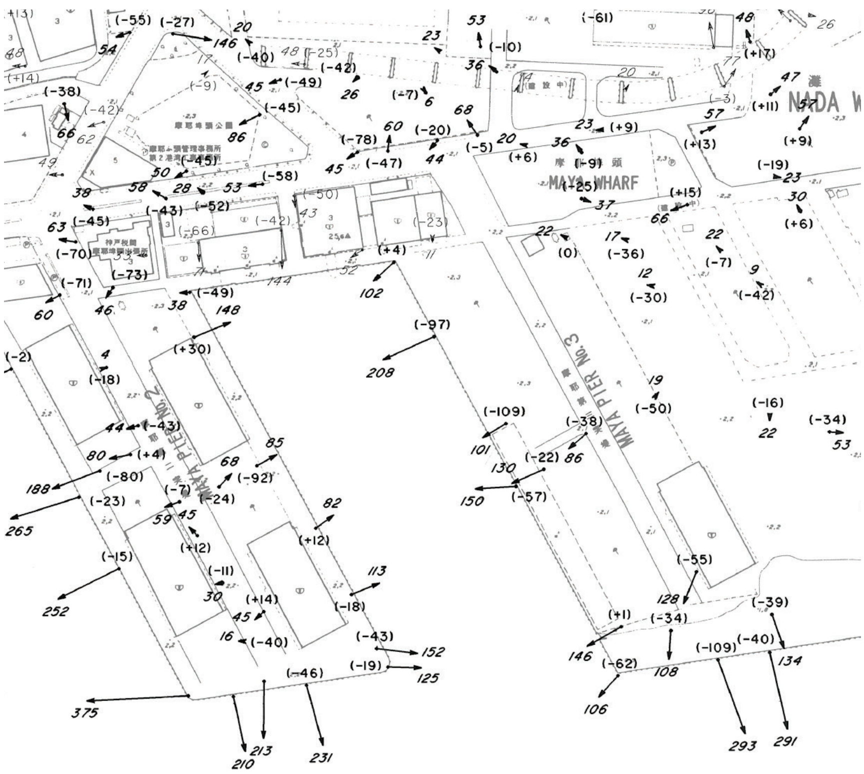


Fig. 2.4.7 Method for Eliminating the Influence of Crustal Movements



(Unit: cm)

Fig. 2.4.8 Distribution of the Displacement at Maya Wharf Due to the 1995 South Hyogo Prefecture Earthquake[6)]

There are several ways of measuring the relative displacement between the reference points and mooring facilities. Among them, the RTK-GNSS[8)] is essentially suitable for the measurement of the relative displacement between two points in that it can cancel out errors associated with the measurement by simultaneously measuring two locations: one on the reference point and the other on the mooring facility. With a measurement accuracy of the relative horizontal displacement between the two locations with a margin of error of about 2 cm, the RTK-GNSS can be sufficiently used for the evaluation of the stresses in the members of mooring facilities.

(4) Preliminary Estimation of the Relationship between the Residual Horizontal Displacement of the Levee Crowns and Stress States

A point of caution in the preliminary estimation of the relationship between the residual horizontal displacement of the levee crowns and stress states of the members is the importance of the individuality of the structures and ground conditions of the respective facilities. For example, the modes of deformation vary depending on the relative relationship between the steel member and the ground stiffness. The deformation modes also vary depending on the relative relationship between the bearing capacity of the members, such as between a sheet pile wall and anchorage work, in the case of a sheet pile wharf. Thus, it is necessary to take into consideration these factors which affect the deformation modes of the mooring facilities when estimating the relationship between the residual horizontal displacement of the levee crowns and stress states. Of course, the deformation of the ground itself needs to be taken into consideration. In the case of a piled pier, if only the superstructure undergoes horizontal displacement without any deformation of the ground, a short pile effect causes the land side piles to become susceptible to severer stress states, but this is not the case when the ground (including rubble) undergoes deformation. In addition, a small difference in ground conditions affect the relationship between the residual horizontal displacement of the levee crowns and stress states. For example, if two types of ground with high stiffness and low stiffness are located next to each other, the ground with low stiffness is subjected to the concentration of strain, thereby affecting the distribution of the stresses in the members.

A ground-structure system dynamic analysis such as the FLIP (refer to **Reference (Part III), Chapter 1, 2 Basic Items concerning Seismic Response Analysis**) can be used as a method for preliminarily estimating the relationship between the residual horizontal displacement of the levee crowns and the stress states of the members while taking into consideration the conditions above. The **References 4)** and **9)** introduce cases of a preliminary estimation of the relationship between the residual horizontal displacement of the levee crowns and stress states of the members conducted through the ground-structure system dynamic analysis with facilities in the jurisdiction of the Chubu Regional Development Bureau as the estimation objects.

2.4.2 Survey to Determine the Usability of Navigation Channels and Basins

In order to enable ports to be utilized as disaster relief bases after a large earthquake and tsunami, it is necessary to be able to not only the mooring facilities but also the navigation channels and basins. For navigation channels and basins, it is necessary to quickly secure temporary water depths (temporary water depths) that allow ships to navigate safely. To that end, it is important to acquire information as early as possible on the extent of the planar distribution of the debris accumulated on the seafloor and the extent of the reduction in water depth due to the accumulation of debris. This information can be effectively acquired with swath sounding machines and side-scan sonars that enable the water depths and obstacles in wide areas to be easily detected, as described in **Reference (Part II), Chapter 2, 4.4 Understanding the Geometry of Underwater Areas**. Furthermore, in order to enhance the accuracy of the water depth measurements, it is necessary to urgently set chart datum level (refer to **Reference (Part II), Chapter 2, 2.5 (3) Emergency Setting of Chart Datum Level in Bathymetric Surveys** and **Reference (Part II), Chapter 2, 3 Resetting of Chart Datum Level after a Large Earthquake and Tsunami**).

2.5 Emergency Restoration Survey

(1) Emergency Restoration Survey

The scope of the emergency restoration survey is the scale of the restoration work (the approximate required quantities of equipment and materials) with respect to the facilities which can be reinstated through emergency restoration work for the early reception of ships transporting emergency disaster relief supplies (the removing port obstacles). For seawalls, the scope of the emergency restoration survey is the scale of restoration work at locations where the temporary restoration of storm surge protection lines through emergency restoration work. The emergency restoration survey needs to be initiated within two days after the occurrence of a disaster and completed within five days, and implemented in parallel with the “survey to determine the usability of facilities.” For the details of the emergency restoration survey, refer to the **Reference 1)**.

Table 2.5.1 shows the standpoints of the emergency restoration survey, an outline of the restoration work and methods for obtaining the approximate work quantities.

Table 2.5.1 Standpoints of the Emergency Restoration Survey, an Outline of the Restoration Work and Methods for Obtaining Work Quantities

Object	Standpoint	Outline of restoration work and method for obtaining work quantities
Vicinity of mooring facilities (onshore)	• Restoration of caving and level differences which interfere with vehicle traffic on aprons and roads behind the mooring facilities (with crushed stone, earth fill, steel plates, etc.) • Removal of obstacles that impede vehicle traffic	○ Laying crushed stone and steel plates as well as removing obstacles • Visual survey (visual observations, photographs, etc.) • Simplified measuring (staffs, rods, pinholes, etc.) • Other (refer to **Section 4**)
Navigation channels, basins and mooring facilities (undersea)	• Identification of abnormal objects on the seafloor and deformation of the front sections of the mooring facilities, which interfere with ship navigation, and setting of temporary water depths (changes in seafloor topography and types of debris)	○ Removal of obstacles undersea and on the seafloor with self-propelled grab barges • Bathymetric survey (refer to **4.4**) • Visual survey by divers • Other
Seawalls (mainly onshore)	• Locations requiring emergency protection to prevent the expansion of inundation damage due to storm surges and high waves and to secure the safety of vehicle traffic (with sandbags, concrete blocks, etc.)	○ Laying of sandbags and concrete blocks • Visual survey (visual observations, photographs, etc.) • Simplified measuring (staffs, rods, pin holes, etc.) • Other (refer to **Section 4**)

(2) Points to Consider when Establishing Emergency Restoration Survey Plans

Emergency restoration surveys are implemented for removing port obstacles as early as possible. Thus, emergency restoration surveys shall be flexibly planned in a manner that allows the measuring accuracy to be lowered and the measuring intervals to be decreased.

(3) Emergency Setting of Chart Datum Level in Bathymetric Surveys

Large-scale earthquakes may cause extensive ground deformation. Thus, when implementing bathymetric surveys for navigation channels and basins, as described in Item (1) above, it is necessary to urgently set the temporary chart datum level for the purpose of urgently calculating the water depths. For the methods for urgently setting the temporary chart datum level, refer to **Reference (Part II), Chapter 2, 3 Resetting of Chart Datum Level after a Large Earthquake and Tsunami**.

2.6 Full-Scale Restoration Survey

(1) Outline

In a full-scale restoration survey, the disaster states of the facilities are accurately observed or measured, and two-stage measurements are implemented for the disaster assessments and restoration design of the damaged facilities. In addition, a full-scale restoration survey shall be implemented at the appropriate time or on appropriate schedules in tandem with the preparation and progress of the emergency restoration work. For detailed information on full-scale restoration surveys, refer to the **Reference 1)**.

- Primary deformation measurement: The post-disaster deformation of all damaged facilities shall be measured using measuring equipment. For facilities with minor deformation, the applications for disaster assessments shall be made based on the primary deformation measurement. The primary deformation measurement shall be initiated within 5 days and completed within 30 days after the occurrence of a disaster.

- Secondary deformation measurement: The post-disaster deformation shall be measured in detail for the facilities determined to have major damage in the primary deformation measurement in order to investigate the reasons for the damage, formulating a restoration design policy and reset the design conditions to be a presupposition of the policy. The secondary deformation measurement shall be initiated within 14 days and completed within 90 days after the occurrence of a disaster. For the facilities subjected to the secondary deformation measurement, the applications for disaster assessments shall be made based on the results of the primary and secondary deformation measurements.

(2) Initiation of Emergency Tide Level Observations and Resetting of Chart Datum Level

Resetting the control points and chart datum level is necessary for the execution of full-scale restoration work. It shall be noted that a delay in resetting will cause delays in the full-scale restoration survey and work. Particularly, emergency tide level observations need to be initiated at the earliest possible time because the resetting of chart datum level requires harmonic constants and other parameters, which are based on the tide level observation data from at least 32 days and nights. For the details of the emergency tide level observation method and the resetting of refer to **Reference (Part II), Chapter 2, 3.3 Resetting of Chart Datum Level for Full-Scale Restoration Projects**.

(3) Primary Deformation Measurement

The primary deformation measurement shall be implemented for the main purpose of observing the damage states of all facilities and preparing the materials necessary to apply disaster assessments. Thus, the primary deformation measurement shall be implemented with a focus on obtaining the minimum information (an estimation of the scale of the restoration work in accordance with the damage to the respective facilities) required for the disaster assessments.

Table 2.6.1 shows the work contents, examples of object facilities and points to consider in the primary deformation measurement.

Table 2.6.1 Work Contents, Examples of Object Facilities and Points to Consider in the Primary Deformation Measurement

Work content	Classification	Example of object facilities and points to consider
Control point survey	Initiation of emergency tide level observations Resetting of chart datum level	• Resetting of chart datum level using the results of emergency tide level observations (from 32 days and nights) (refer to **3.3**)
Onshore measurement	Measurement of deformation on breakwaters, quaywalls and seawalls above sea levels	• Measurement of the misalignment of normal lines, inclination and settlement of facilities • In the case of caisson type breakwaters, identification of the turnover, sliding and settlement of each caisson by measuring the positions and heights of at least four locations on each caisson • In the case of mooring facilities, identification of the misalignment of normal lines and unevenness by measuring at least four locations in each span
	Measurement of cavities in aprons	• Necessity of drilling for the final confirmation of cavities in aprons
Undersea measurement	Measurement of deformation on breakwaters, quaywalls and seawalls in the sea	• Necessity of confirming whether or not work quantities can be easily calculated from survey results in the case of using swath sounding machines (including the narrow multi-beam method; refer to **4.4**) because of the necessity of indicating work quantities in the application for disaster assessments • Necessity of confirmation by divers
	Measurement of scouring and siltation of navigation channels and basins	

(4) Secondary Deformation Measurement

Because the secondary deformation measurement is implemented for the main purpose of investigating the reasons for such damage, formulating a restoration design policy and resetting the design conditions to be a presupposition of the policy, the methods for the secondary deformation measurement differ depending on the locations and the levels of damage. **Table 2.6.2** shows the purposes, items and outlines of the secondary deformation measurement.

Table 2.6.2 Purposes, Items and Outlines of the Secondary Deformation Measurement

Large classification	Measurement purpose and item	Outline of measurement
Onshore measurement	○ Observation of accurate disaster states • Plane survey	• Plane survey of facilities with total stations
	○ Approximate evaluation of the residual bearing capacity of the sheet pile quaywalls • Measurement of the deformation tie rods on sheet pile quaywall	• Survey of horizontal and vertical displacement, looseness and corrosion states of tie rods
	○ Confirmation of the deformation of steel pipe piles and sheet piles • Measurement of the deformation of steel pipe piles and sheet piles	• Steel pipe piles: Survey of verticality and deformation through borehole radar and elastic wave measurements as well as video recordings in the pipe • Sheet piles: Survey of deformation using clinometers
	○ Confirmation of the residual bearing capacity of sheet pile quaywalls and pile piers • Measurement of the wall thicknesses of steel pipe piles and sheet piles	• Wall thicknesses: Using ultrasonic thickness indicators
	○ Evaluation of the residual bearing capacity of sheet pile quaywalls and piled piers • Residual bearing capacity survey (sheet pile quaywalls and piled piers)	• Steel pipe piles and sheet piles: Measurement of wall thicknesses • Tie rods: Lift-off test • Bearing piles: Dynamic bearing capacity test and static bearing capacity test
	○ Confirmation of the soundness of concrete structures • Crack survey	• Survey of cracks, peeling, falling, swelling and joint openings • Hammering test (as needed)
	○ Confirmation of the soundness of concrete structures • Concrete degradation survey	• Observation of the quality of existing concrete structures (compression strength, chlorine contents and neutralization)
	○ Prediction and determination of liquefaction • Soil survey	• Stratum structure, in-situ tests and laboratory soil tests

Large classification	Measurement purpose and item	Outline of measurement
Undersea measurement	○ Disaster states of seafloor topography • Bottom sediment survey	• Superficial bottom sediment: Side-scan sonars and lead line • Types of sediment: Sampling of bottom sediment
	○ Confirmation of the deformation of gravity-type and sheet pile quaywalls • Confirmation of the structural soundness in front of quaywalls	• Front sections of quaywalls: Survey of inclination, joint openings, damage and scouring (by divers, clinometers, underwater cameras, acoustic video cameras and underwater 3D scanners)
	○ Confirmation of the deformation of piled piers • Confirmation of the structural soundness at the bottom sections of the piled piers	• Cracks, peeling, falling and swelling of the lower faces of superstructures (beams and floor slabs) • Presence or absence of pile deformation (irregular geometry, buckling, etc.) (by divers, clinometers, underwater cameras, acoustic video cameras and underwater 3D scanners)
	○ Confirmation of the deformation of caisson type breakwaters • Confirmation of the damage situations of areas around the caissons	• Caissons exposed above sea levels: Survey of joint openings, sliding states, damaged states due to impacts, scattering of mound protection blocks and armor blocks, scouring immediately below the caissons, and cavities • Submerged caissons: Survey to determine the reusability of submerged caissons • Breakwater mounds: Continuous survey of mound damage states

(5) Surveys and Analyses for Formulating a Restoration Work Policy

In addition to the primary and secondary deformation measurements, the following surveys and analyses need to be implemented for formulating the main restoration work policies, including the priorities and orders of the respective facilities in the restoration work, and the resetting of the restoration designs based on the tsunami alleviation effects of breakwaters. It is necessary to initiate the following surveys and analyses at the early stages of the full-scale restoration survey because they require a significant amount of time for preparation and implementation.

- **Harbor calmness analysis:**

  Overall port restoration procedures shall be determined based on the evaluation results of the states of reduction in the cargo handling operation rates at the respective mooring facilities in ports due to the damage to the breakwaters and the levels of the restoration of the cargo handling operation rates for the respective restoration cases (with different orders of breakwater restoration). The cargo handling operation rates can be evaluated as harbor calmness (ratios of waves having wave heights measured at the front of quaywalls that do not exceed the critical wave height for cargo handling to all waves) calculated based on the conditions for setting the deepwater wave heights, the results of the wave transformation calculation (using the energy balance equation) up to the port entrances, and the wave height ratios (ratios of wave heights at the fronts of quaywalls to the deepwater wave heights) at the respective mooring facilities in the ports.

  The Takayama method[10] has been mainly used for calculating the wave height ratios. However, recently, in order to cope with a problem with reductions in harbor calmness due to long-period waves, another method using the Boussinesq equation, which is capable of simultaneously calculating the wave transformation and wave height distribution inside ports, has come into use.

- **Tsunami trace survey:**

  In the tsunami trace survey, the extent of inundation due to a tsunami in terms of the areas and depths is measured through the traces from the tsunami at major facilities which show the highest elevations that the tsunami reached (through indirect leveling from known elevations).

- **Inundation damage prediction analysis in the case of the collapse of breakwaters:**

  In the inundation damage prediction, the extent of inundation due to tsunamis and the associated damage are evaluated in a manner that conducts inundation simulations with conditions set by combining several states of the breakwaters, such as before, during and after construction, as well as when damaged by disasters and restored after disasters, with several tsunami heights, such as the design heights for restoration work and heights exceeding the design heights. Then, the evaluation results are used in the formulation of the restoration work policies. Specifically, the evaluation results are used for setting the design tsunami heights for the restoration of damaged breakwaters, the examination conditions to improve the durability of the breakwaters, and the new crown heights of the seawalls at the back of the breakwaters.

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3 Resetting of Chart Datum Level after a Large Earthquake and Tsunami

3.1 Measures Taken for the 2011 Great East Japan Earthquake and New Positioning Technologies

3.1.1 Damage Situations

The crustal movements due to the Great East Japan Earthquake (on March 11, 2011) put the existing control points, including local electronic control points and benchmarks, out of commission as known coordinates. In addition, all the tidal observatories (refer to **Fig. 3.1.1**) in the Tohoku region were completely disabled, including the tide level observation benchmarks (the level indicators that show level differences from the lowest levels [D.L.: Datum Line] at the tidal observatories). Under these circumstances, the Hydrographic and Oceanographic Department of the Japan Coast Guard deleted the reference tables of the mean, highest and lowest water levels at

almost all the ports located along the Pacific coast of east Japan from Hachinohe Port to Choshi Fishery Port, which were published on the Internet, on March 31, 2011.

Fig. 3.1.1 Example of a Damaged Tidal Observatory (Left: Before the Disaster, Right: After the Disaster)

3.1.2 Issues with Emergency Restorations

As mentioned above, the Great East Japan Earthquake put the existing control points and GNSS (GPS at that time) out of commission. Thus, the surveys of port facilities were implemented in a manner that set the temporary control points and proceeded with the survey work using temporary local coordinates. The bathymetric surveys immediately after the earthquake were implemented in a manner that established temporary benchmarks as references to calculate the water depths, set temporary chart datum level (temporary lowest levels) based on the estimated tide levels and measurements using auxiliary gauges (refer to **Reference [Part II], Chapter 2, 3.2 Emergency Setting of Chart Datum Level**), and temporarily used the relationships of the elevations between the temporary benchmarks and temporary chart datum level.

