Chapter 3 Geotechnical Conditions

[Public Notice] (Geotechnical Conditions)

Article 13

Geotechnical conditions shall be set appropriately in terms of the soil's physical and mechanical properties based on the results of ground investigations and soil tests.

[Interpretation]

7. Setting of Natural Conditions

(5) Items related to ground (the Article 6 of the Ministerial Ordinance and the interpretation related to Articles 13 to 15 of the Public Notice)

① **Geotechnical conditions**

The various geotechnical conditions represent the geotechnical characteristics taken into consideration during performance verification of the facilities subjected to the technical standards. The geotechnical conditions shall be set by appropriately determining their reliability based on the results of ground investigations and soil tests carried out by appropriate methods.

② **Ground investigation**

Ground investigations for setting the geotechnical conditions shall appropriately take into consideration the structures, scale, and importance of the facilities subjected to the technical standards, as well as the properties of the ground close to the facilities' locations.

③ **Soil tests**

The soil tests for setting the geotechnical conditions shall be carried out using the methods capable of appropriately setting the geotechnical conditions considering performance verification of the facilities subjected to the technical standards.

1 Ground Investigations

1.1 Methods for Determining Geotechnical Conditions

(1) The geotechnical conditions necessary for performance verification and construction planning include: the stratification conditions of ground, such as depths of the bearing strata, depths of the engineering foundation strata, thicknesses of weak strata; groundwater levels (residual water level); density (degree of compaction and relative density); physical characteristics; shear characteristics; consolidation characteristics; permeability; and liquefaction characteristics. Soil is strongly stress-dependent and greatly changes its mechanical characteristics over time in the case of consolidation or fluctuations in overburden, etc. Therefore, new ground investigations shall be planned and carried out when necessary depending on the situation. However, considering the limited ground investigations which can be actually implemented, it is necessary to proactively use previous information (including databases, etc.) obtained from document surveys. In this case, due consideration shall be given to confirming whether the fluctuations in overburden or progress of consolidation have changed the geotechnical conditions.

1.2 Positions, Intervals, and Depths of Ground Investigation Spots

(1) The positions, intervals, and depths of ground investigation spots shall be determined in accordance with the stress distribution in the ground caused by the sizes and weights of object facilities, and the uniformity in ground stratification. However, in light of the need for due consideration of the construction costs and facilities' importance, it is not possible to categorically regulate the number of investigation spots and their depths. When determining the locations, intervals, and depths of investigation spots, the homogeneity or inhomogeneity of the ground is the most important aspect. Therefore, it is effective to conduct a preliminary check on the homogeneity or non-uniformity of the ground, with reference to the past investigation results, and the topography of land areas, or through geophysical exploration methods, such as sonic wave and surface wave exploration.

Although it is preferable not to determine target intervals of investigation spots mechanically to the extent possible, the target intervals and ranges of boring and sounding shown in **Table 1.2.1** can be used as references.

The ground investigation depths shall be enough to confirm strata with sufficient bearing capacity. Whether or not strata have sufficient bearing capacity depends on the types and scales of facilities and, therefore, cannot be categorically determined. As a guide, however, the ground investigations may be terminated after confirming: the strata which have an SPT-N value obtained through the standard penetration tests of 30 or more and thicknesses of at least several meters in the case of small-scale facilities or facilities having foundation structures other than end-bearing piles; or the strata which have an SPT-N value of 50 or more and thicknesses of at least several meters in the case of large-scale facilities expected to be supported by end-bearing piles. Also, the ground investigations for performance verification of seismic resistance shall be conducted until the strata having shear wave velocities of 300 m/s or more (engineering bedrock) are confirmed.

(2) For details on ground investigation methods, **Reference (Part II), Chapter 1, 3 Ground Investigations and Tests, and Methods and Commentaries for Ground Investigation**[1)] can be used as a reference.