Because the bathymetric survey data is of particular importance for setting temporary water depths on which the elimination of obstacles from navigation channels is based, it is necessary to estimate the temporary chart datum level (reference levels that take into consideration the influences of the settlement or uplift of the ground due to crustal movements with a certain level of accuracy) as soon as possible after an earthquake. However, at the time of the Great East Japan Earthquake, there were cases where it took a long time before the temporary chart datum level were set because the method for estimating the levels in a short period of time after crustal movements (refer to **Reference [Part II], Chapter 2, 3.2 Emergency Setting of Chart Datum Level**) was not fully understood. Furthermore, there were cases of obstacles preventing the restoration work due to a lack of knowledge on the basic requirements of the tide level observation for 32 days and nights before resetting the chart datum level.

In addition, after waiting for the redistribution of renewed electronic control points by the Geographic Survey Institute after two and a half months had passed since the earthquake, the positional information on the temporary control points needed to be immediately converted to the Japanese Geodetic Datum (JGD) through control point surveys. At the same time, the vertical coordinates and the water depth values based on the temporary chart datum level (lowest levels), which were set using temporary tide level observations, also needed to be converted to the JGD in accordance with the survey results using the renewed electronic control points.

3.1.3 New GNSS Positioning Technology (PPP-AR System) and Its Utilization after Earthquakes

The advancements in positioning technology since the Great East Japan Earthquake make the GNSS survey capable of precise point positioning with ambiguity resolution (hereinafter referred to as “PPP-AR”) available with an accuracy of about ±5 cm, both in vertical and horizontal directions within a wide area of about 1,000 km from the control point network (worldwide control point network) when the conditions of the satellite positions at the time of the surveys and the visibility of the sky above the survey points are favorable (refer to **Reference [Part II], Chapter 2, 4.3.2 Precise Point Positioning with Ambiguity Resolution [PPP-AR]**). Thus, even in cases where all the existing control points and benchmarks fail over a wide range (all over the country), PPP-AR enables the temporary control points to be newly set immediately after an earthquake.

In addition, with the required information on the existing control points, such as Port B.M. and tidal observation B.M., preliminarily stored before the occurrence of an earthquake, PPP-AR enables the temporary chart datum level described in **Reference [Part II], Chapter 2, 3.1.2 Issues with Emergency Restorations** to be promptly set after an earthquake through the calculation (subtraction) of the differences between the coordinate values. The details of the setting of chart datum level with PPP-AR is described in **Reference [Part II], Chapter 2, 3.2 Emergency Setting of Chart Datum Level** below.

3.2 Emergency Setting of Chart Datum Level

3.2.1 Basic Procedures

This section describes the procedures to promptly set the temporary chart datum level required for the initial and emergency restoration surveys. It shall be noted that these procedures are based on the assumption that the GNSS PPP-AR survey is available after an earthquake.

The temporary chart datum level can be set through either **Procedure i** or **Procedure ii**, with the former being able to be implemented at an earlier stage in the restoration than the latter. **Fig. 3.2.1** illustrates the concepts of several types of values used in the detailed descriptions of the procedures below.

[Procedure i]

① Preliminarily acquisition of the following information on the existing control points (Point II in the figure) before the earthquake

- The ellipsoidal height of control point II (A in the figure) through a GNSS survey[1)]
- The level difference between control point II and the chart datum level (B in the figure) (refer to **Reference [Part II], Chapter 1, 2.3.5 Organization and Coordination of Tide Level Observation Data**)
- The ellipsoidal height of the chart datum level (C in the figure) based on the above height and difference

② Acquisition of the following information on the existing control point (II in the figure) after the earthquake

- The ellipsoidal height of control point II, identical to the above (A’ in the figure), through a GNSS (PPP-AR) survey after the earthquake

③ Setting of a temporary chart datum level

- Acquisition of the relationship in the heights between control point II and the chart datum level after the earthquake (B’ in the figure) (B’ = A’ − C)

[Procedure ii]

(*When a GNSS survey has not been implemented before the earthquake, or when setting a new control point after the earthquake)

① Acquisition of the following information on the existing or new control point (II in the figure) after the earthquake

- Acquisition of the ellipsoidal height of control point II (A’ in the figure) through a GNSS (PPP-AR) survey

② Visual observation of tide levels using a staff-gauge (hereinafter referred to as “staff-gauge observation”) after the earthquake (refer to **Reference [Part II], Chapter 2, 3.2.2 Staff-Gauge Observation and Method for Setting Temporary Chart Datum Level Using a Staff-Gauge**)

③ Setting of the temporary chart datum level (refer to **Reference [Part II], Chapter 2, 3.2.2 Staff-Gauge Observation and Method for Setting Temporary Chart Datum Level Using a Staff-Gauge**)

- Acquisition of the relationship in the heights between control point II and the chart datum level (B’ in the figure) after the earthquake using the staff-gauge observation results and the estimated tide levels at identical times (astronomical tides)

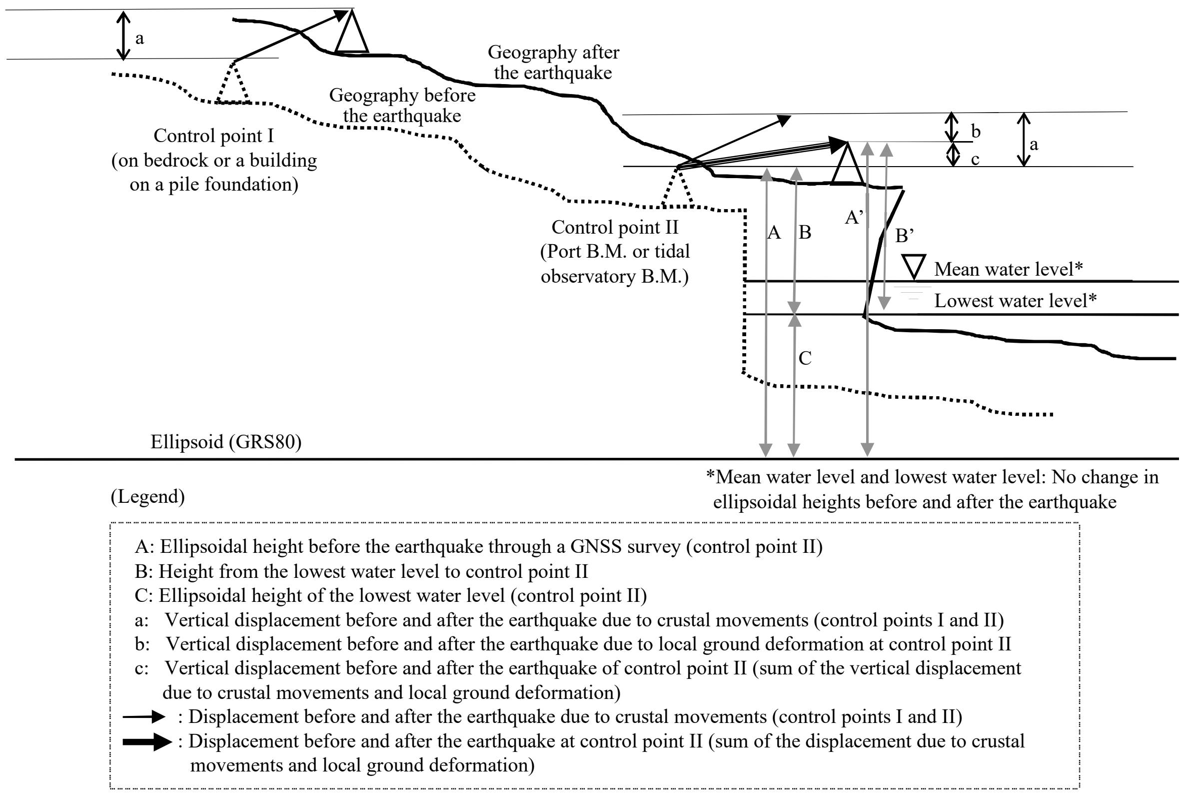


Fig. 3.2.1 Illustration of Setting the Temporary Chart Datum Level Using a GNSS PPP-AR Survey

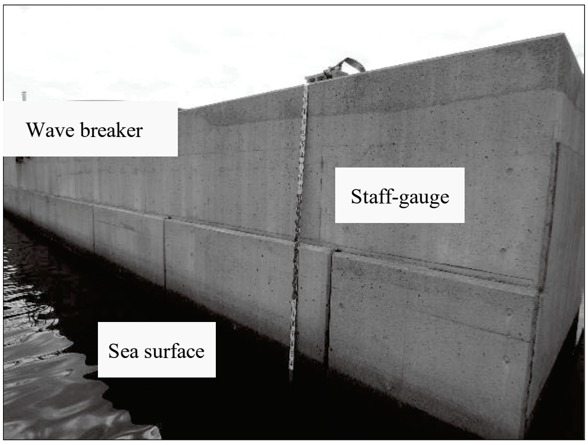
3.2.2 Staff-Gauge Observation and Method for Setting Temporary Chart datum level Using a Staff-Gauge

(1) Outline

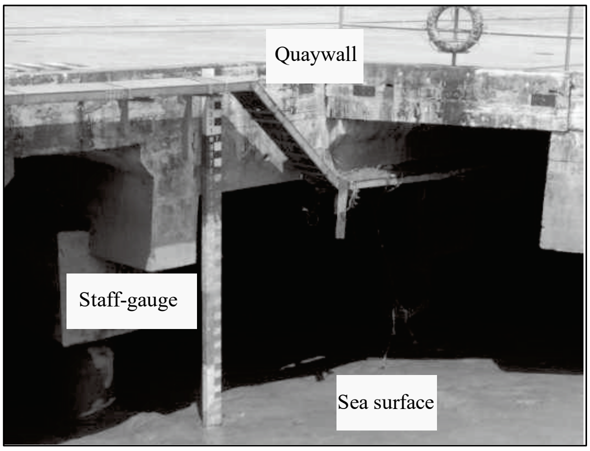
This section describes a specific method for calculating the relationship in the heights between control point II and chart datum level (B’ in the figure) after an earthquake, as required in **Procedure ii** in **3.2.1 Basic Procedures** above. This method is used to estimate the temporary chart datum level using the information on two types of tide levels: the visual tide level observation results using a staff-gauge, as described in Item (2) below, and the estimated tide levels (tide levels estimated from astronomical tides), as described in Item (3) below.

(2) Visual Tide Level Observation Using a Staff-Gauge (Staff-Gauge Observation)

Staff-gauges are staffs[2), 3)] with scales installed on quaywalls to observe the tide levels (refer to **Fig. 3.2.2**). Those installed near tide gauges for comparative observations are sometimes called “auxiliary gauges.” Observations of tide levels using staff-gauges are basically implemented through visual observations at time zones with calm hydrographic conditions. In the case of emergency situations, visual tide level observations shall be implemented for acquiring two or more sets of data, with the respective sets acquired in a manner that measures the tide levels for 30 minutes at 5-minute intervals before and after the highest and lowest water levels on different days. It shall be noted that observers need to reliably observe the tide levels at identical locations and eye heights.



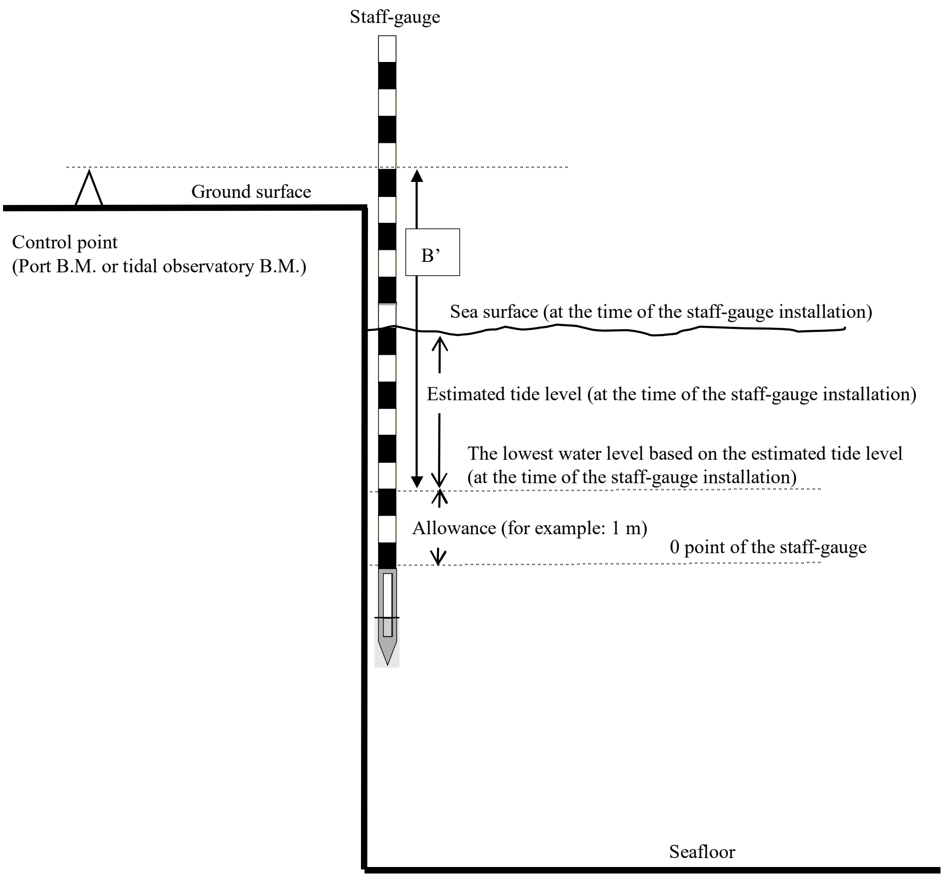
(a) Example of a Tide Level Observation on a Wave Breaker[2)]



(b) Example of a Tide Level Observation on a Quaywall[3)]

Fig. 3.2.2 Example of a Tide Level Observation Using a Staff-Gauge

When installing staff-gauges after an earthquake, it is necessary to measure the level differences between the staff-gauges and the onshore fixed points (e.g., control points including port B.M.) through direct leveling. In addition, the staff-gauges shall be installed with their 0 points positioned sufficiently low enough to enable the lowest water levels to be reliably measured. Specifically, staff-gauges can be installed with their 0 points set at the levels determined in a manner that first lowers the sea surfaces by heights corresponding to the estimated tide levels (with reference to the lowest water levels at the time the staff-gauges are installed) at the time they are installed, and then further lowers the levels by about 1 m as an allowance (refer to Fig. **3.2.3**).



B': Height between control point II and the port management datum level after the earthquake (Fig. 3.2.1)

Fig. 3.2.3 Schematic Drawing of a Visual Tide Level Observation Using a Staff-Gauge

(3) Estimated Tide Levels

The estimated tide levels mean the astronomical tides calculated based on the harmonic constants obtained for 60 tidal constituents using tide level observation data for at least one year. For the definition of the estimated tide levels, refer to **Reference [Part II], Chapter 1, 2.3.1 (4) Explanations of the Terms Related to Tide Level Observations**.

For the astronomical tides, the Japan Coast Guard has published hourly "sea level heights" in the form of a "tide table." The sea level heights mean the heights from the lowest water levels. When using the estimated tide levels after an earthquake disaster, it is necessary to acquire the times and sea level heights of the low and high tides from the tide table and interpolate the sea level heights between the low and high tides at 10-minute intervals.

In addition, when actually using the estimated tide levels, it is necessary to take into consideration the differences between the estimated tide levels and the actual tide levels. These differences are called "sea-level deviation." The magnitudes of the sea-level departures vary depending on the ocean areas, seasons, time zones and the presence or absence of disturbances due to typhoons; however, under normal meteorological and hydrographic conditions (that allow bathymetric surveys to be implemented), the sea-level departures are around ±20 cm or less, and within ±30 cm in most cases, except under abnormal meteorological conditions. The sea-level departures of the estimated tide levels also need to be considered when setting the temporary chart datum level. Considering that the sea-level departures are mainly affected by atmospheric pressure variations, the errors in the measurements of the sea-level departures can be reduced by averaging three pieces of data obtained on different days.

(4) Setting of Temporary Chart Datum Level

The temporary chart datum level can be set by comparing the results of Items (2) and (3) above, specifically by visualizing the results of both the staff-gauge observation through the method described in Item (2) and the estimated tide levels at the time of the staff-gauge observation through the method described in Item (3). After confirming that there are no major differences in the values and fluctuation patterns between the tide levels obtained through the staff-gauge observation and estimated tide levels, the lowest water levels based on the estimated tide levels are set as the temporary chart datum level.

3.3 Resetting of Chart Datum Level for Full-Scale Restoration Projects

3.3.1 Simplified Methods for Emergency Tide Level Observations

(1) Outline

After setting the temporary chart datum level as a provisional measure in the emergency restoration survey, the chart datum level shall be subjected to resetting for the implementation of full-scale restoration work. For resetting the chart datum level, continuous tide records shall be obtained with simplified observation equipment that is additionally installed. The chart datum level are normally set based on tidal data observed for one year or longer. In contrast, in the case of emergency tide level observations, the chart datum level are set based on tidal data observed for 32 days and nights.

The following two types of observation methods, (1) the ultrasonic method and (2) the hydraulic method, are introduced as typical methods for simplified tide level observations.

(2) Simplified Tide Level Observations through the Ultrasonic Method

The equipment for the ultrasonic method measures the water levels (tide levels) on a principle that takes the time required for the ultrasonic waves emitted from a detector installed on a quaywall, revetment or inner breakwater to reflect off the sea surface and return to the detector and converts it into distance (refer to **Fig 3.3.1**). The equipment can be mounted on a temporary trestle fixed to a revetment or other structure with anchor bolts, thereby enabling speedy installation of the equipment and early implementation of the tide level observations. The ultrasonic method has the advantage that the equipment can be easily maintained because it can measure the distance to the sea surface without touching the sea surface.[2] There are also cases of combining the ultrasonic method with visual observations using an auxiliary gauge to directly confirm the actual tidal variability.

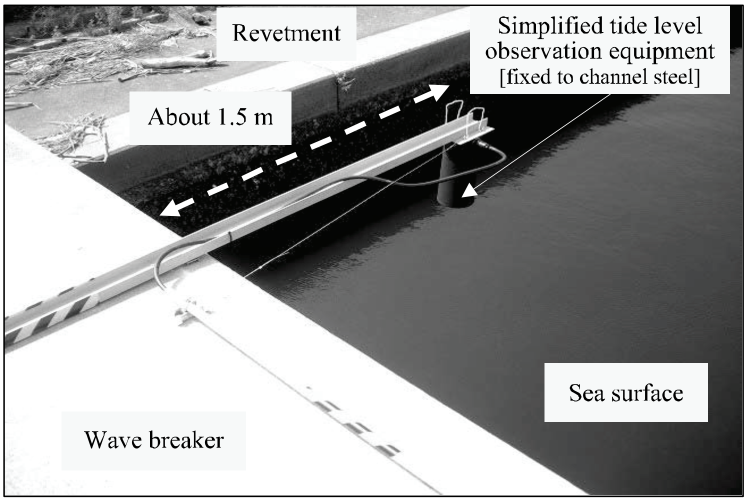


Fig. 3.3.1 Example of an Installation of Simplified Tide Level Observation Equipment (Ultrasonic Method)

(3) Simplified Tide Level Observations through the Hydraulic Method

The equipment for the hydraulic method uses a principle that takes the water pressure detected through pressure sensors preliminarily inserted into single pipes, which are attached to the side faces of the quaywalls, revetments or inner breakwaters, or fixed to the seafloor if possible, and converts it into water levels (refer to **Fig. 3.3.2**). Due to having an advantage in enabling speedy installation of the equipment and early implementation of the tide level observations, the hydraulic method was used in the tide level observations after the 2011 Great East Japan Earthquake. In the hydraulic method, it is necessary to calculate tide level data through reduction rate correction (a comparison between the values of the water pressure sensors and readings of the auxiliary gauges) based on the results of a continuous auxiliary gauge observation for 8 hours or longer at spring tides.