Table 1.2.1 Target Intervals and Ranges of Boring and Sounding Investigation Spots

① In the case where the stratigraphical conditions are comparatively uniform both horizontally and vertically

(Units: m)

		Face line direction		Perpendicular to face line direction			
		Spacing layout		Spacing layout		Distance from face line (maximum)	
		Boring	Sounding	Boring	Sounding	Boring	Sounding
Preliminary survey	Wide area	300–500	100–300	50	25	50–100	
	Small area	50–100	20–50				
Detailed survey		50–100	20–50	20–30	10–15		

② When the stratigraphical conditions are complex (Units: m)

	Face line direction		Perpendicular to face line direction			
	Spacing layout		Spacing layout		Distance from face line (maximum)	
	Boring	Sounding	Boring	Sounding	Boring	Sounding
Preliminary survey	50 or less	15–20	20–30	10–15	50–100	
Detailed survey	10–30	5–10	10–20	5–10		

Note) A sounding survey may or may not require a borehole.
The sounding surveys in the table are only those for which a borehole is not necessary. For sounding surveys that require a borehole, “the boring column” is applicable.

1.3 Selection of Investigation Methods

(1) The investigation methods most suitable for investigation purposes shall be selected considering the scope of investigations, the facilities’ importance, and economic efficiency.

(2) **Table 1.3.1** shows the investigation methods by investigation purposes and ground information obtainable through the methods.

Table 1.3.1 Investigation Methods by Investigation Purposes

Classification	Survey objective	Survey method	Survey details	
Stratigraphical conditions	Confirmation of stratigraphical conditions	Boring Sounding Geophysical exploration	Foundation depth Thickness of weak strata Sequence of strata	
Physical characteristics	Classification of soil properties	Undisturbed sampling (disturbed sampling is possible for all except γ_t, but pay attention to the discharge of fine grain fraction or particle crushing, etc.)	Wet unit weight Water content Soil particle density Particle size distribution Consistency	γ_t w ρ_s w_L, w_P, I_P
(Hydraulic conductivity)	Hydraulic conductivity	Undisturbed sampling In-situ tests	Hydraulic conductivity	k
Mechanical properties	Bearing capacity Slope stability Earth pressure	Undisturbed sampling Sounding In situ tests	Undrained shear strength Unconfined compressive strength Shear strength Cohesion Angle of shear resistance Relative density	c_u q_u τ_f c φ D_r
	Consolidation characteristics	Undisturbed sampling	Compression index Compression curve Coefficient of consolidation Coefficient of volume compressibility Coefficient of secondary consolidation	C_c e-log p c_v m_v C_a
	Compaction characteristics	Disturbed sampling also applicable In- situ tests	Maximum dry density Optimum water content CBR	$\rho_{d\max}$ w_{opt}
	Dynamic characteristics	Undisturbed sampling In- situ tests	Shear modulus Attenuation coefficient Liquefaction characteristics	G h_P

(3) Types of ground investigation methods

① Collection and analyses of existing materials

It is necessary to examine the outline of the ground properties of investigation areas through boring data, soil databases, records of groundwater levels, deformation of neighboring structures, records of ground settlement, and ground formation processes (geological information in the case of natural ground and development records in the case of artificial ground).

② Field surveys for geological and topographical conditions

Although the field surveys which are generally practiced for onshore ground investigations are not available for offshore ground investigations, the onshore field surveys in the areas close to offshore investigation spots are still important, even for offshore ground investigation, because offshore geology has close relation with onshore geology.

③ Geophysical explorations (elastic wave exploration, surface wave exploration, electric resistance)

Because geophysical explorations can be implemented on ground surfaces, they can be used to obtain information on the stratification conditions and groundwater levels in extensive areas. However, it is preferable to verify the soil properties and depths obtained through the geophysical explorations by comparing them to boring data to the extent possible.

④ Acoustic exploration

Acoustic exploration is effective to grasp offshore geology promptly. These technologies were originally developed to explore mineral resources like crude oil; therefore, they have been used for deep ground explorations. Owing to the improvement in sound sources, receivers, and processing systems, as well as

enhanced positioning accuracy with the use of GPSs, however, there has been considerable improvement in acoustic exploration accuracy. Still, there may be cases where the gases and gravel in ground cause acoustic waves to diffuse before reaching predetermined depths. Also, it shall be noted that the values obtained through acoustic exploration are not distances, but reflection time of compression waves. Thus, it is always necessary to combine acoustic exploration with boring or sounding, which enables depths to be measured reliably so as to confirm which reflecting layers obtained through the acoustic exploration correspond to which geological strata obtained through boring or sounding.

⑤ **Boring**

Boring is conducted to create soil profiles by getting information on stratification conditions and soil types through observing mud water from boreholes while drilling them. Also, the boreholes are used for sampling specimens and implementing sounding, such as the standard penetration test. To sample high quality specimens or obtain high quality sounding test results, it is necessary to implement boring so as not to disturb soil to the extent possible. For this purpose, boring is generally implemented with rotary type mechanical boring machines, which are considered to least disturbing to soil. When obtaining various constants necessary for performance verification, it is preferable to implement sampling for cohesive soil and the standard penetration test for sandy soil.

⑥ **Sampling**

When examining cohesive soil properties, some tests require undisturbed (less disturbed) specimens while others do not. When conducting mechanical tests to obtain shear strength and consolidation characteristics, it is preferable to use undisturbed specimens sampled with thin-walled samplers with fixed pistons. When such samplers are not available, undisturbed specimens can be obtained with rotary double tube samplers (Denison samplers) or rotary triple tube samplers. Sampling is to acquire less disturbed specimens which are used in laboratory tests as undisturbed specimens for obtaining the characteristic values of constants used in performance verification. The specimen collection device attached to the tip of equipment used for the standard penetration test is also called a sampler. However, there is no reason to use such a collection device for purposes other than soil classification. It is necessary to keep in mind that sampling is not just to collect soil, but to collect specimens subjected to soil tests.

⑦ **Sounding**

The methods for examining ground properties are classified into laboratory tests using specimens obtained through sampling and sounding (alternatively called in-situ tests) to obtain soil parameters with measuring devices directly inserted into, rotated inside, or pressed against the ground. Sounding is a collective term of methods which measure parameters (SPT-N values, tip resistance, pore water pressure, etc.) through in-situ tests and indirectly estimate soil parameters using the relational formula (empirical or theoretical formula) showing the relationships between the parameters and the soil parameters. Sounding is further classified into the tests using boreholes drilled to the depths where soil is tested in-situ, as is the case with the standard penetration test and the tests which can be implemented (without using boreholes) in a manner that directly inserts cone probes into ground, as is the case with the electric cone penetration test.

The comparisons of the characteristics of the laboratory tests and sounding are as follows.

(a) Disturbance

No matter how carefully specimens are sampled, they are inevitably subjected to disturbance due to stress release when exposed to atmospheric pressure. In contrast, sounding measuring parameters through in-situ tests are less likely to be affected by such disturbance.

(b) Boundary conditions

The triaxial test is a good example for easily understanding the difference in boundary conditions between laboratory tests and sounding. The boundary conditions of laboratory triaxial tests are clearly known, and the laboratory triaxial test, which can control drainage conditions, is advantageous in that soil shows different mechanical behavior depending on drainage conditions. In contrast, the boundary conditions of sounding, for example, stress, strain, and drainage conditions, are unclear and the sounding results require rarely theoretical but mostly empirical interpretations.

(c) Time and costs

Generally, laboratory tests take a long time and are expensive. In contrast, sounding, which makes measurement results readily available in-situ, can reduce costs. In addition to the above, there are many

laboratory test and sounding methods. Thus, it is necessary to select appropriate methods suitable for the types of ground, stratification conditions, and the types, as well as required accuracy, of object soil parameters.

[Reference]

1) The Japanese Geotechnical Society: JapaneseGeotechnical Society Standards, Geotechnical and Geovironmental Investigation methods, 2013.

2 Geotechnical Properties

2.1 Estimation of Geotechnical Properties

(1) General

The design values of geotechnical propaerties used in performance verification are, in principle, estimated in accordance with the procedure[1)] shown in **Fig. 2.1.1** following the **Design Principle for Foundation Structures based on the Performance Design Concept (JGS 4001)**. However, if there is a rational reason based on the characteristics of the ground investigations and the soil tests, derived values may be used as characteristic values. For example, in the case of measured SPT-N values through the standard penetration test, derived values can be used as characteristic values because there have been proposals of empirical and correlation equations, taking the variations in the measured values into consideration. Also, as with shear wave velocities measured by the geophysical logging, some measured values are obtained from evaluating the complex in-situ conditions and characteristics of the ground, and each measurement location has a different evaluation object. In these cases, derived values may also be used as characteristic values because statistical processing of plural measurement result is inappropriate.