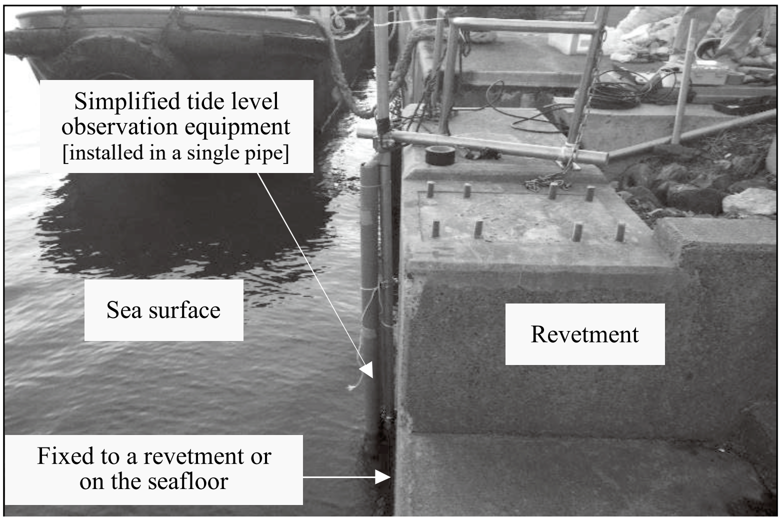


Fig. 3.3.2 Example of an Installation of Simplified Tide Level Observation Equipment (Hydraulic Method)

[References]

1) R.Naito, T. Asai, K.Kawaguchi, T. Inomata, D.Tatsumi and K. Narita: Estimation of Sea Level Rise Using the Tide Gauge Records in Port Areas and Their Characteristics, TECHNICAL NOTE of National Institute for Land and Infrastructure Management, No.855. 2015. (in Japanese with English synopsis)

2) R. Naito, K. Kumagai, T. Suzuki and T.Suzuki(2017): A field experiment for the simple measurement of tide level using ultrasound, TECHNICAL NOTE of National Institute for Land and Infrastructure Management, No.959.(in Japanese with English synopsis)

3) Japan International Cooperation Agency (2016): Final Report on Data Gathering and Identification for Improving Waterways of Yangon Port in Myanmar, No.2, Observation Data Sheets. (in Japanese)

4 Survey Methods after a Large Earthquake and Tsunami

4.1 General

Depending on the stages of the survey after a large earthquake and tsunami, there may be cases where identical survey methods are used for different purposes and scopes, which require different degrees of accuracy. **Table 4.1.1** summarizes the survey purposes at each stage from the initial stages to full-scale restoration stages with respect to seven survey objects throughout the three stages.

Table 4.1.1 Survey Objects and Survey Purposes at Each Stage

Object	Purpose		
	Initial stage	Emergency restoration stage	Full-scale restoration stage
Understanding the Overall Damage Situation (**4.2**)	General understanding of the overall damage situations of onshore facilities and offshore facilities above sea surfaces in ports	Understanding of the distribution of floating objects for the determination of the soundness of ports and the elimination of port obstacles	
Understanding the Geometry of Onshore Areas (**4.3**)	Understanding of the damage situations and determination of the usability of each port facility	Understanding of the scale of the emergency restoration work for quaywalls and cargo handling equipment	Acquisition of basic information for the restoration design of each facility Determination of the reasons for the damage
Understanding the Geometry of Underwater Areas (**4.4**)	Understanding of the distribution of debris on the seafloors of navigation channels and basins as well as in front of quaywalls Determination of available water depths for the temporary use of these facilities	Confirmation of the safety of waterways and basins, including navigation channels Understanding of the scale of emergency restoration work for mooring facilities, including piled piers	Acquisition of basic information for the restoration design of each facility Determination of the reasons for the damage
Understanding Broad-Based Ground Deformation (**4.5**)		Understanding of the tendencies of ground deformation in wide areas, including ports	Understanding of the tendencies and terminations of crustal movements in wide areas Understanding of fault structures
Surveys on Ground Liquefaction (**4.6**)		Confirmation of the safety of quaywalls, including aprons and yards Understanding of the scale of emergency restoration work for these facilities	Determination of the reasons for the damage Acquisition of basic information for the restoration design of each facility
Understanding the Cavities at Apron Sections (**4.7**)		Confirmation of the safety of quaywalls, including aprons and yards Understanding of the scale of emergency restoration work for these facilities	Determination of the reasons for the damage Acquisition of basic information for the restoration design of each facility
Understanding the Deformation on Underground Structures (**4.8**)			Determination of the reasons for the damage to underground structures, such as the steel piles of piled piers Acquisition of basis information for the restoration design of each facility

4.2 Understanding the Overall Damage Situation

4.2.1 General

When disaster occurs due to large earthquakes and tsunamis, it is necessary to promptly establish initial reaction systems. In order to understand the overall damage situation due to a large-scale disaster. At ports, it is necessary to have a general understanding of the overall damage situation of the onshore and offshore facilities above sea surfaces at the initial survey stage. Furthermore, at the emergency restoration survey stage, it is necessary to understand the usability of ports and distributions of floating objects to determine navigation routes.

Table 4.2.1 shows main survey methods for understanding the overall damage situation and its characteristics. Immediately after a disaster, visual observations such as note taking and sketching, and taking photos using cameras are major survey methods to assess the damages. Although these methods are simple, they are not suitable for observing damage situations in wide areas and are not available for observing damage situations at inaccessible locations. In contrast, aerial photography using UAVs (Unmanned Aerial Vehicles) or radio control balloons can be employed for understanding the damage situations at inaccessible locations. Similarly, satellites imagery and aerial photography can be employed for understanding the damage situations in wide areas.

The following sections describe the **utilization of satellite imagery (4.2.2)**, **aerial photography using aircraft (4.2.3)**, **aerial photography using UAVs (4.2.4)**, and **aerial laser surveys (4.2.5)**.

Table 4.2.1 Main Survey Methods for Understanding the Overall Damage Situation and the Characteristics of the Methods

Main survey method	Initial stage	Emergency restoration stage	Characteristic
Visual observation, note taking and sketching	○		Simple method available on site Not suitable for wide-area observations
Cameras and video cameras	○	○	Simple method available on site Not suitable for wide-area observations
Satellite imaging (**4.2.2**)	○	○	Implementation when disaster charter is activated Availability of a variety of information obtained through a variety of sensors
Aerial photography using aircraft (**4.2.3**)	○	○	Implementation when disaster charter is activated Availability of detailed and extensive information
Aerial photography using UAVs (**4.2.4**)	○	○	Availability of detailed information Suitable for the observation of narrow areas
Aerial photography using radio control balloons	○		Availability of detailed information Suitable for the observing small areas Longer airborne duration than UAVs
Aerial laser surveys (**4.2.5**)	○	○	Suitable for the observation of wide areas Availability of the detection of geography below trees

4.2.2 Utilization of Satellite Imagery

(1) Outline

Considering the difficulty in accessing damaged areas immediately after a large-scale disaster, satellite imagery can be used effectively to understand the damage situations in wide areas, such as entire port and its surrounding water areas. By comparing pre- and post- disaster satellite imagery, satellite imagery provides variety of useful information for understanding damage situations in wide areas shown in Item (3).

In the case of the 2011 Great East Japan Earthquake, imagery captured by X-band synthetic aperture radar satellite (TerraSAR-X) and numerous optical satellites were used for analyzing (estimating) inundated areas and monitoring submerged areas due to the tsunami.[1)]

Currently, more than 20 satellite types are available in Japan, and satellite ground stations (antennas) have been in operation to quickly provide post-disaster information. Also, satellites have been used for surveying and remote sensing application services such as disaster prevention, agriculture, forestry and environment. These services have been developed by analyzing and processing satellite imagery.[2).]

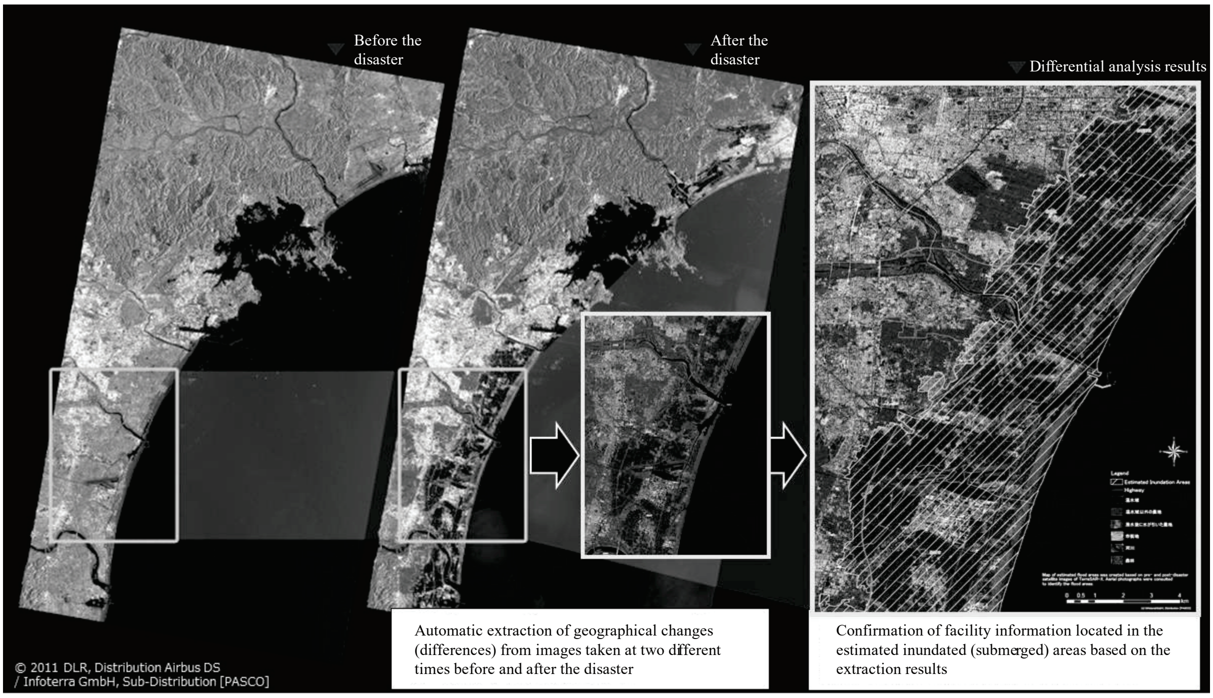


Fig. 4.2.1 Estimation of the Areas with Inundation Damage
(at the Time of the Great East Japan Earthquake in March 2011)[1)]

(2) Types and Characteristics of Observation Satellites

Earth observation satellites are classified into two types: (a) optical satellites and (b) SAR satellites. **Table 4.2.2** shows the usability and quick response capability of satellite images, and **Table 4.2.3** shows the characteristics of each type of satellite.

Table 4.2.2 Usability and Quick Response Capability of Satellite Images[3)]

Wide areas	Instantaneously capturing imagery of wide areas	Effective means to acquire ground surface information of relatively wide areas without time lag
Reliablility	Capable of capturing images in bad weather	Increasing use of SAR satellites, which can capture images without being affected by weather conditions
Quickness	Quickly conduct change detection	By using pre-disaster archive imagery, changes can be quickly extracted by comparing imagery from two different acquisition date.

Table 4.2.3 Characteristics of Optical and SAR Satellites

Optical satellite	SAR satellite
○ Maximum resolution: 50 cm ○ Availability of color images × Cannot capture imagery under cloudy conditions and at night ○ Normally imagery is captured once a day ◎ Familiar imagery (can be understood by looking at them)	○ Resolution: 1 m, 0.25 m at most ○ Monochrome images only ◎ Capture imagery under rainy or cloudy conditions and at night ◎ Imagery is captured twice a day (morning and evening) ◎ Quickly conduct quantitative change extraction by semiautomatic analysis of two imagery taken at two different times pre- and post- disaster

*SAR: Synthetic Aperture Radar

(a) Optical Satellites

Currently, almost all commercially available digital map data has been created through image data obtained by aerial photography. Since it takes roughly 10 years to collect aerial photography for an entire area of Japan, it is nearly impossible to acquire or update the map data in within a certain time period.. In contrast, optical satellites can conduct data acquisition for an entire area of Japan within a month, thus ground surface change information can be extracted in a short period of time, resulting in fundamental changes in a way digital map is updated and maintained. In order to move towards an establishment of advanced digital map society, optical satellite has been used to create and update high resolution and updated digital map data..[4)]

(b) Synthetic Aperture Radar (SAR) Satellites

SAR satellites can be useful for quickly understanding the extensive damage of tsunamis and typhoons. In the cases of the Sumatra Earthquake and the Niigata Chuetsu Earthquake, it took a long time to assess damage situations in a secluded coastal and intermountain regions, therefore resulting in long delay in emergency response in those regions. SAR satellites are capable of measuring changes in ground surfaces during the day and night, and are not affected by weather conditions which makes it suitable for capturing damage situations and determine proper emergency measures. SAR satellites can also be used for monitoring crustal movements, ground subsidence and active faults for predicting volcanic activities and earthquakes.[5)]

(3) Obtainable Information and Methods for Utilization

By using satellite imagery, topics ① through ⑤ provides variety of information to assess the damaged situation for wide areas (mainly ports and the surrounding water areas).

① Understanding present damage situation of port facilities (mainly by using optical and SAR satellites)

- Assessing damage situation of port facilities and surrounding areas from wide swath imagery
- Assessing damage situation of facilities such as breakwaters and quaywalls, through wide swath imagery
- Assessing present obstructive situation of navigation channels due to the accumulation of debris and floating objects
- Assessing situation of the settlement of quaywalls and wharf areas

② Estimating damaged and inundated areas (mainly by using optical and SAR satellites)

- Automatic extraction of geographical changes (differences) from two imagery taken at two different times pre- and post-disaster to estimate the damaged areas
- Understanding the estimated inundated (submerged) areas through images regularly taken after the disaster

③ Estimating the amount of debris and floating objects (mainly by using optical satellites)

- Understanding the amount of debris based on the damaged areas extracted and estimated through images regularly taken after the disaster

④ Comparing port facilities pre- and post-disaster (mainly by using optical satellites)

- Detailed extraction of the misalignment of the normal lines of facilities from two imagery taken at two different times pre- and post- disaster to estimate the damaged areas to determine the usability of these facilities

⑤ Assisting in field surveys (mainly by using optical and SAR satellites)

- Assistance in the determination of the prioritized locations for the field surveys until the tsunami warnings and advisories are lifted

Differences in use of data obtained through optical and SAR satellites are as follows. Imagery obtained by optical satellites can be used to assess the extent of the damages from the disaster. Although the resolution of a SAR imagery is inferior to that of optical satellites, SAR satellites can be used even in a bad weather. It can be used to understand changes in land coverage and ground deformation by analyzing two imagery taken at two different times pre- and post- disaster.

4.2.3 Aerial Photography Using Aircraft

(1) Outline

Considering the difficulty in accessing damaged areas immediately after the occurrence of a large-scale disaster, aerial photography using aircraft can be effectively utilized for understanding the damage situations in wide areas (entire ports and the surrounding water areas) immediately after the occurrence of a disaster. Basically, images shall be taken by oblique and vertical shooting using digital cameras. Digital surface models (DSM) and digital elevation models (DEM) obtainable through concurrent implementation of the aerial laser measurement (surveying) to be described in **Reference [Part II], Chapter 2, 4.2.5** with aerial photography enable the amount of debris to be estimated.

With aerial photography using aircraft, the types of cameras generally used are handheld cameras for oblique shooting and dedicated aerial cameras (vertical cameras and oblique cameras). The oblique cameras can perform vertical and oblique shooting concurrently.

(2) Obtainable Information

The results of the aerial photography, or the results of the aerial laser measurements implemented concurrently with the aerial photography, can be used for obtaining the same information as that obtainable through optical satellite photography among a variety of information for understanding the damage situations (**Reference [Part II], Chapter 2, 4.2.2 Utilization of Satellite Photography (3**) in wide areas (entire ports and the surrounding water areas) immediately after a disaster. However, these results cannot be used for the automatic extraction of inundated areas through multi-spectrum analyses, unlike in the case of optical satellite photography.

The following section describes the types of images and an outline of the information specific to aerial photography using aircraft.

① **Vertical images**

Aerial images are generally vertical images taken using dedicated cameras. These vertical images are texture images of vertical planes, which are generally roads and the roofs of buildings. The vertical images taken in the course of a survey after a large earthquake and tsunami can be used for understanding the outline of the damage situations of the entire port (collapse and inundation situations and positional relationships).

② **Oblique images**

Oblique images are taken in oblique directions from aircraft using handheld cameras or dedicated cameras (oblique cameras). These oblique images are bird's-eye images, which are advantageous in making positional relationships of the subjects with the surrounding objects, and the sizes as well as three-dimensional shapes of subjects easily understood. As with Item ① above, the oblique images can be used for understanding the outline of the damage situations of the entire port (collapse and inundation situations and positional relationships).

Fig. 4.2.2 Example of a Vertical Image (the Ofunato area at the time of the Chilean Earthquake in 1960)[6)]

Fig 4.2.3 Example of an Oblique Image (Kesennuma City, Miyagi Prefecture, at the time of the Great East Japan Earthquake)[7)]

③ **Orthophotos and three-dimensional models**

Orthophotos are composite images created by connecting vertical images with identical scales so as to cover wide areas and using elevation data when vertical images are taken with the positions of the respective vertical images adjusted through aerial triangulation. In many emergency cases, the processing time to generate orthophotos is shortened by using the data of GNSS/IMUs (the measuring positions of aircraft) and elevation data.

Orthophotos can be used for directly generating three-dimensional point groups, which can be output as three-dimensional models. Then, using the three-dimensional models as the latest elevation data, updated orthophotos can be generated. In addition, the three-dimensional models can be used for three-dimensional measurements (of the amounts of debris, etc.) in damaged areas.

Fig. 4.2.4 Example of an Orthophoto (Ishinomaki City, Modification of Reference 8))

4.2.4 Aerial Photography Using UAVs (Unmanned Aerial Vehicles)

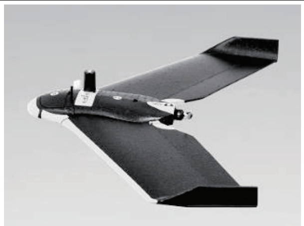
(1) Outline

Considering the difficulty in accessing damaged areas immediately after the occurrence of a large-scale disaster, aerial photography using UAVs (also called drones or UASs [Unmanned Aerial Systems]) is effective in understanding the damage situation in relatively small areas (a few ha to a few km^2).

Recently, aerial photography using UAVs has been used for researching and surveying object areas smaller than those of aerial photography using aircraft. There have also been cases of coastal surveying through aerial photography using UAVs.[9)] Aerial photography using UAVs is particularly effective in observing those places in ports which need to be surveyed urgently, cannot be observed directly (due to obstacles blocking access), and are difficult to access (such as offshore breakwaters).

Table 4.2.4 shows the types of UAVs and their outlines.

Table 4.2.4 Outlines of Several Types of UAVs

Type	Rotating wing		Fixed wing
	Single rotor	Multiple rotor	
Photograph			
Power source	Gasoline	Motor	Gasoline or motor
Flight performance	• Limitation in flight altitudes • Longer flight time and larger payload (can carry a large amount of weight) compared to the battery type	• High flight stability • Can be used at high flight altitudes (1000 m or higher) • Requires an onboard battery, which is susceptible to cold temperatures and reduces flight times	• Longer flight time and distances compared to rotating wing types • Requires a large area for taking off and landing

(2) Obtainable Information

Aerial photography using UAVs can be used for obtaining the same information as that obtainable through optical satellite photography among the variety of information for understanding the damage situations (**Reference [Part II], Chapter 2, 4.2.2 Utilization of Satellite Photography (3**) in relatively small areas (a few ha to a few km^2) immediately after a disaster. However, aerial photography using UAVs cannot be used for the automatic extraction of inundated areas through multi-spectrum analyses, unlike in the case of optical satellite photography.

The types of information specific to aerial photography using UAVs are as follows.