Also, it is difficult to take into account the extent of individual influences of ground investigation or soil test methods on the variations of ground constants in each performance verification case. Thus, assuming that reliability of ground investigation or soil test methods appears in the form of data variations, characteristic values are subjected to corrections according to the variations. This approach simplifies the performance verification method, enabling partial factors (load resistance factors) to be set regardless of ground investigation and soil test methods. It shall be noted that the characteristic value to be set when the number of data is small or data has large variations is slightly different from the essential concept of making the average derived value a characteristic value, as stipulated in **JGS 4001**.

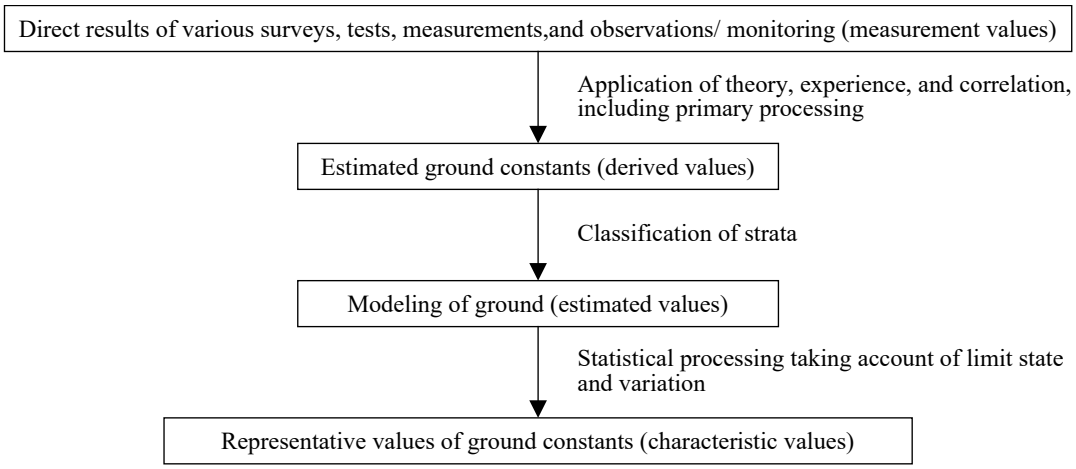


Fig. 2.1.1 Examples of the Procedure for Setting the Characteristic Values of Ground Constants

(2) Methods of estimating derived values

As described below, derived values can be obtained from measured ones through either method which: uses measured values directly as derived ones; applies primary processing to measured values; or converts measured values into different engineering quantities.

① The methods which use the measurement values directly as derived ones are, literally, direct ground constant measurements.

② The methods which apply primary processing to measured values include: area correction in shear tests; correction for the strain rates' effect on shear strength; and simple correction by multiplying measured values by coefficients. The primary processing also includes simple processing of test results such as calculating water contents w, wet density p_t, soil particle density ρ_s, and grain sizes; obtaining deformation moduli E from stress-strain relationships; and obtaining consolidation yield stress p_c from the e–log p relationship in compression curves.

③ The methods which convert measured values into different engineering quantities use theoretical or empirical formulas, or obtain fitting parameters, in accordance with theoretical formulas. The methods include: converting SPT-N values into angles of shear resistance ϕ using empirical formulas and obtaining consolidation coefficients c_v by fitting theoretical consolidation curves to settlement-time curves.

(3) Methods of setting characteristic values

① General

Characteristic values are set generally in accordance with the procedure shown in **Fig. 2.1.2**. When the number of derived values is large enough to be subjected to statistical processing, and the variations of the derived values are small, characteristic values can be calculated as the averages (expected) values of the derived ones in principle. Given that the number of the data of derived values n is 10 or more, and they have no significant variation with a coefficient of variation of less than 0.1, the statistical processing results of such data are considered to have a certain level of reliability, enabling their average (expected) values to be characteristic values. However, if there is an insufficient number of data on the derived values to carry out statistical processing, and the variation in the derived values is large, it is necessary to set characteristic values by correcting their average values (expected values) through the method shown below.

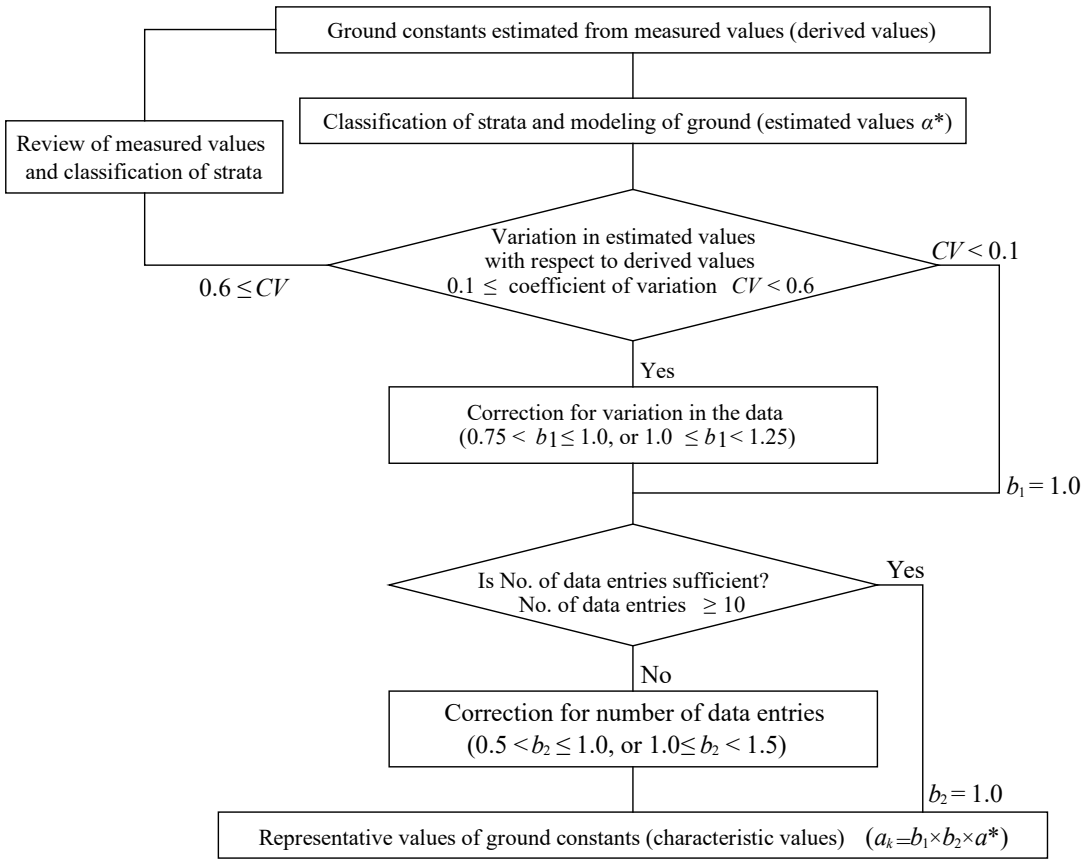


Fig. 2.1.2 Example of the Procedure for Setting the Characteristic Values of Ground Constants

② Correction of the average (expected) values of derived values

When the number of derived values is limited, or the variation in the derived values is large, the characteristic values shall be set not by simply and automatically obtaining the average (arithmetic average value) of the derived values but by appropriately taking into consideration the estimated error of the statistical average value. In this case, the following method may be used. Because characteristic values have such uncertain factors as errors in the ground investigations and soil tests, estimation errors in the derived values, and inhomogeneity in ground itself, it is desirable to determine characteristic values carefully, with due consideration of the ground investigation conditions (such as the types of survey equipment), soil test conditions (such as the types of test equipment and methods and condition of test specimen), and other soil information, such as stratal

organization. The method of correcting the average (expected) of the derived values described here is expected to be applied not only to the values for stability verification of facilities, but also to soil constants in general, including settlement prediction values. The method specified in **JGS 4001** is to set the characteristic values in accordance with confidence intervals, assuming that derived values show a normal distribution, if the standard deviation of the population is known, or a *t*-distribution if the standard deviation is unknown. However, unlike quality indices for factory products, simple statistical processing is difficult when dealing with geotechnical parameters because of errors attributable to ground investigation and soil test methods, estimation errors of derived values, and the distribution and variations in the derived values attributable to the inhomogeneity of ground itself as well as sedimentation conditions.