① **Aerial photographs and videos**

In addition to aerial photographs, aerial videos become available by changing the types of cameras mounted on the UAVs. Although the image quality is not favorable due to band limitations, real-time videos can be observed on the ground by communicating with the UAVs.

② **Three-dimensional models and orthophotos**

The photographs taken with UAVs can be used for generating three-dimensional models and creating orthophotos through photographic surveying. Owing to the popularization of software using SfM (Structure from Motion), aerial photographs can be easily processed into three-dimensional point group data and three-dimensional models with texture images.[10)] UAVs enable the three-dimensional models of locations under overhangs to be generated, as shown in the right side of **Fig. 4.2.5**, by photographing these locations sideways. Orthophotos can also be used for photomaps of damaged areas. In addition, the three-dimensional models generated from orthophotos can be used for measuring the damage situations (e.g., the inclinations of buildings).

Fig. 4.2.5 Example of a Three-Dimensional Model of a Coastal Cliff[11)]

4.2.5 Aerial Laser Surveys

(1) Outline

Considering the difficulty in accessing damaged areas immediately after the occurrence of a large-scale disaster, laser surveys using aircraft (aerial laser surveys) are effective for understanding the damage situations in wide areas (entire ports and the surrounding water areas) immediately after a disaster.

In aerial laser surveys, the sensor units (laser scanners, GPS/IMUs and digital cameras) mounted on board aircraft are used for acquiring laser point group data (elevation data) of wide areas by combining three-dimensional data (DSM/DEM) with two-dimensional digital images taken with digital cameras. The laser point group data can be processed into microtopography three-dimensional maps and sedimentation variation distribution maps. Furthermore, in surveys after a large earthquake and tsunami, comparing the laser point group data before and after the disaster enables the amounts of debris to be estimated and ground subsidence to be confirmed.

In addition, there are cases where the damage situations of shallow water areas around port facilities become obtainable by combining general aerial lasers with ALB (Airborne LiDAR Bathymetry).

(2) Obtainable Information

① **Three-dimensional terrestrial topographic information**

Aerial laser surveys are capable of acquiring three-dimensional topographic information of the ground surfaces even when the ground surfaces are covered with vegetation such as trees. **Fig. 4.2.6** shows an example of a digital surface model (DSM) representing the ground surface information, including trees, and a digital elevation model (DEM) representing topographic information. DSMs and DEMs are generally used as two-dimensional images represented by gradient tint maps, shaded-relief maps and red-relief maps.

In the case of the surveys after a large earthquake and tsunami in ports, DSMs can be directly used for understanding the damage situations of buildings and distribution patterns of debris. **Fig. 4.2.7** shows an example of the measurement of the damage situation (washing out and inundation) in Ishinomaki Port after the Great East Japan Earthquake.

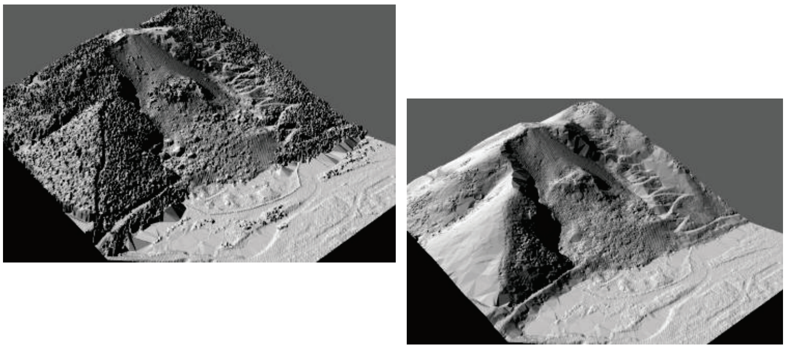


Fig. 4.2.6 DSM (Left) and DEM (Right)[12)]

Fig. 4.2.7 Example of an Aerial Laser Survey in a Port (Vicinity of Ishinomaki Port, Modification of the Port in Reference 8))

② **Sounding information in shallow water areas through ALB**

Seafloor topography and obstacles to ship navigation in shallow water areas can be measured by irradiating the seafloor with both normal near-infrared pulses used in laser measurements and green pulses, which transmit through seawater. In addition, water depths can be calculated as the differences between the elevations measured through the pulses reflected on sea surfaces and those reflected on the seafloor. Although there may be cases where ALB cannot be used in the water areas around port facilities immediately after a disaster, because measurements with ALB are subjected to the wave conditions, ALB is generally capable of measuring water depths 1.5 times the underwater visibility. ALB is also capable of measuring seafloor topography of extremely shallow water areas (-3 m or less) where measurements with narrow multi-beam echo sounders are not available or are inefficient (**Fig. 4.2.8**).

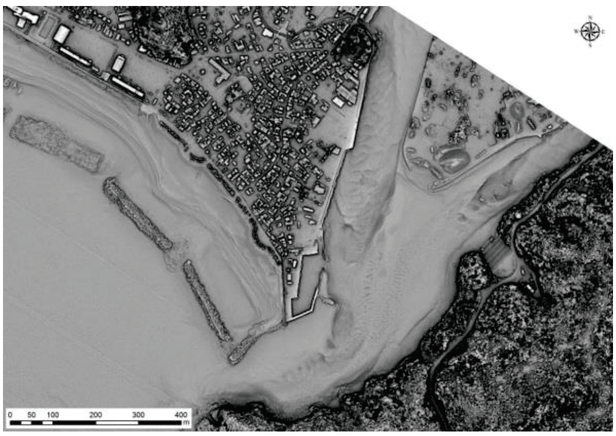


Fig. 4.2.8 Example of a Measurement with ALB
(at the Mouth of the Hioki River, Shirahama Town, Wakayama Prefecture)[13)]

4.3 Understanding the Geometry of Onshore Areas

4.3.1 General

In order to utilize ports as disaster relief bases after a large earthquake and tsunami, it is first necessary to understand the damage situation of the onshore facilities in the ports. For the requirements of the three survey stages, a general understanding of the damage situation of the port facilities, a determination of the usability of the quaywalls and cargo handling equipment, and the acquisition of basic information for the restoration design and investigation of the reasons for the damage are required in the initial, emergency restoration and full-scale restoration stages, respectively.

Table 4.3.1 shows the main survey methods for understanding the damage situation of the structures above ground, the geography and the geometry of the structures. In the initial stage, an understanding of the damage situations is made mainly through visual confirmation, cameras and video recordings, as well as by shooting images with UAVs. In the emergency restoration stage, measurements using staffs and rods are additionally used for understanding the damage situations. In the full-scale restoration stage, a topographic survey is additionally implemented to understand the geometry of the structures above ground. The deliverables of the topographic survey in this stage need to be as accurate as those of a general topographic survey when the situation is normal.

It is expected that extensive crustal movements cause the existing control points to be put out of commission when topographic surveys are to be implemented after a large earthquake and tsunami. Thus, in the following section, based on the assumption that new control points are set through satellite positioning for a topographic survey in the full-scale restoration stage after an earthquake, outlines of the satellite positioning methods and utilization methods of the respective satellite positioning methods after an earthquake are described in the order of: **4.3.2 Precise Point Positioning with Ambiguity Resolution (PPP-AR)**; **4.3.3 Real-Time Kinematic Survey**; and **4.3.4 Network-Type VRS-RTK-GNSS**.

Table 4.3.1 Survey Methods for Understanding the Geometry of Structures above Ground

Initial stage	Emergency restoration stage	Full-scale restoration stage		
Visual confirmation, note-taking, cameras, video recordings and imaging with UAVs	Visual confirmation, note-taking, cameras, video recordings, imaging with UAVs, and measurements with staffs, rods, pin holes and levels	Topographic survey	Control point survey	PPP-AR
			Plane survey	Total station (TS), RTK method, UAV, VRS-RTK method*
			Cross-section survey	TS, leveling instruments (auto levels, etc.)
			Settlement survey	Leveling instruments (auto levels, etc.)
			Displacement survey	TS, laser surveys

*: Not available in the event of the occurrence of crustal movements

4.3.2 Precise Point Positioning with Ambiguity Resolution (PPP-AR)

(1) Outline

The PPP-AR is a positioning method capable of point positioning by receiving precise orbit information and clock information from GNSS satellites as correction data in real time, and measurements with an accuracy of ±5 cm in vertical and horizontal directions under favorable conditions (**Fig. 4.3.1**).

The correction data enables highly precise positioning data to be obtained in an area with a 1000-km radius with a local control point not affected by crustal movements as its center. The time required for measuring a position is 30 minutes for the initial setting and another 20 minutes for satellite observation.

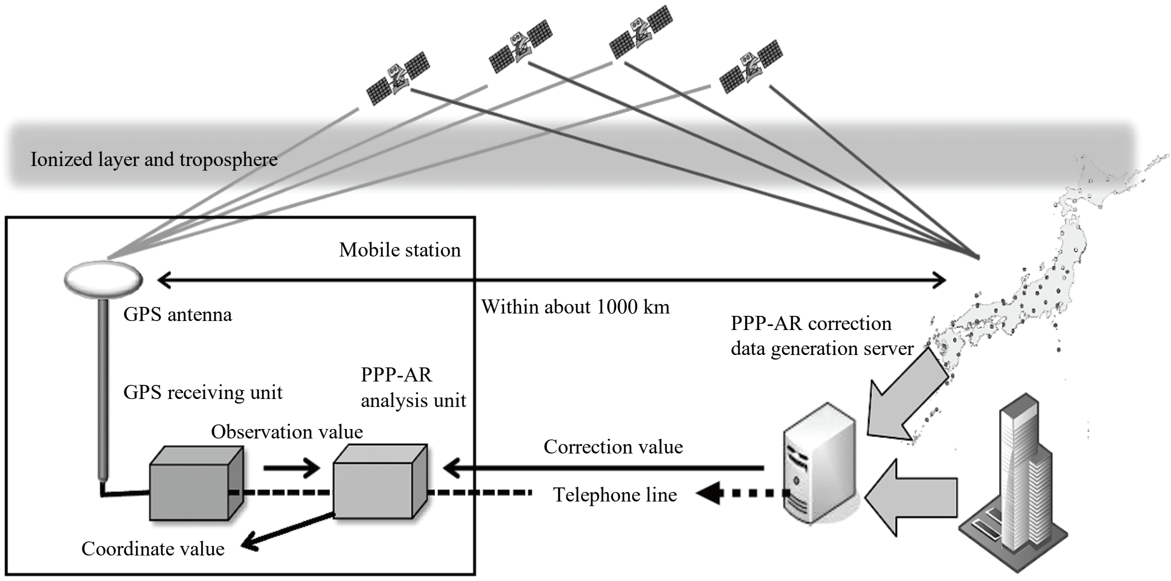


Fig. 4.3.1 Outline of Precise Point Positioning with Ambiguity Resolution (PPP-AR)

(2) Use of the PPP-AR after a Large Earthquake and Tsunami

① The positioning results of the PPP-AR can be used for setting temporary control points immediately after an earthquake even when the existing control points, including electric control points and benchmarks, are out of commission due to crustal movements (refer to **Reference [Part II], Chapter 2, 4.3.3 Real-Time Kinematic Survey**).

② The PPP-AR enables the coordinates (JGD) and elevations (CDL) of the main control points to be easily established even on facilities isolated from land, such as breakwaters.

③ The PPP-AR enables the elevation observation of B.M. installed near temporary tidal observatories.

④ The equipment of the PPP-AR has not been used in general surveys because there have been no provisions pertaining to the PPP-AR in the public survey work rules. In order to use the PPP-AR method in emergency cases, it is necessary to prepare the facilities and receive training.

4.3.3 Real-Time Kinematic (RTK) Survey

(1) Outline

As shown in **Fig. 4.3.2**, the RTK survey obtains a baseline vector between a known point (control point: base station) and a new point (mobile station) based on corrected observation data wirelessly transmitted from the known point and the GNSS radio waves acquired at the new point, instantaneously calculates the coordinates of the new point, and displays the observed data on a GNSS controller mounted on the mobile station. The observation accuracy of the RTK survey is ±2 to 5 cm and ±5 to 7 cm in horizontal and vertical directions, respectively.

Because the RTK survey requires the wireless transmission of data from the base stations, the availability of high accuracy measurements is limited to a relatively small range (with a radius of about 500 m) from the base stations. The time required for measurements through the RTK survey is 1 minute for the initial setting and another 10 seconds for satellite observations. Thus, the RTK enables observations to be performed in real time.

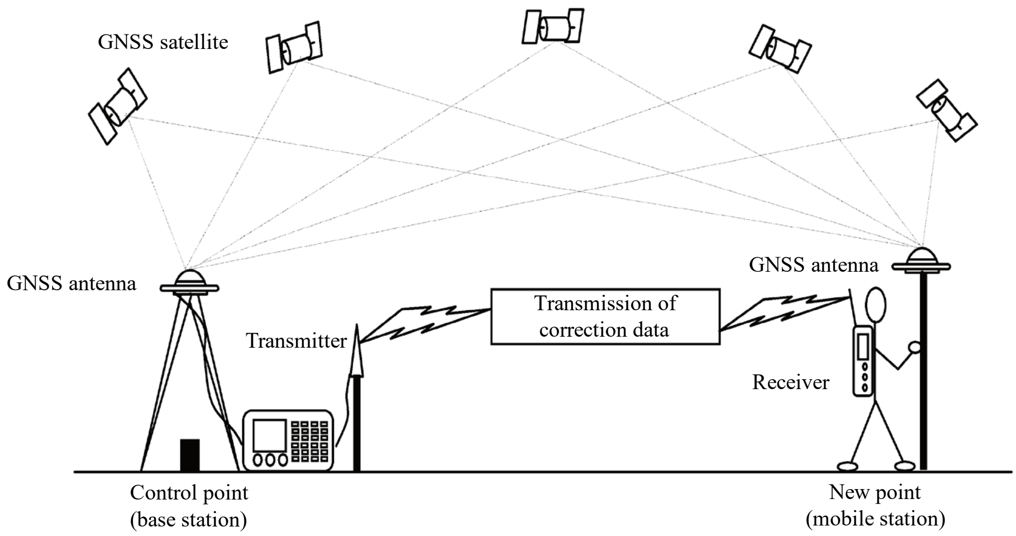


Fig. 4.3.2 Schematic Drawing of an RTK Survey[14)]

(2) Utilization of the RTK Survey after a Large Earthquake and Tsunami

① The RTK survey cannot be used when the control points near the damaged areas are out of commission due to crustal movements or damage caused by the earthquake, as was the case with the Great East Japan Earthquake. However, the RTK survey becomes available by using temporary control points installed through the PPP-AR as base stations, as described in **Reference [Part II], Chapter 2, 4.3.2 Precise Point Positioning with Ambiguity Resolution (PPP-AR)**.

② The RTK survey system under development by the Port and Airport Research Institute will enable the relative positional relationships between the control points and port facilities to be measured regardless of the presence or absence of ground deformation, thereby allowing for displacement to be measured for determining the usability of the quaywalls, even immediately after the earthquake.[15)]

4.3.4 Network-Type VRS-RTK-GNSS (Virtual Reference Station-Real-Time Kinematic-Global Navigation Satellite System) Survey

(1) Outline

In a network-type VRS-RTK-GNSS survey, the virtual control points around the mobile stations are established (in software) from the data on the electronic control point networks if no control points exist within 20 km of the mobile stations. The network-type VRS-RTK-GNSS survey can calculate the positions of new points in real time in a manner that transmits the approximate positions of the new points to a distributor through a mobile telephone circuit, receives the observation data and correction information for the virtual control points, and performs base line analyses using the virtual control points and new points (**Fig. 4.3.3**). The observation accuracy of the network-type VRS-RTK-GNSS survey is about ±3 to 5 cm in a horizontal direction and about ±5 to 10 cm in a vertical direction under favorable conditions.

With the virtual control points automatically set around the mobile stations, the network-type VRS-RTK-GNSS survey enables wide-area positioning to be performed.

The time required for a measurement through the network-type VRS-RTK-GNSS survey is about 1 minute for the initial setting and another 10 seconds for satellite observations. Thus, the RTK enables observations to be performed in real time.

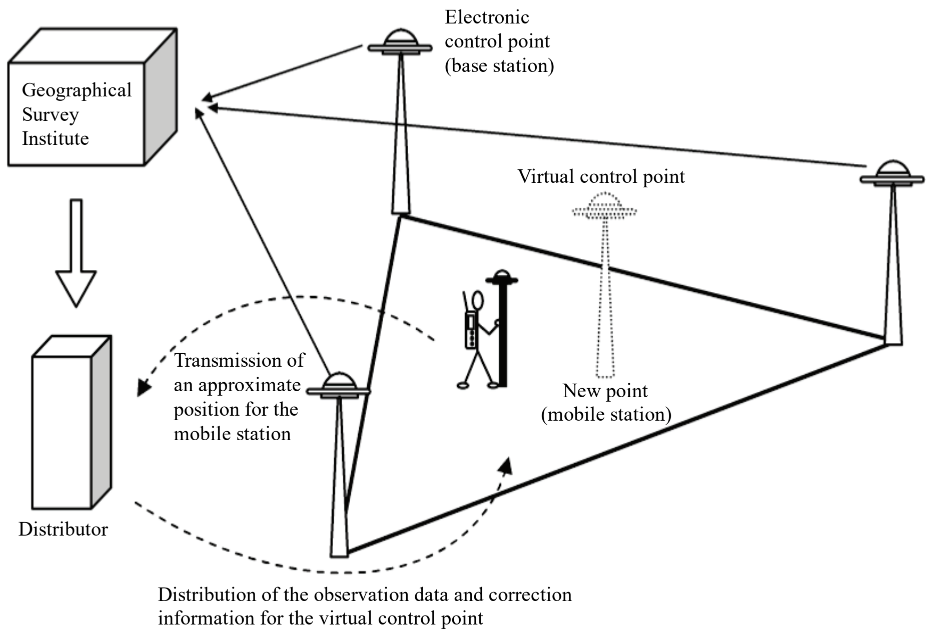


Fig. 4.3.3 Schematic Drawing of a Network-Type VRS-RTK-GNSS Survey[14)]

(2) Utilization of the Network-Type VRS-RTK-GNSS Survey after a Large Earthquake and Tsunami

① The network-type VRS-RTK-GNSS survey cannot be used when the electronic control points near the damaged areas are out of commission due to crustal movements or damage caused by earthquakes, as was the case with the Great East Japan Earthquake (because the Geographical Survey Institute stopped data distribution).

② In the case of the Great East Japan Earthquake, the Geographical Survey Institute resumed the distribution of data on new electronic control points two and a half months after the earthquake.

4.4 Understanding the Geometry of Underwater Areas

4.4.1 General

In order to utilize ports as disaster relief bases after a large earthquake and tsunami, it is first necessary to determine the damage situation and debris accumulated on the seafloor of the navigation channels, basins and underwater sections of the breakwaters and mooring facilities. In the initial and emergency restoration stages, surveys are required for confirming the safety of waterways and basins, including navigation channels, and determining the usability of mooring facilities, such as piled piers and sheet pile quaywalls. In the full-scale restoration stage, surveys are required for the acquisition of basic information for the restoration design and investigation of the reasons for the damage.

The methods to understand the underwater conditions and water depths are based in optical devices (e.g. still and video cameras) that collect images, and swath sounding devices (e.g. narrow multibeam swath sounders) that measure water depths, in addition to the visual confirmations by divers and lead sounding.

Table 4.4.1 shows the main survey methods for understanding the conditions of the underwater sections and water depths. In the initial and emergency restoration stages, because of the urgent need to quickly measure the water depths and understand the debris accumulated on the seafloor in large areas, including navigation channels, basins and the water areas in front of mooring facilities, it is effective to implement swath sounding (refer to **Reference [Part II], Chapter 1, 2.6 Bathymetric Survey** and **Reference [Part II], Chapter 1, 2.7 Swath Sounding**) using survey boats (with capacities of about 70 PS). In the full-scale restoration stage, because of the need to determine and acquire detailed information on the deformation of the facilities, it is effective to implement acoustic sounding using acoustic cameras and underwater three-dimensional cameras in addition to the above sounding.

The following section describes several types of acoustic equipment used to measure underwater features, focusing on their use in areas affected by a large earthquake and tsunami: **4.4.2 Swath Sounders**, **4.4.3 Side Scan Sonars** and **4.4.4 Acoustic Video Cameras and Underwater Three-Dimensional Scanners**. In the full-scale restoration stage, it is also necessary to detect the deformation of the vertical plane of the structures above sea level, in addition to the deformation of the underwater sections of breakwaters and mooring facilities. In such cases, to detect the deformation of the vertical plane of the structures above the sea level can be surveyed by mounting laser scanners, widely used in onshore topographic surveys, on the survey boats. To that end, **4.4.5 Laser Scanners Mounted on Survey Boats** is described at the end of this section.

Table 4.4.1 Methods for Understanding the Geometry of Underwater Areas

	Initial and emergency restoration stages	Full-scale restoration stage
Navigation channels and basins (waterways and basins)	(Confirmation of safety) ① Water areas where survey boats with a capacity of about 70 PS are navigable ● Narrow multibeam swath sounding machines ● Interferometry swath sounding machines ● Side scan sonars, etc. ② Water areas where boats with outboard engines are navigable ● Single beam echo sounders ● Lead sounding ● Underwater three-dimensional scanners, etc.	(Sounding) ① Understanding of water depths for disaster assessments Restoration design (dredging design, etc.) ● Narrow multibeam swath sounding machines ● Single beam sounders
Structures such as breakwater and mooring facilities and water areas in front of these structures	(Confirmation of safety) ① Water areas where survey boats with a capacity of about 70 PS are navigable ● Narrow multibeam swath sounding machines ● Side scan sonars, etc. ② Water areas where survey boats are not navigable due to floating debris ● Visual confirmation by divers ● Underwater cameras, underwater video cameras, etc.	(Survey for restoration design) ① Detailed measurements of facility deformation [Underwater section] ● Narrow multibeam swath sounding machines ● Side scan sonars ●Acoustic video cameras ● Underwater three-dimensional scanners ● Underwater cameras, underwater video cameras, etc. [Vertical plane of the structures above sea surfaces] ● Laser scanners mounted on board survey boats

4.4.2 Swath Sounders

(1) Outline

Swath sounding is a method for batch measurements of water depths at multiple measuring points during the navigation of survey boats in a manner that emits acoustic beams with sharp directivity toward the seafloor in a port-starboard direction. In swath sounding, the measured raw data is subjected to several types of processing (noise reduction, oscillation correction such as pitch and roll of the survey boats, tidal correction etc.) and is finally converted into planar bathymetric data (seafloor topographic data). Depending on the sounding principles, swath sounding is classified into a multibeam system (alternatively called a "narrow multibeam system") and an interferometry system.

The interferometry system has a wide swath angle, allowing it to collect depth soundings in large areas on a single operation. However, since the interferometry system requires a lot of effort during the noise reduction processing, it takes a relatively long time to produce deliverables. In addition, it shall be noted that the interferometry system is likely to have larger measurement errors in shallow water areas, and the areas immediately beneath the transducers have a distribution of measuring points that is less dense than the other areas, which might create the possibility of overlooking anomalies on the seafloor. Thus, careful consideration shall be given to the survey line intervals. When measuring the anomalies (deformations) of structures using swath sounders, there may be cases where the collected data of acoustic reflections do not correspond to the actual structures, caused by defused reflections of the acoustic waves. In contrast, the narrow multibeam systems can be used for both the sounding and detection of obstacles with highly reliable sounding data.

For the details of swath sounding, refer to **Reference [Part II], Chapter 1, 2.6 Bathymetric Survey** and **Reference [Part II], Chapter 1, 2.7 Swath Sounding**.

(2) Obtainable Information and Points to Consider

① **Initial and emergency restoration stages**

(Obtainable information)

Swath sounding can be used to confirm the water depths of navigation channels and basins and the water areas in front of mooring facilities after an earthquake, and the distribution states of debris floating into the sea, which has accumulated on the seafloor as a result of the disaster, can be obtained. Thus, swath sounding can be used for acquiring information (to confirm safe navigation areas, positions of the obstacles and the areas that need dredging etc.) necessary for the elimination of port obstacles.

(Points to consider)

i) Simplification of patch tests

For swath sounding implemented for the elimination of port obstacles, by taking into consideration the emergency situation, a simplified patch test can be used in place of the test specified in **Reference [Part II] , Chapter 1, 2.7 Swath Sounding** (an offset correction of each sensor in the sounding system, a correction of the deflection of the installation angles of the transducers [bias] and a correction of the recording delays for each device). An ordinary patch test is implemented daily with respect to the parallel and round-trip survey lines, and if necessary, cross survey lines, if necessary, in addition to the actual measurement. In the simplified version, a patch test can be implemented with respect to the actual survey line data.

ii) Conversion to water depths using temporary chart datum level

In the emergency restoration stage, the chart datum level affected by crustal movements cannot be used as they are. Thus, in the emergency restoration stage, the swath sounding data can be converted to water depths by using the temporary chart datum level set in accordance with **Reference [Part II], Chapter 2, 3.2 Emergency Setting of Chart Datum Level**. In the implementation of the elimination of port obstacles, abnormal objects on the seafloor need to be treated as abnormalies points instead of noise.

iii) Selection of equiangular mode

The narrow multibeam system requires a large number of survey lines in shallow water areas so as not to leave any unmeasured water areas. Considering the improved availability of the types of swath sounding machines capable of changing beam forming systems in recent years, it is preferable to implement highly accurate measurements by adopting an equiangular mode when exploring abnormal objects on the seafloor of shallow water areas.

② **Full-scale restoration stage**

(Obtainable information)

Swath sounding can clarify the clarify the states of displaced or scattered armor and wave-dissipating blocks for breakwater mounds, and the scope as well as quantities of the restoration work for displaced or scattered blocks, thereby contributing to the restoration design of the facilities.

Swath sounding is effective in obtaining information for formulating the design and construction plans pertaining to the overall restoration work, such as the states of scouring and debris accumulation in front of the revetments and quaywalls, as well as the deformation of the revetments and quaywalls, necessary for formulating the design and construction plans pertaining to the overall restoration work.

4.4.3 Side Scan Sonars

(1) Outline

Side scan sonars are capable of obtaining planar acoustic image records of a certain width in a manner that laterally emits ultrasonic waves in a fan-like formation toward the seafloor from a device (side scan sonar) towed by a survey boat, and receives the ultrasonic waves reflected off the seafloor or underwater structures (refer to **Fig. 4.4.1**). Sounding conducted by a side scan sonar can classify the seafloor materials (mud, sand and gravel) relatively, and detect the states of the displaced or scattered blocks through planar imaging, as it generates images of the reflected wave intensity (refer to **Fig. 4.4.2**). Side scan sonars are highly convenient, because they are compact and user friendly, they can be used in narrow and shallow water areas, and they come in a wider variety of popular models than narrow multibeam echo sounders.

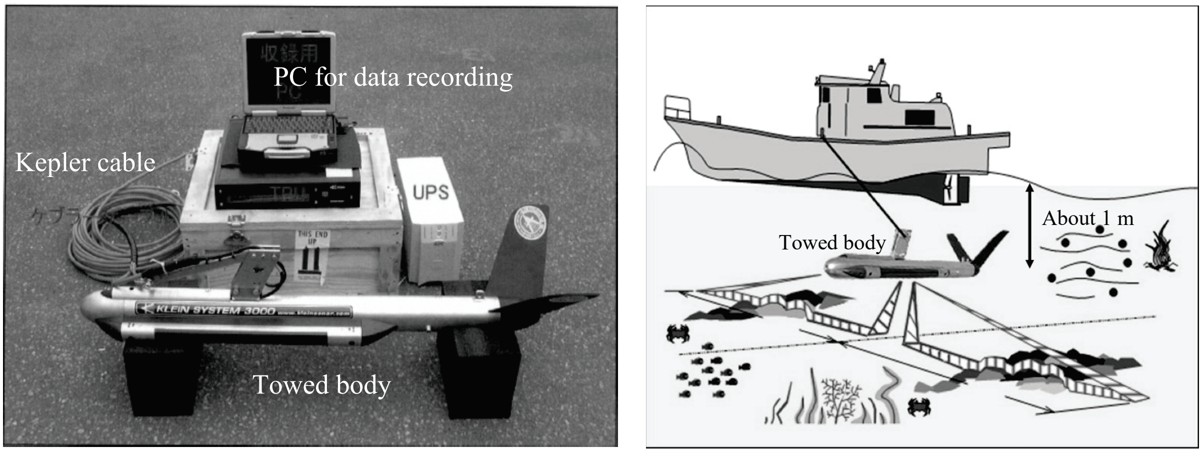


Fig. 4.4.1 Side-Scan Sonar and Schematic Drawing of a Side-Scan Sonar Towed for Sounding

Fig. 4.4.2 Image Obtained Using a Side-Scan Sonar
(State of Displaced and Scattered Foot Protection Blocks of a Damaged Breakwater)

(2) Obtainable Information and Points to Consider

① Initial and emergency restoration stages

(Obtainable information)

Side scan sonars can detect the distribution states (planar positions) of debris (including containers and vehicles) accumulated on the seafloor in navigation channels, basins and water areas in front of mooring facilities after the disaster, contributing in determining safe water areas for navigation and detecting the presence of unsafe obstacles for navigation.

(Differences between swath sounding machines and side scan sonars)

In the initial and emergency restoration stages, the scattering intensity data obtained by swath sounders (refer to **Reference [Part II], Chapter 1, 2.7 Swath Sounding** and **Reference [Part II], Chapter 2, 4.4.2 Swath Sounders**) may be used for detecting submerged debris, but swath soundings require too much time for a data analysis to produce quick estimations at the sites. In contrast, side scan sonars can detect debris of certain sizes on the monitor screens onboard survey boats during sounding.

Furthermore, there may be cases where inundation due to tsunamis temporarily generates suspended mud above the seafloor of shallow waters. In such cases, acoustic waves with frequencies of 200 to 400 kHz normally used in swath sounders cannot reach objects below the upper surfaces of the suspended mud. In contrast, acoustic waves with frequencies of about 100 kHz used by side scan sonars can penetrate suspended mud, and therefore have a high possibility of detecting debris in the suspended mud (it is also difficult for divers to visually identify debris in suspended mud).

(Points to consider)

During soundings by side scan sonars, the debris scattered on the seafloor can be identified as objects of particularly strong reflections on a monitor screen on board the survey boat. The positions of the objects of particularly strong reflections are then recorded in a computer and analyzed on shore later for final confirmation of whether or not these objects represent debris or not. **Fig. 4.4.3** shows a typical example of an image on a monitor screen of a side scan sonar that has been determined to be debris.

In the case of the elimination of port obstacles requiring an urgent determination of debris, the data of the estimated types and positions of the objects can be used as the information for the port obstacle removal work. In such cases, it is preferable to obtain the information on the approximate water depths of these objects.

Generally, after measuring with a side scan sonar, the records of the survey lines are compiled on a single mosaic seafloor topographic map to enable the overall distribution of the debris to be understood. It is also preferable to organize the data on the positions (latitudes and longitudes and national coordinates), lengths and widths of the objects determined to be debris in entirely explored water areas to enable the data to be effectively used in the obstacle removal work.

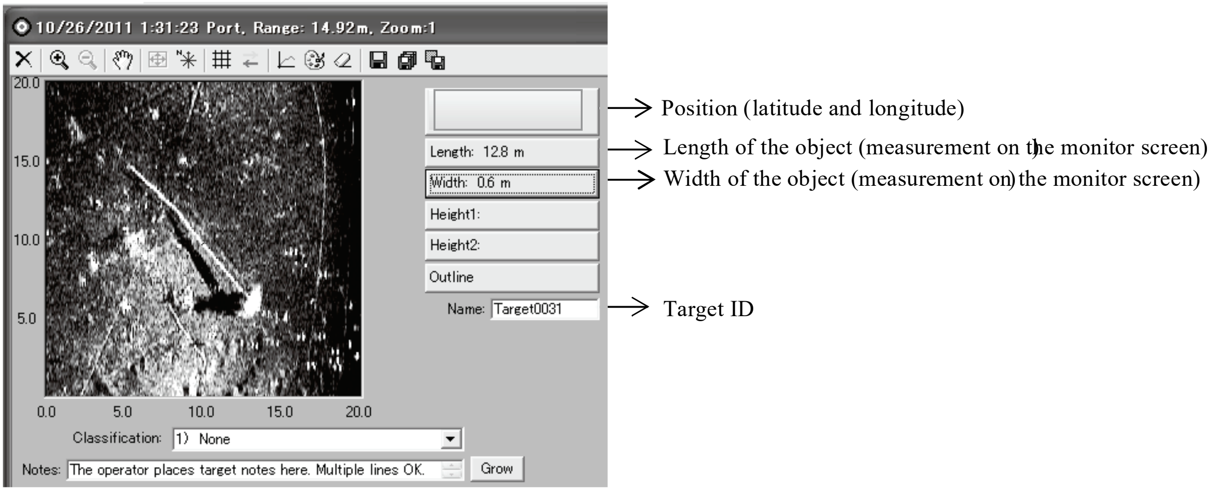


Fig. 4.4.3 Example of a Recording from a Side-Scan Sonar (Japanese Cedar Driftwood)

② **Full-scale restoration stage**

(Obtainable information)

In the full-scale restoration stage, side scan sonars can be used to obtain information on the states of the displaced and scattered armor and wave-dissipating blocks of sloping breakwaters in shallow water. This information can be used for determining the scopes and quantities of the block restoration work and in the restoration design of the facilities.

4.4.4 Acoustic Video Cameras and Underwater Three-Dimensional Scanners

(1) General

The information required in the full-scale restoration stage are the detailed deformation states of the underwater sections of facilities in relatively wide areas, which can be effectively obtained through acoustic imaging sonars. These sonars visualize underwater spaces or measure coordinates using acoustic waves. Although acoustic imaging sonars can be used in turbid waters with low visibility, or in nighttime measurements with low illuminance using the physical properties of the sound waves, they may have fuzzy images or incorrect coordinate data due to unnecessary responses generated by multipath or residual images in narrow indented water areas.

Acoustic imaging sonars can be selected from a variety of models for different types of transmission and reception as well as image provision systems. Thus, it is important to select models suitable for the use purposes. Among the various types, a device capable of displaying real-time acoustic images (acoustic video cameras) and a device capable of conducting surveys using acoustic data (underwater three-dimensional scanners) are described below.

(2) Acoustic Video Cameras

① **Outline**

Acoustic video cameras (refer to **Fig. 4.4.4**) are acoustic imaging sonars capable of sequential updates of high-resolution acoustic images of two-dimensional underwater spaces. Although acoustic video cameras are not suitable for surveying comparatively large water areas because of their narrow angle of view, they can be used for detailed (close range) investigations of comparatively narrow water areas, and, therefore, are suitable for detailed visual surveys of deformed sections identified through wide range explorations. Outputting measured data is not available with general models because the measured data are two-dimensional video images. In contrast, the acoustic video cameras developed by the Port and Airport Research Institute (refer to Item ③ below) are capable of outputting such data.

In addition to the method for visualizing underwater spaces with acoustic video cameras attached to survey boats, they can be carried by divers or mounted on ROVs or AUVs, taking advantage of their reduced size and weight and energy-saving performance.

Fig. 4.4.5 shows the comparison of an image of an underwater section of a steel sheet pile taken by an acoustic video camera on the left side, with an image of an above-water section of a steel sheet pile taken by an optical camera on the right side.

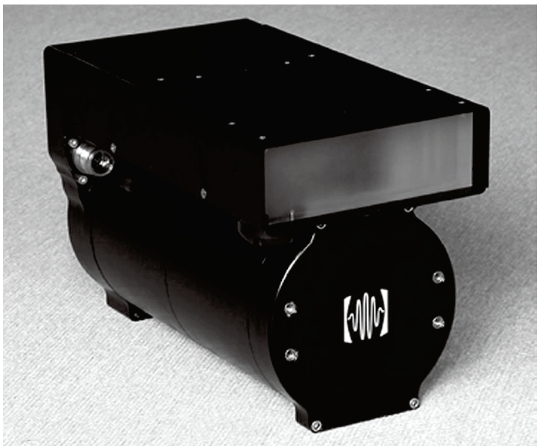


Fig. 4.4.4 Acoustic Video Camera[16)]

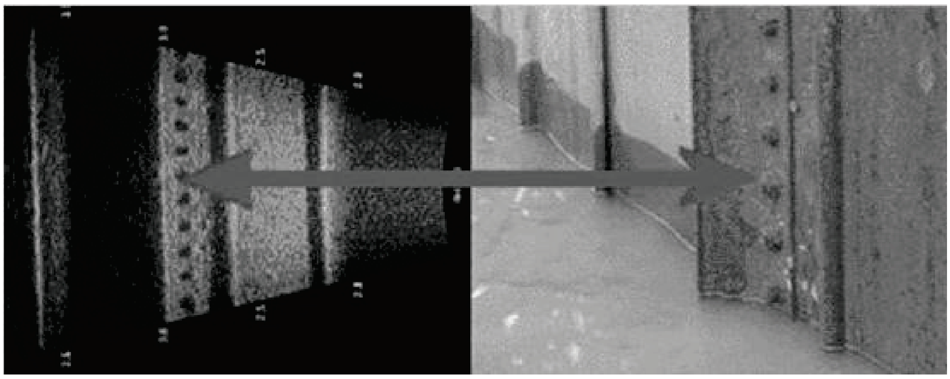


Fig. 4.4.5 Underwater Section of a Steel Sheet Pile Taken by an Acoustic Video Camera (Left) and Above-Water Section of a Steel Sheet Pile taken by an Optical Camera (Right) (provided by the manufacturer)

② **Obtainable information and points to consider**

(Obtainable information)

In the full-scale restoration stage, acoustic video cameras can be used effectively for detailed investigations of deformed sections identified through wide area explorations. For example, acoustic video cameras are suitable for detailed (close range) visual investigations of comparatively narrow areas, such as underwater sections with suspected damage on the mooring facility or revetment.

In addition, by taking advantage of their high update rates, acoustic video cameras can be used for visualizing the movements of objects in real time (e.g., the monitoring of oil leaks from underwater pipelines, or the safety monitoring of underwater work by divers).

(Points to consider)

Acoustic video cameras are:

- not suitable for wide-area explorations because of their narrow angle of view;
- not suitable for coordinated measurements because of the two-dimensional display of the azimuth direction and range;
- not physically capable of observations of small cracks and corrosion because of the limitations in the acoustic wavelengths expressed by wavelength (m) = acoustic velocity 1500 (m/s)/frequency (Hz);
- slower in the visualization of close-range images than information collection through visual observations by divers when underwater visibility is high.

③ **Other**

Unlike the acoustic video camera shown in **Fig. 4.4.4**, the camera developed by the Port and Airport Research Institute is capable of sequentially updating high-resolution acoustic images of wide three-dimensional underwater space in real time and providing acoustic images close to the visual perception of human beings. Thus, the acoustic video camera developed by the Port and Airport Research Institute is expected to improve the performance of detailed visual confirmations of damage situations. Furthermore, because of its capability of measuring three-dimensional coordinates while conducting real-time monitoring, it can be applied to the direct visual supervision of underwater work.

(3) Underwater Three-Dimensional Scanners

① **Outline**

Underwater three-dimensional scanners are acoustic imaging sonars capable of acoustic scanning linear (one-dimensional) spaces widely in a vertical direction and extremely narrow in a horizontal direction. By mechanically rotating the sonar head (**Fig. 4.4.6**) installed on a tripod or ROV, underwater three-dimensional scanners enable three-dimensional coordinate measurements to be easily implemented.

For coordinate measurements where a sonar head fixed on a tripod is rotated, since a single probe requires 1 second, it takes about 6 minutes to acquire coordinate data by rotating the sonar head 360 degrees horizontally. The coordinate data is 360-degree, three-dimensional point group data with the sonar head as the center and converted into three-dimensional data of the structures after being subjected to post-processing, such as tide level and acoustic velocity correction.

Fig. 4.4.7 shows an example of a three-dimensional coordinate measurement of underwater structures and seafloor topography implemented as a verification test[17)] of the seafloor state observation technology for the elimination of port obstacles.

In addition to the method for visualizing underwater spaces with sonar heads attached alongside survey boats, there are cases of using underwater three-dimensional scanners in a manner that attaches sonar heads to fixtures extended from quaywalls for simple measurements and mounts sonar heads on ROVs, taking advantage of their reduced size and weight and energy-saving performance.

Fig. 4.4.6 Underwater Three-Dimensional Scanner[17)]

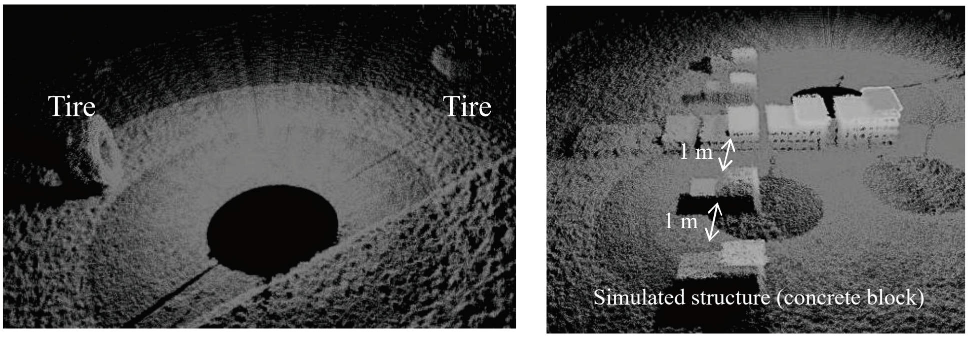


Fig. 4.4.7 Result of a Verification Test of Seafloor State Observation Technology for the Elimination of Port Obstacles[17)]
(Three-dimensional coordinate measurement with an underwater three-dimensional scanner installed on the seafloor)

② **Obtainable information and points to consider**

(Obtainable information)

In the full-scale restoration stage, underwater three-dimensional scanners are used for acquiring three-dimensional coordinate data by mechanically rotating a sonar head attached to a tripod, and converting the three-dimensional coordinate data into three-dimensional point group data through post processing with the sonar head as the center.

Thus, underwater three-dimensional scanners enable to determine the damage states of underwater structures, the amounts of scouring on the seafloor around the structures, and the quantities of debris removal work.

(Points to consider)

- Underwater three-dimensional scanners are generally used with sonar heads installed on tripods set on the seafloor by divers.

- In the case of environments that do not allow divers to access the seafloor, underwater three-dimensional scanners can still be used in a manner that suspends sonar heads in the seawater from fixtures set above the sea levels.
- The point group data obtained through the underwater three-dimensional scanners can be analyzed concurrently with other point group data obtained through onshore laser scanners or narrow multibeam swath sounders.
- As underwater three-dimensional scanner is small and light, it can be used as a mobile three-dimensional measurement device by fitting out on small, light-weight boats with outboard engines with an oscillation correction device. It is effective to use underwater three-dimensional scanner even in water areas where survey boats with a capacity of 70 PS cannot approach due to the floating objects and debris.

4.4.5 Laser Scanners Mounted on Board Survey Boats

(1) Outline

Laser scanners mounted on survey boats are scanners widely used for onshore topographic surveys, with oscillation and heading correction devices fitted out on the survey boats. The laser scanners mounted onboard can measure the three-dimensional point group data on the vertical plane of the port structures above sea levels with a high accuracy and density to be acquired while navigating the survey boats alongside the structures (**Fig. 4.4.8**). In particular, laser scanners mounted onboard are effective to measure the vertical plane of treacherous port structures, such as wave-dissipating blocks positioned in front of breakwaters and revetments, safely with a high density and accuracy.

(2) Seamless Three-Dimensional Data from Underwater Sections to above Sea Levels

Seamless three-dimensional point group data on the vertical plane of the port structures from the underwater sections to above sea levels can be obtained by combining the three-dimensional point group data on the structures above sea levels obtained using laser scanners mounted on survey boats with the three-dimensional point group data shown below.

(Example of point group data)

- Three-dimensional point group data on the port structures above sea levels measured with a UAV (e.g., breakwater superstructures and wave-dissipating blocks)
- Three-dimensional point group data on the port structures below sea levels measured with narrow multibeam swath sounders or underwater three-dimensional scanners.

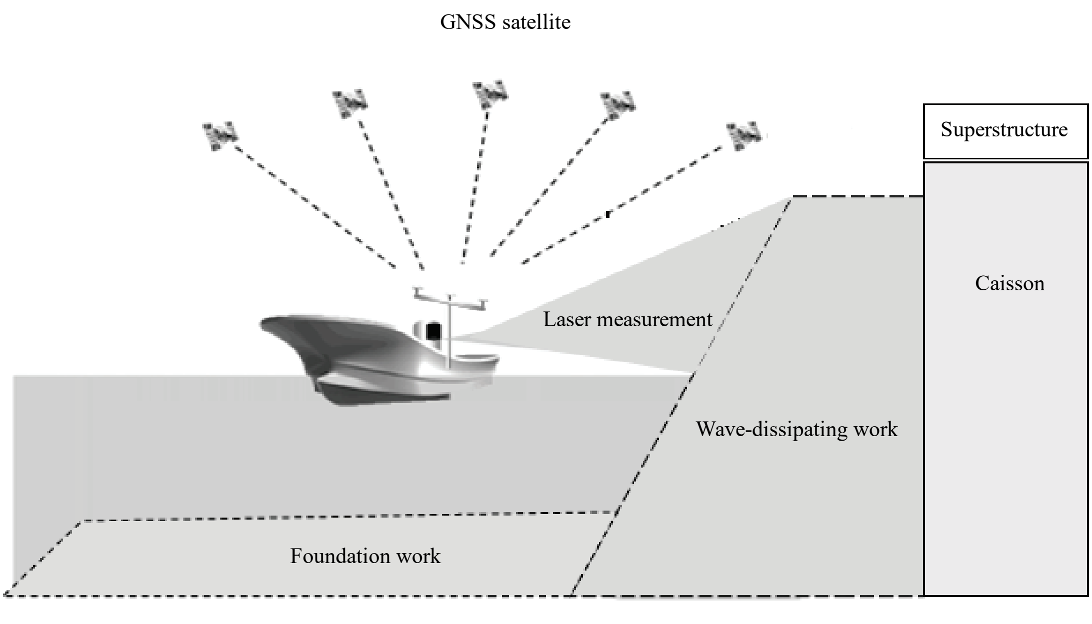


Fig. 4.4.8 Schematic Drawing of a Survey with a Laser Scanner Mounted on Board a Survey Boat

4.5 Understanding Broad-Based Ground Deformation

4.5.1 General

In the case of extensive ground deformation due to a large-scale earthquake, it is necessary to understand the trends of ground deformation in broad areas, including entire ports, determine the presence or absence of faults, and implement long-term observation of the trends. The survey methods for understanding broad-based ground deformation are twofold, as shown in **Table 4.5.1**. The following section describes these two methods (**4.5.2 Broad-Based Ground Deformation Analysis Using Satellite Images** and **4.5.3 Broad-Based Ground Deformation Analysis Using Aerial Laser Data**). A variety of methods have been developed for broad-based ground deformation analysis using satellite images. The description in the following section focuses on the PSInSAR method, which is suitable for coastal areas with a heavy concentration of artificial structures, such as port areas.

Table 4.5.1 Method for Understanding Broad-Based Ground Deformation

	Emergency restoration stage	Full-scale restoration stage
Main survey method	• Broad-based ground deformation analysis using satellite images • Broad-based ground deformation analysis using aerial laser data	Same as described in the column to the left

4.5.2 Broad-Based Ground Deformation Analysis Using Satellite Images (PSInSAR)

(1) Outline

PSInSAR is an abbreviation for “Persistent Scatterer Interferometric Synthetic Aperture Radar,” which is a technology applicable to analyzing ground deformation in millimeters using SAR images taken by satellites.

PSInSAR is capable of measuring ground deformation in millimeters using the phase differences of reflected microwaves emitted toward the ground from satellites. The broad-based ground deformation analysis using satellite images is conducted with respect to the reflected microwaves from specific reflectors, such as artificial structures, by selecting only the points (persistent scatterers) that can ensure a certain level of coherence (the phase function accuracy of the sine waves). The deformation amounts of the persistent scatterers are calculated in a manner that superimposes multiple images taken at different times and reduces noise by correcting orbit errors, distortions due to unevenness in the ground surfaces, and the influences of the atmosphere included in the reflected waves.

PSInSAR can analyze the deformation of a wide area at one time. For example, in urban areas, PSInSAR can perform a broad-based ground deformation analysis at around 200 points per km^2 (1 point in a 50- to 100-meter square) to 600,000 points per km^2 (60 points in a 10-meter square), depending on the observation modes. PSInSAR conducts measurements using reflections from the roofs of existing buildings without requiring measuring equipment or special devices to be brought in and set up at the measuring points. The satellite image data available for broad-based ground deformation analysis is limited to that obtained after 1992. In addition, PALSAR-2, which is a domestic SAR on board the Japanese satellite “Daichi No. 2,” has been observing the whole of Japan since 2014.

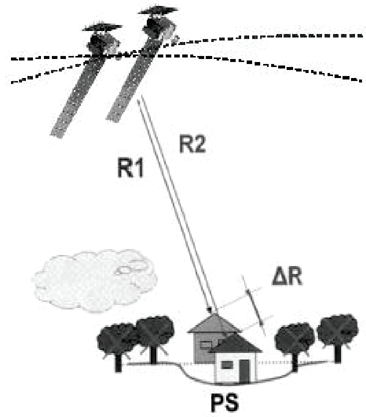


Fig. 4.5.1 Schematic Drawing of the Measurement Principle of PSInSAR

(2) Obtainable Information and Points to Consider

(Obtainable information)

The broad-based ground deformation analysis using satellite images enables the amounts of ground deformation and the chronological changes in the deformation after a disaster to be monitored using the SAR(Synthetic Aperture Radar) images before and after the disaster.[18)] In the case of the Great East Japan Earthquake, extensive ground deformation due to the earthquake included the ground subsidence along coastal areas, which increased the risk of secondary disasters due to inundation and storm surges. The broad-based deformation analysis using satellite images is effective in understanding the damage situations of extensive areas, as is the case with the extraction of the areas affected by ground deformation or liquefaction[19)] as a result of a large-scale earthquake (**Fig. 4.5.2**).

When surveying the areas and magnitudes of onshore ground deformation in the emergency restoration stage, the ground deformation can be analyzed in centimeters, both in horizontal and vertical directions, by combining the measurements through a simplified interferometer SAR with data correction with reference to the GNSS values.[20)]

In the full-scale restoration stage, the amounts of ground deformation can be analyzed in millimeters by obtaining multiple satellite images after the disaster over time, thereby enabling the restoration plans to be formulated in consideration of the distribution of the damage quantities caused by extensive ground deformation with temporal changes.

(Points to consider)

The Great East Japan Earthquake caused extensive ground uplift and subsidence, with the amounts of ground subsidence exceeding 1 m in some locations. After the earthquake, the uplifted or subsided ground deformed back to the original elevations (after the effect deformation); this deformation still continues with a gradual attenuation of resilience. After a large-scale earthquake, it is important to understand the presence or absence of broad-based ground deformation and local changes in the ground elevations due to local damage, such as liquefaction, and reflect on the influences of these different types of ground deformation when deliberating on countermeasures. Furthermore, when analyzing the trend of ground deformation, it is necessary to fully consider the situation in the survey area because ground deformation such as land subsidence may occur due to nearby construction.

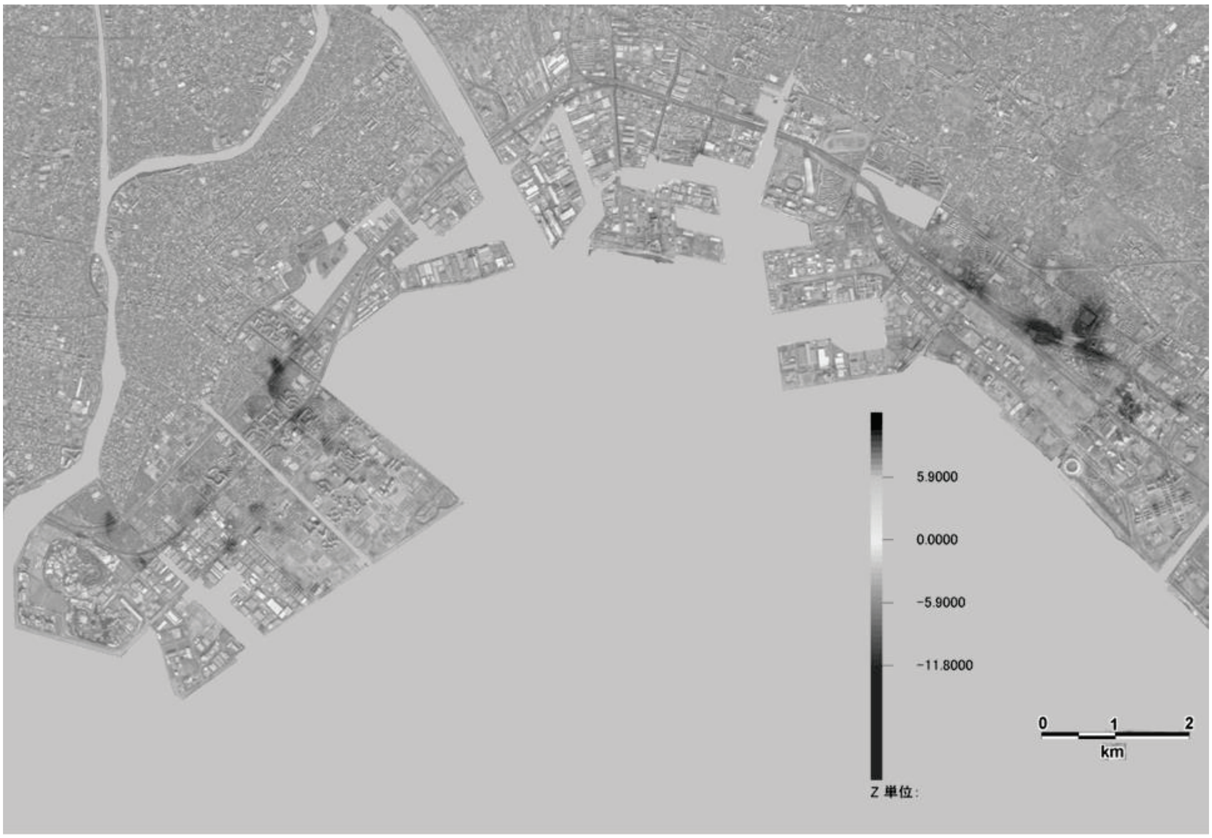


Fig. 4.5.2 Example of an Analysis of the Subsidence Amount Using PSInSAR in the Tokyo Bay Area, Which Underwent Liquefaction Due to the Great East Japan Earthquake (Modification of Reference 19))

4.5.3 Broad-Based Ground Deformation Analysis Using Aerial Laser Data

(1) Outline

There are cases where broad-based ground deformation can be analyzed by comparing the data obtained through aerial laser surveys (refer to **Reference [Part II], Chapter 2, 4.2.5 Aerial Laser Surveys**) before and after an earthquake. For the execution of accurate analyses, it is preferable to compare the data obtained on dates close to each other before and after the earthquake.

(2) Obtainable information and points to consider

(Obtainable information)

Faults can be identified as level differences in the images showing the differences in the DSMs (Digital Surface Models) before and after an earthquake. In addition, horizontal displacement before and after an earthquake can be identified from the differences in DSMs, and additional image processing or point group processing can enable the horizontal displacement amounts to be automatically estimated.

Fig. 4.5.3 shows a default location map (upper diagram) and a displacement vector analysis diagram (lower diagram) created from the differences in the aerial laser data obtained before and after the Kumamoto Earthquake (on April 16, 2016).[21)] The displacement vector analysis was executed through a method called ICP (Interactive Closest Point) used for aligning three-dimensional point groups.

(Points to Consider)

The same points to consider as described in **Reference [Part II], Chapter 2, 4.5.2 Broad-Based Ground Deformation Analysis Using Satellite Images (PSInSAR)** apply to the use methods of the broad-based ground deformation analysis using aerial laser data.

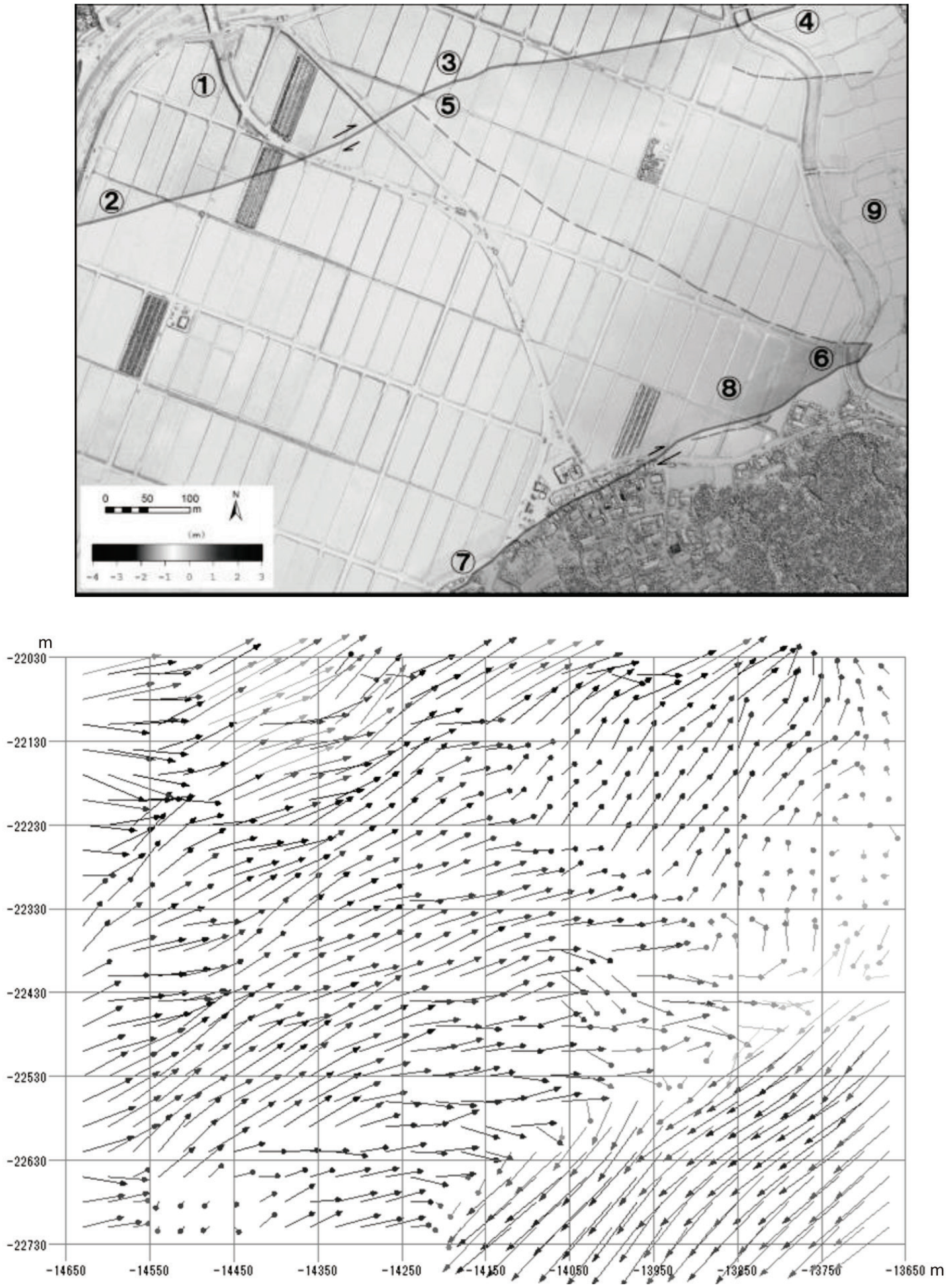


Fig. 4.5.3 Extraction of a Fault through the Differences in Aerial Laser Data[21)]

4.6 Surveys on Ground Liquefaction

4.6.1 General

The occurrence of ground liquefaction during an earthquake intensifies the damage to port structures such as mooring facilities and revetments. Liquefaction can cause an increased seaward displacement of mooring facilities and cavities beneath aprons, and it is important to understand whether or not liquefaction has occurred as basic information to determine the safety of using the mooring facilities immediately after an earthquake. It is also effective to acquire information on whether or not liquefaction has occurred for the restoration design and investigation of the reasons for the damage in the full-scale restoration stage. **Table 4.6.1** shows the methods for understanding whether or not liquefaction has occurred and survey methods which can be used for predicting and determining the possibility of liquefaction in the design stage.

In the emergency restoration stage, whether or not liquefaction has occurred at the quaywalls and aprons can be confirmed through visually identifying, sketching or taking photographs of any evidence of sand boiling. The evidence of sand boiling will remain on the ground surfaces when no tsunamis run up to the sites after an earthquake, but it can be easily erased by rainfall or storm surges. Thus, in cases where the determination of whether or not liquefaction has occurred is considered to largely affect the determination of the reasons for the damage, it is necessary to identify early the occurrence of liquefaction after the earthquake.

In the full-scale restoration stage, for the purpose of determining restoration design and identifying the reasons for the damage, there may be cases of predicting and determining the occurrence of liquefaction and the presence or absence of cavities in the surrounding ground when subjected to design earthquake motions or reproduced earthquake motions as needed. In addition to normal boring surveys, the methods listed in **Table 4.6.1** can be used for the above purpose. Among these methods, the outlines of **two-dimensional surface wave exploration** and the **Piezo Drive Cone (PDC)** are described in **4.6.2** and **4.6.3**, respectively.

Table 4.6.1 Liquefaction Determination Survey Methods

	Emergency restoration stage	Full-scale restoration stage
Main survey method	(Confirmation of whether or not liquefaction has occurred) Visual confirmation, sketching, photographs and video recordings	(Prediction and determination of liquefaction) Boring, two-dimensional surface wave exploration and the Piezo Drive Cone (PDC)

4.6.2 Two-Dimensional Surface Wave Exploration

(1) Outline

Two-dimensional surface wave exploration is a survey method for obtaining the S-wave velocity distribution in the ground by measuring the surface waves (Rayleigh waves) in surface layers with thicknesses of about 20 m (**Fig. 4.6.1**).[22] In two-dimensional surface wave exploration, a cable with geophysical exploration seismometers attached alongside it with intervals of 1 m is laid on a survey line and ground motions are artificially applied to the surface layer by hitting it with a wooden maul (**Fig. 4.6.2**). Then, the measurements of the surface waves propagating through the shallow ground with multichannel seismometers (12 to 48 channels) are processed into an S wave velocity structure profile of the ground along the survey line.

Because S wave velocities correlate with the stiffness of the ground, they can be used for understanding the ground's geophysical properties. When combined with an analysis of the *N* values obtained through the standard penetration test, the surface wave exploration enables a two-dimensional *N*-value profile to be estimated. The two-dimensional *N*-value profile can be used for predicting and determining the liquefaction of the ground and identifying large cavities.

Fig. 4.6.3 shows a contour figure of the S wave velocity distribution in the ground of S Airport. The two-dimensional surface wave exploration was implemented in the course of the supervision of the liquefaction countermeasure work for the runways and taxiways. The runways and taxiways for which the liquefaction countermeasure work was implemented did not undergo liquefaction during the Great East Japan Earthquake.

Two-dimensional surface wave exploration has also been used to understand the stratum structure of very soft ground such as tidal flats.

As is the case with the study on the sedimentary structure of the soil under the tidal flat in the Buzen Sea shown in **Fig. 4.6.4**, the two-dimensional surface wave exploration that enables the shear wave velocity distribution, which represents the hardness or softness of the ground, to be quantitatively understood is proven to be an effective means to efficiently evaluate the stratum structures of the ground of tidal flats.[23]

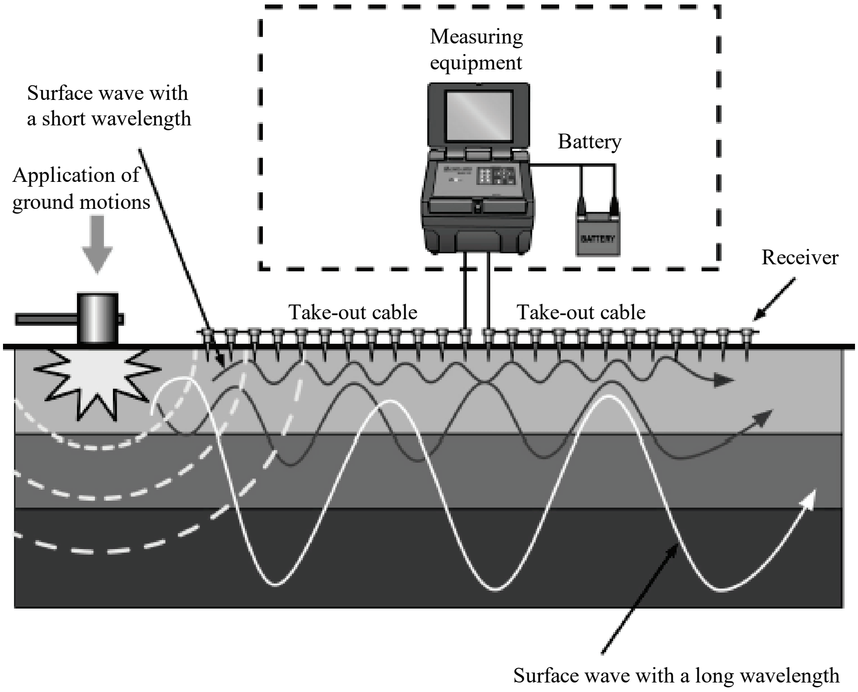


Fig. 4.6.1 Schematic Drawing of Two-Dimensional Surface Wave Exploration[22)]

Fig. 4.6.2 Geophysical Seismometer (Left) and Application of Ground Motions (Right)

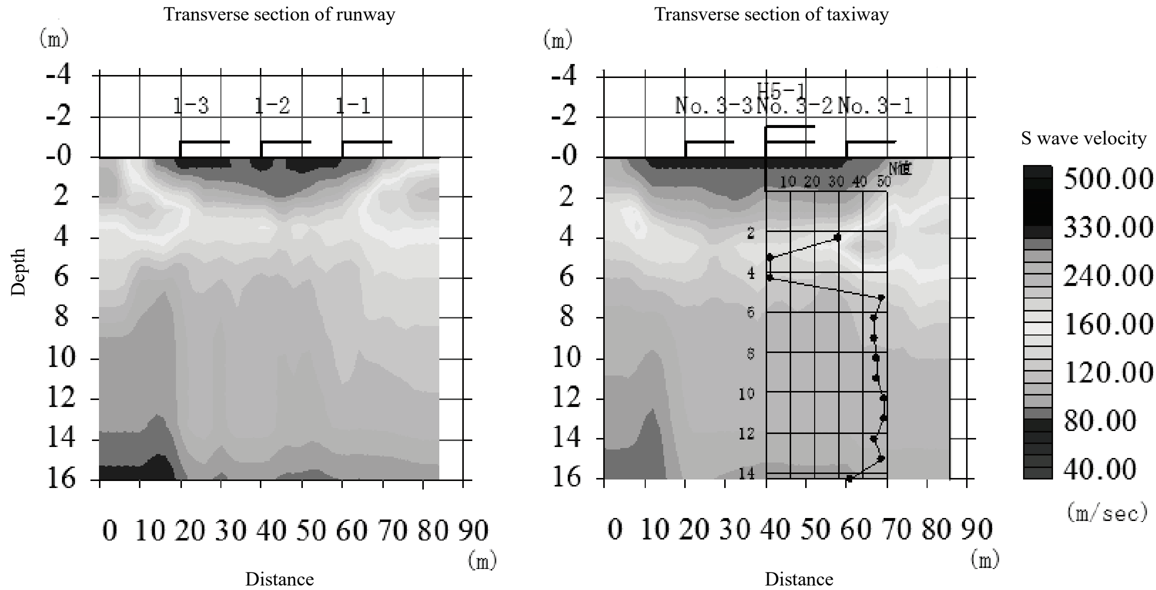


Fig. 4.6.3 Example of Two-Dimensional Surface Wave Exploration Results (After the Liquefaction Countermeasure Work at S Airport)

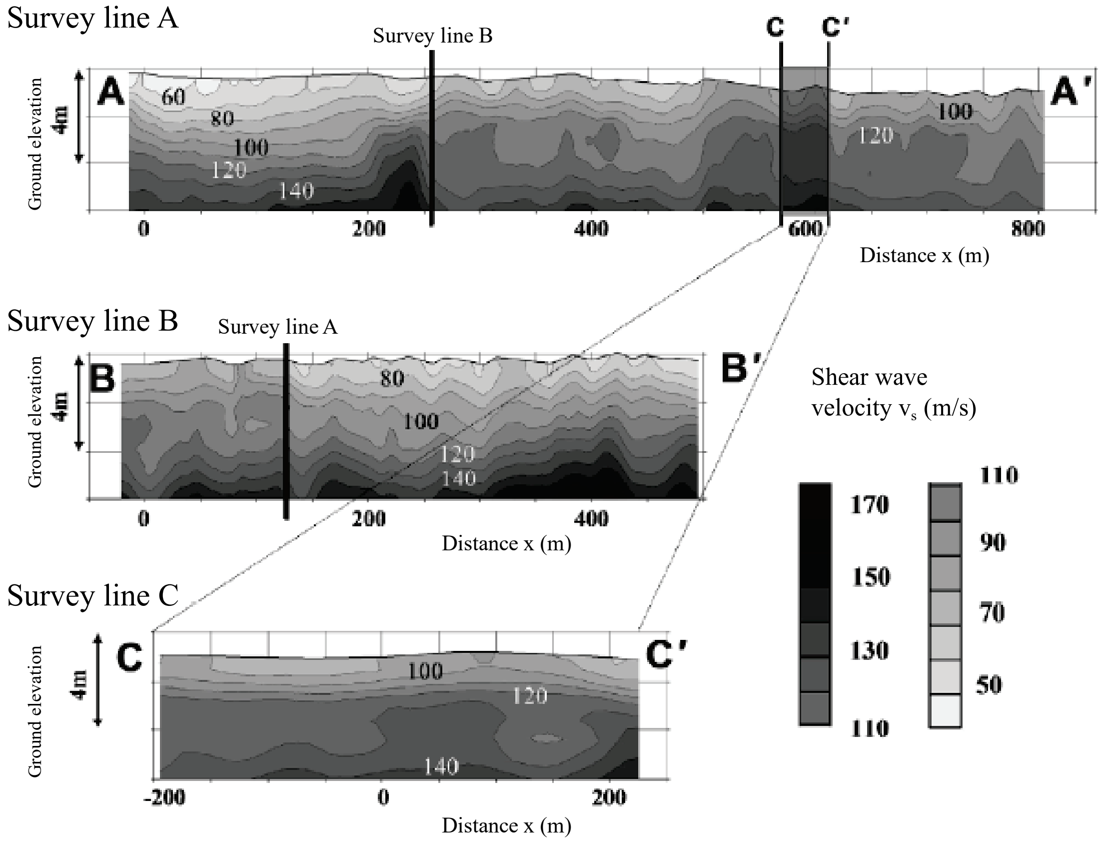


Fig. 4.6.4 Example of Two-Dimensional Surface Wave Exploration Results (Sedimentary Structure of the Soil under the Tidal Flat in the Buzen Sea)[23)]

(2) Obtainable Information

In the full-scale restoration stage, the combination of the two-dimensional surface wave exploration results with the boring or sounding survey results enables the two-dimensional stratum structure in a vertical direction to be understood and facilitates the selection and examination of the types and scopes of liquefaction countermeasure work.

(3) Points to consider

Because the surface wave exploration analyzes the surface waves that propagate through horizontally stratified ground, there may be cases where the exploration results are affected by structures when the survey line is set parallel to a structure, such as a quaywall or revetment. Thus, it is preferable to consult with a specialist about the appropriateness of setting the survey lines close to the structures when planning the surface wave exploration.

4.6.3 Piezo Drive Cone (PDC)

(1) Outline

The PDC is capable of surveying the in-situ liquefaction strength of the ground in a manner that measures the ground resistance against percussive penetration and pore water pressure generated in the ground at the tip of the cone when the cone is percussively penetrated into the ground, evaluates the *N* values representing the penetration resistance of the ground, and estimates the ground water level and the soil properties (fine contents [F_C]) by depth (**Fig. 4.6.5**).

Because the PDC enables measurements to be easily conducted in a short period of time, it can be used for planar and dense surveys, even when the object ground is extensive and inhomogeneous. In addition, the PDC can shorten the survey periods and reduce survey costs because the measurements of the PDC can be used directly without requiring additional laboratory soil tests, such as grain size tests, and the penetration of the PDC into the ground can be executed easily with a compact dynamic penetration machine.

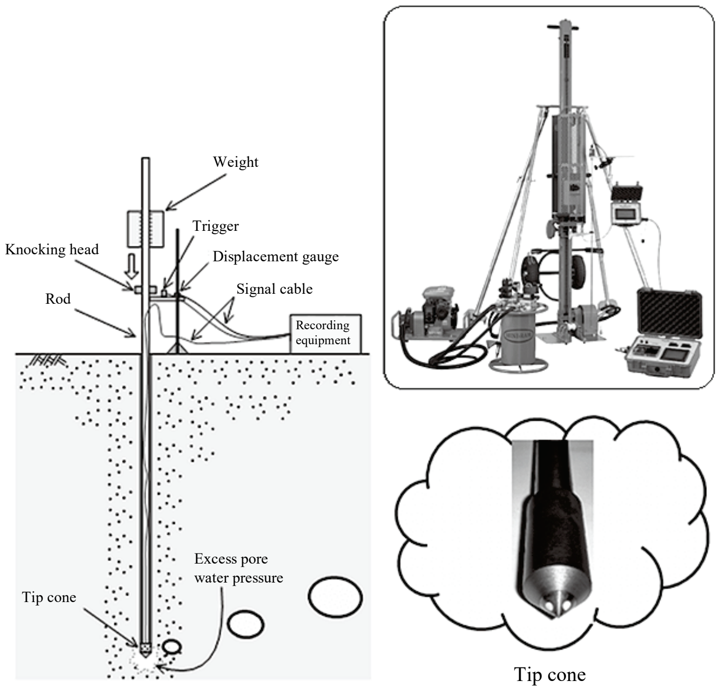


Fig. 4.6.5 Conceptual Drawing of the Piezo Drive Cone (Modification of Reference 24))

Fig. 4.6.6 shows an example of a liquefaction strength evaluation using measurements done with the PDC.[25)] The N_d values measured with the PDC are almost equivalent to the *N* values of the standard penetration test, and can express the changes in soil properties in the depth direction with a higher resolution than the *N* values in that the N_d values can be obtained continuously in the depth direction, while the *N* values are available every 1 m. Thus, compared to the conventional liquefaction strength R_L obtained from the *N* values of the standard penetration test and F_C (fine contents), the PDC enables the continuous distribution of the liquefaction strength in the depth direction to be evaluated. Furthermore, the liquefaction strength obtained through measurements using the PDC is proven to correspond well to the conventional liquefaction strength. It is also confirmed that the PDC enables the settlement amounts to be predicted with a high accuracy through comparisons between the predicted values of settlement due to the dissipation of excess pore water pressure after liquefaction and the actual measurements.

Currently, the exploratory committee for ground improvement work through the chemical grouting method in reclaimed areas (under the Ports and Harbours Bureau of the Ministry of Land, Infrastructure, Transportation and Tourism) has been studying the applicability of the PDC to the evaluation of ground improvement effects.[26)]

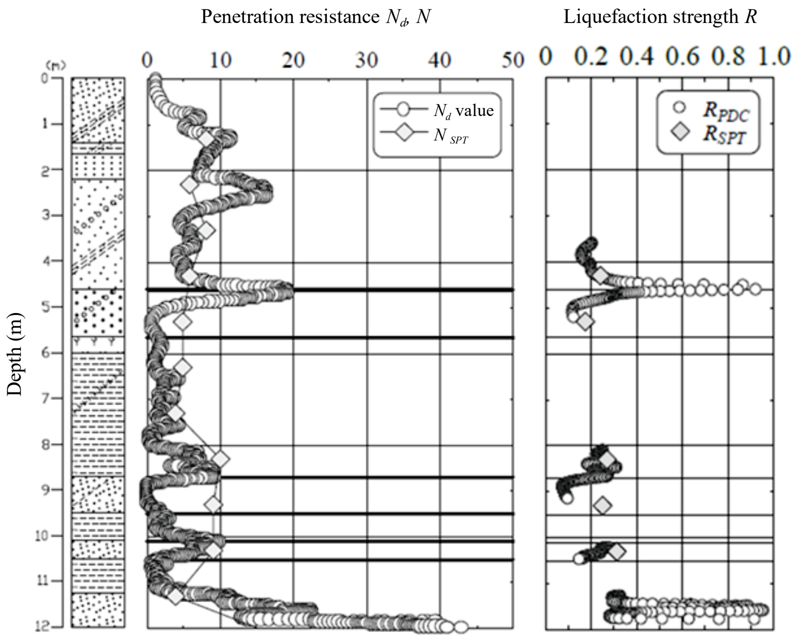


Fig. 4.6.6 Example of Measurement Results (Modification of Reference 25))

(2) Obtainable Information

The estimation results using the PDC can be combined with the boring survey results (as needed) for obtaining the three-dimensional stratum structure, which can be used for selecting and examining the types and scopes of liquefaction countermeasure work.

(3) Point of Caution

When using the PDC, an appropriate penetration machine shall be selected in accordance with the hardness or softness of the ground and depths to be surveyed.

4.7 Understanding the Cavities at Apron Sections

4.7.1 General

It is important to determine the presence or absence and the extent of cavities in the ground immediately below aprons (concrete pavement) and yards (asphalt and concrete pavements) as basic information to determine the usability of mooring facilities immediately after an earthquake. In addition, such information can be effectively used for the restoration design and the investigation of the reasons for the damage in the full-scale restoration stage. **Table 4.7.1** summarizes the methods which can be used for determining the presence or absence of cavities in aprons and yards.

In the emergency restoration stage, the presence or absence of cavities is estimated based on the states of subsidence and deformation of pavement surfaces identifiable through visual confirmation, sketches or photographs.

In the full-restoration stage, the areas of aprons and yards suspected to have cavities below them are estimated through nondestructive exploration, such as underground radar exploration, on-vehicle subsurface exploration and two-dimensional surface wave exploration (refer to **Reference [Part II], Chapter 2, 4.6.2 Two-Dimensional Surface Wave Exploration**) for determining the restoration design and investigating the reasons for the damage. There may also be cases where the pavement in areas suspected to have cavities needs to be drilled so as to enable the cavities to be visually identified with endoscopic cameras. This section describes **underground radar explorations** and **on-vehicle subsurface explorations** in **4.7.2** and **4.7.3**, respectively.

Table 4.7.1 Survey Methods for Exploring Cavities below Aprons and Yards

	Emergency restoration stage	Full-scale restoration stage
Main survey methods	(Estimation of the presence or absence of cavities) Visual confirmation, sketches, photographs and video recordings	(Estimation of the presence or absence of cavities through nondestructive exploration) Underground radar exploration, on-vehicle subsurface exploration and two-dimensional surface wave exploration (Direct confirmation of the presence or absence and the extent of cavities) Drilling and excavation surveys

4.7.2 Underground Radar Exploration

(1) Outline

The underground radar exploration is a method for surveying objects in the ground by emitting electromagnetic waves into the ground and receiving them reflected from boundaries that have different electric properties (**Fig. 4.7.1**).[27] The boundaries with different electric properties correspond to, for example, cavities, buried pipes, buried objects, stratum boundaries, cracks, crushed zones, the upper surface of structures and waste. The exploration depths of the underground radar exploration are 2 to 3 m in the ground of general soil. Electromagnetic waves with longer periods can explore deeper ground but produce shape information at lower resolutions. Mobile underground radar exploration can be implemented by mounting the equipment on a dedicated cart. Furthermore, underground radar explorations enable the planar exploration of underground objects to be easily implemented by simultaneously obtaining their positional information through RTK-GNSS satellite positioning (**Fig. 4.7.2**).

Fig. 4.7.3 shows an example of the identification of an area suspected to have a cavity through underground radar exploration and confirmation of the cavity through an excavation survey.

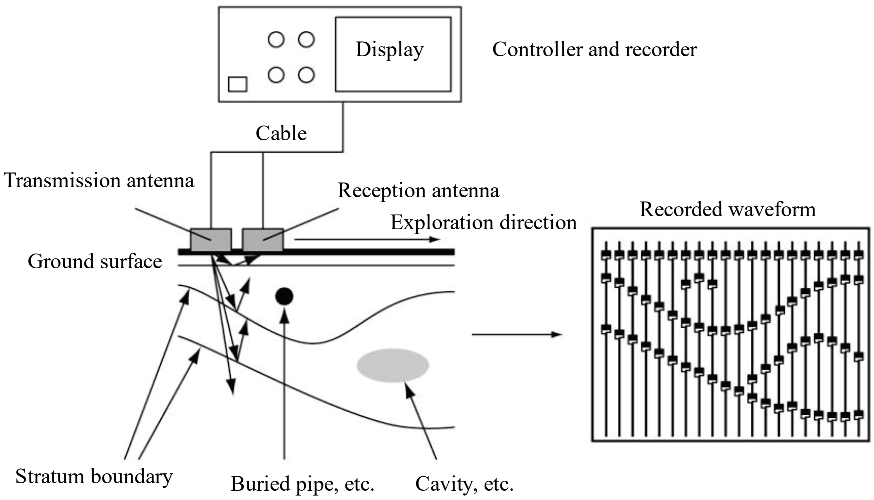


Fig. 4.7.1 Schematic Drawing of Underground Radar Exploration[27]

Fig. 4.7.2 Underground Radar Exploration

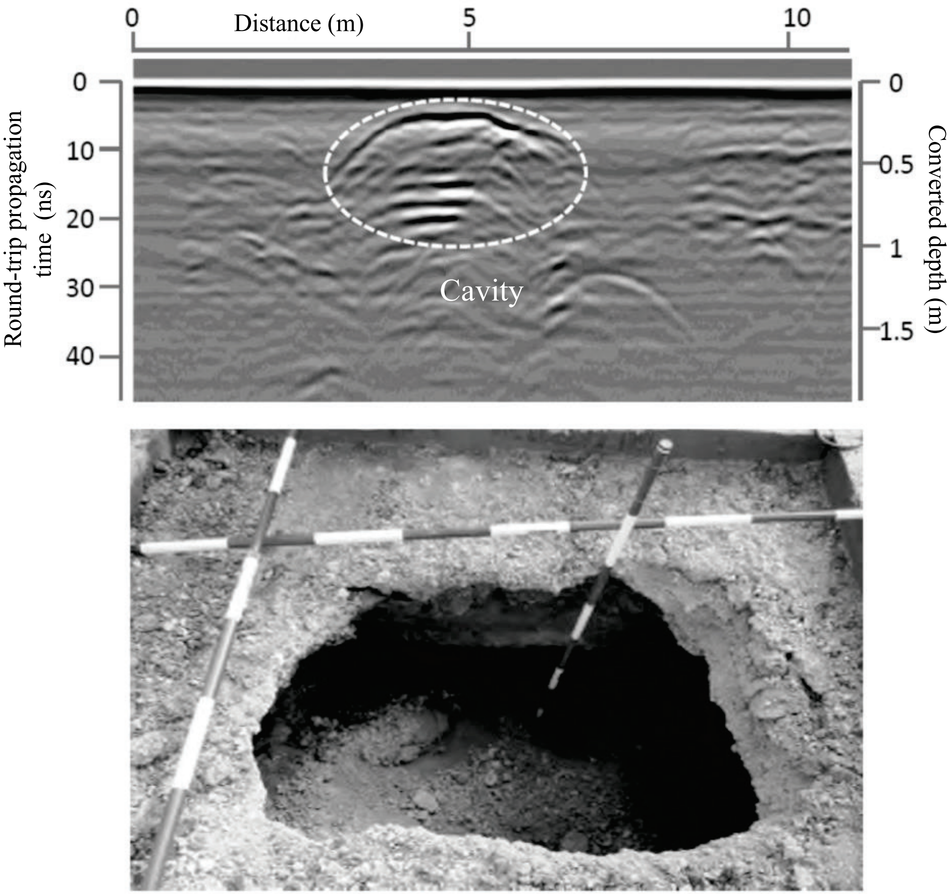


Fig. 4.7.3 Example of a Cavity Identified through an Underground Radar Exploration
(Upper image: record showing a cavity; lower image: photograph of an uncovered cavity)

(2) Obtainable Information

The results of the underground radar exploration can be used for estimating the presence or absence and extent of the cavities in the ground, which cannot be confirmed from the surfaces of the aprons and yards. The information on the cavities in turn can be used for determining the running safety of the construction vehicles, heavy machines and cargo handling equipment (prevention of secondary disasters such as cave-in). In the full-scale restoration stage, the information on the cavities can be used for estimating the work quantities for repairing the concrete pavement in the aprons and filling the cavities, as well as positioning the buried objects that require restoration. It shall be noted that there may be cases where the underground radar exploration cannot produce correct information (e.g., shallower depths than the actual depths when affected by an abnormal specific resistance of ground, as is the case with ground subsidence bringing ground surfaces very close to the residual groundwater accumulated below them).

4.7.3 On-Vehicle Subsurface Exploration

(1) Outline

The on-vehicle subsurface exploration is a method for surveying the presence or absence of cavities below road surfaces with underground radar exploration equipment (refer to **Reference [Part II], Chapter 2, 4.7.2 Underground Radar Exploration**) mounted on a vehicle traveling on the road (**Fig. 4.7.4**). The on-vehicle subsurface exploration enables the presence or absence of cavities and looseness in the ground below road surfaces to be efficiently and speedily surveyed by simultaneously exploring the subsurface conditions in the transverse direction of the roads using multiple radar antennas. The exploration depth of the on-vehicle underground radar exploration is about 2 m.

Fig. 4.7.5 shows an example of a cavity identified through an on-vehicle subsurface exploration.

Fig. 4.7.4 Image of an On-Vehicle Subsurface Exploration[28)]

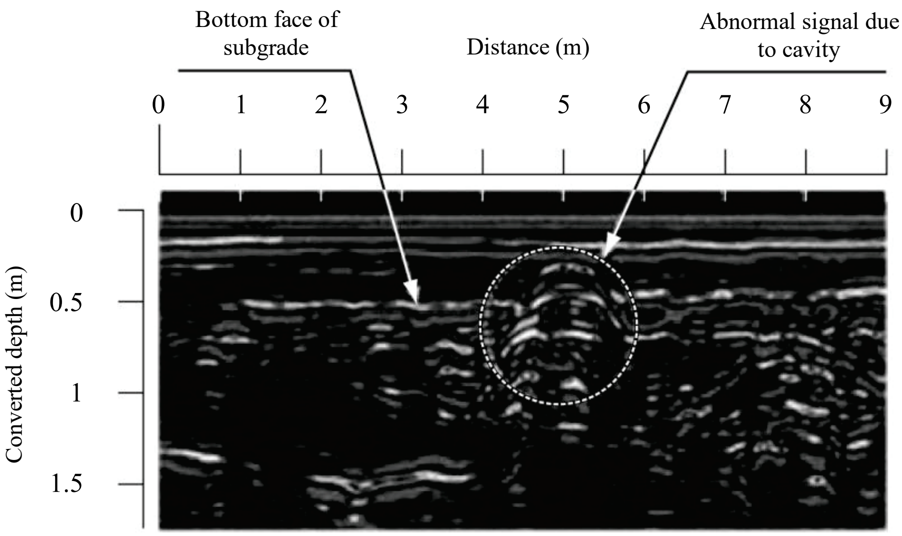


Fig. 4.7.5 Example of a Cavity Identified through On-Vehicle Subsurface Exploration (Vertical Section)

(2) Obtainable Information

The information obtainable through an on-vehicle subsurface exploration is the same as that described in **Reference [Part II], Chapter 2, 4.7.2 Underground Radar Exploration**.

4.8 Understanding the Deformation on Underground Structures

4.8.1 General

The deformation on underground structures driven into the ground, such as the steel pipe piles of piled piers, steel sheet piles of sheet pile quaywalls and foundation piles of cargo handling cranes, needs to be surveyed for obtaining the basic information to investigate the mechanisms of the damage when these structures are subjected to earthquakes and tsunamis, and to facilitate the restoration design of the damaged facilities.

The methods for surveying the deformation of steel pipe piles include verticality measurements with ultrasonic measuring equipment inserted into the steel pipe pile after the soil inside the pile has been removed with a submerged sand pump, and visual identification of the deformed locations by taking video images inside the pile using monitoring equipment, such as a borehole camera inserted into the steel pile after the soil inside the pile has been removed. In addition, impact elastic wave exploration can be used for measuring the lengths of the piles and identifying the locations of breakage on the piles, including concrete piles.

Table 4.8.1 lists the survey methods useful for understanding the deformation on underground structures. From this list, the **impact elastic wave exploration** and the **borehole radar exploration** are described in **4.8.2** and **4.8.3**, respectively.

Table 4.8.1 Main Survey Methods for Understanding the Deformation on Underground Structures

	Full-scale restoration stage
Main survey method	Ultrasonic measuring equipment, video recording inside piles, impact elastic wave exploration and borehole radar exploration

4.8.2 Impact Elastic Wave Exploration

(1) Outline

The impact elastic wave exploration is a survey method for understanding the damage states of concrete piles, steel pipe piles and steel sheet piles driven into the ground. The impact elastic wave exploration, alternatively called the "integrity test," can be used for evaluating the soundness of concrete, measuring the embedment lengths of the piles and the locations of the cracks and breakage by measuring the time required for elastic waves to generate when impacts are applied to an object structure with a hammer to reflect off the tip of the pile or cracks along the way and return to the signal reception sensor.

The signal reception sensor is attached to the upper edge of the object structure, and an impact is applied using a hammer to a point on the structure close to the sensor.

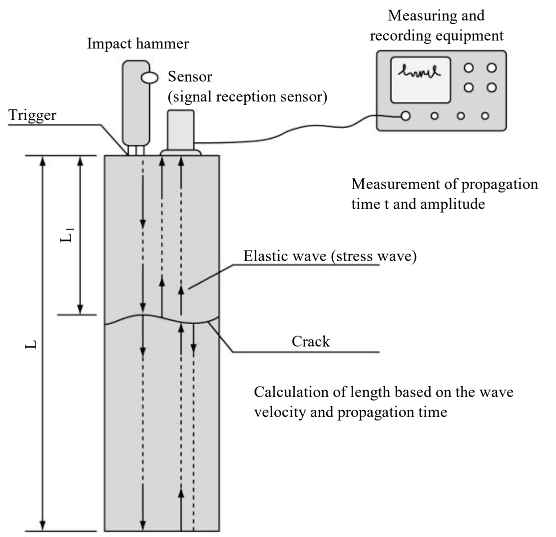


Fig. 4.8.1 Schematic Drawing of an Impact Elastic Wave Exploration[29)]

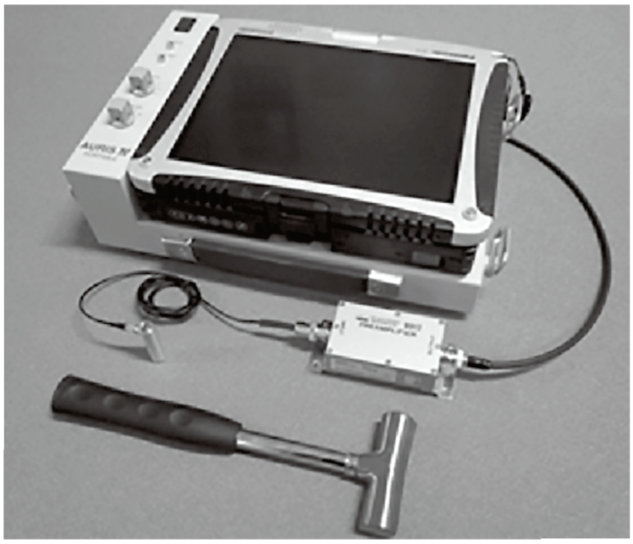


Fig. 4.8.2 Example of Measuring Equipment[29)]

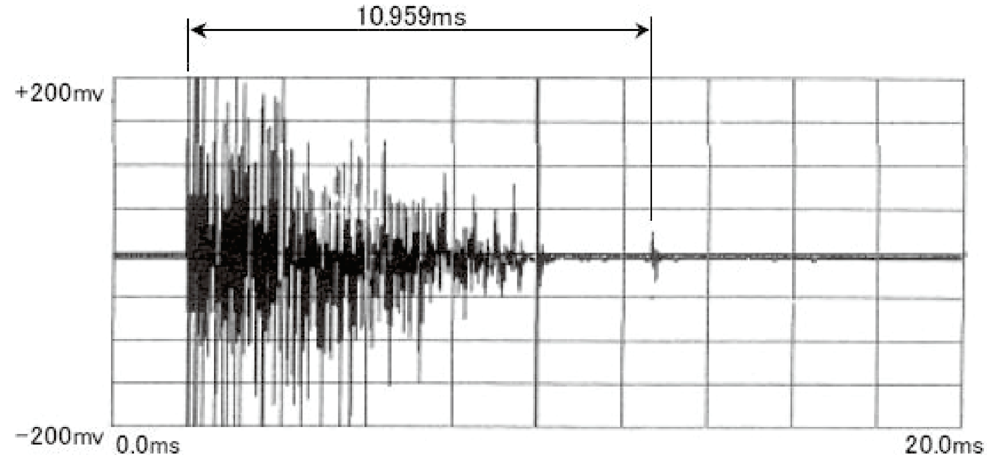


Fig. 4.8.3 Example of the Waveform of an Elastic Wave Propagated in a Steel Pipe Pile in an Impact Elastic Wave Exploration[30)]

Fig. 4.8.3 shows an example recording of the impact elastic wave exploration used for an experimental measurement of the length of a foundation pile from the upper surface of the apron concrete to the superstructure of a piled pier. The waveform of the elastic waves was recorded with a signal reception sensor installed on the upper face of the apron concrete above the center of the pile head when the elastic waves, generated by an impact applied to a point close to the sensor, reflected off the pile tip and returned to the sensor. Based on the propagation time obtained through in-situ exploration and the propagation velocity preliminarily obtained with a test pile, the length of the steel pipe pile on the piled pier was estimated. In the experiment, the effectiveness of the impact elastic wave exploration was reportedly verified with a certain level of allowance.[30)]

(2) Degrees of Damage to Foundation Structures

Ahead of the full-scale restoration of damaged facilities, the impact elastic wave exploration is used for the purpose of understanding the deformation of underground structures, such as the foundations. The impact elastic wave exploration is also one of the most effective methods for confirming whether or not the foundation structures are damaged when the fluidization of the ground, such as liquefaction, has caused the superstructures to have significant displacement.

(3) Points to consider

It is preferable to use the impact elastic wave exploration with due consideration to its applicability to the types and situations of foundation structures that are to be the exploration objects. Because the impact elastic wave exploration requires measurement spaces at the edges of the exploration object structures, it is necessary to coordinate the procedures for the restoration work and exploration work. In the case of foundation piles, it is considered to be practical to start the impact elastic wave exploration with the piles around the structures.

4.8.3 Borehole Radar Exploration

(1) Outline

The borehole radar exploration is a type of underground radar exploration that uses special types of equipment that are usable inside boreholes. The borehole radar exploration can also be used for surveys on geological structures, such as the depths of the boundaries between the soil and bedrock, as well as the locations of the cracks in the bedrock, and for confirming the states of the existing underground structures, such as the embedment lengths of the existing piles.[31)]

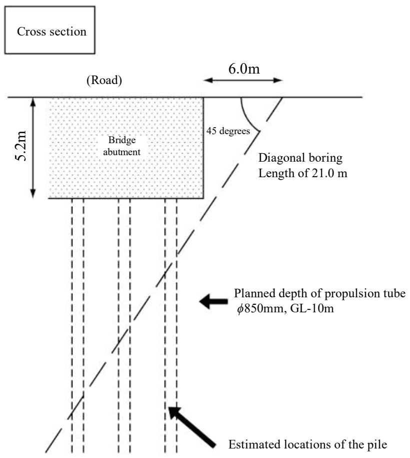


Fig. 4.8.4 Example of a Borehole Radar Exploration Used for Confirming the Existing Piles[31)]

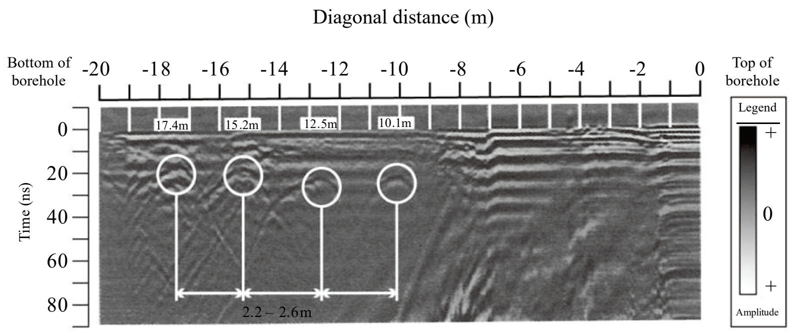


Fig. 4.8.5 Example of a Measurement Record in the Left Example (Fig. 4.8.4)[31)]

Fig. 4.8.5 shows an example of a borehole radar exploration used for understanding the positions of foundation piles of the bridge abutment constructed with positional relationships, as shown in **Fig. 4.8.4**. The positions of the foundation piles were explored in a manner that drilled a diagonal borehole toward the lower section of the bridge abutment, inserted borehole radar equipment into the borehole and explored the existing piles positioned adjacent to the diagonal borehole. The positions of the foundation piles beneath the bridge abutment were calculated based on the measured diagonal distances.

(2) Understanding the Embedment Depths

The borehole radar exploration can be used for compensating for the lack of information on the foundation of the damaged facilities in designing the full-scale restoration work. For example, in the case of a lack of information on the presence or absence, positions and embedment depths of existing foundation piles, a borehole can be drilled as close to these foundation piles as possible, and the measurements of the reflection waves from these piles can be converted into information on the positions and embedment depths of the piles.

(3) Points to Consider

The borehole needs to be positioned close to the exploration object structures (e.g., within 1 m). Furthermore, there may be cases where the borehole needs to be drilled diagonally. Thus, it is necessary to prevent the existing structures from being damaged when drilling the borehole or from being adversely affected by its proximity, as well as confirming its straightness.

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