To obtain geotechnical parameters (the averages, corresponding to characteristic values, with statistical errors incorporated into them) for reliability design, it is necessary to collect a sufficiently large number of test results for statistical processing. Also, to reflect the soil investigation and soil test results in performance verification, it is necessary to model the distribution in the depth direction of the estimated values a^* of the geotechnical parameters (expressed by "a" here), for example: uniform distribution in the depth direction ($a^* = c_1$); linear distribution with estimated values increased in proportion to depth ($a^* = c_1 z + c_2$); and quadratic distribution in the depth direction ($a^* = c_1 z^2 + c_2 z + c_3$). Here c_1, c_2, and c_3 are constants. At least 10 pieces of test data are required when a portion of ground is modeled to a certain depth and the model is subjected to statistical processing. The geotechnical parameters obtained through different soil tests (such as undrained shear strength by triaxial test and unconfined compression tests) have different reliability. Therefore, different partial factors (load resistance factors) shall be set for respective parameters; however, there is no way of knowing the degrees of difference in these partial factors. In contrast, it is well-known that the variation coefficients of these two tests results are significantly different. Based on this, the characteristic values are to be calculated not simply by making the arithmetic averages, but by multiplying the estimated values by the correction coefficients that take into account the variation of the derived values. This method is based on having sufficient test data for statistical processing. Therefore, when there is insufficient test data, it is necessary to further set the characteristic values on the safe side in a manner that multiplies the estimated values by the correction coefficients with respect to insufficient test data. In other words, the characteristic values are calculated either by **equation (2.1.1)** or **equation (2.1.2)**. Here, **equation (2.1.2)** is used when it is reasonable to examine the variations of, for example, consolidation yield stress p_c, consolidation coefficients c_v, volume compressibility coefficients m_v, etc. on logarithmic axes.

$$a_k = b_1 b_2 a^* \tag{2.1.1}$$

$$\log a_k = b_1 b_2 \log a^* = \log a^{*b_1 b_2} \tag{2.1.2}$$

where

a_k : a representative value of a ground constant (characteristic value);

b_1 : a correction coefficient with respect to the variation in the derived values;

b_2 : a correction coefficient with respect to the number of data on derived values; and

a^* : a model value of the ground constant (estimated value).

A specific correction method (correction coefficient setting method) is described below. When dealing with quantities considered to have balance between action and bearing sides in essence, as is the case with the unit weight of original ground in stability analyses, the correction coefficient values b_1 and b_2 can be set at 1.

③ **Method of setting correction coefficients with respect to derived value variations**

When examining the variation of test results a with the estimated geotechnical parameters, obtained by modeling the distribution of the test results expressed by a^*, it is convenient to use the standard deviation a/a^*, the normalization of a with a^* (called a coefficient of variation, or *CV*). Here, this is based on the major premise that a^* is uniformly distributed in a model stratum at its average value or distributed in a manner that enables the least square method to minimize errors. The *CV*s of the geotechnical parameters, obtained by sampling less disturbed specimens from uniform ground with a thin-walled tube sampler with a fixed piston, and carefully conducting variety of soil tests using the sampled specimens as undisturbed specimens, are 0.1 or less. In other words, test results inevitably vary at this level because even homogeneous ground has a certain

amount of inhomogeneity, and even carefully conducted soil tests are subjected to errors. Test results may have larger variations in cases where the ground is inhomogeneous, sampling causes large disturbance in specimens, soil test methods are conducted improperly, or the ground is modeled with inappropriate distribution of values in depth direction. In such cases, characteristic values need to be set on the safe side considering the effects of uncertain factors without applying estimation values a^* directly to the characteristic values.

Therefore, the correction coefficient b_1 with respect to the variations of the derived values are set in accordance with the *CV*s, defined as the standard deviations *SD* of (a/a^*). When an object parameter a contributes to the bearing side (advantageous for design such as shear strength) in performance verification, the correction coefficient can be set at about $b_1 = 1 - (CV/2)$. When contributing to the action side (disadvantageous for design such as the unit weight of earth fill and compression indexes), the correction coefficient can be set at about $b_1 = 1 + (CV/2)$. Based on this concept, the values to be used in performance verification are calculated and summarized as shown in **Table 2.1.1**. The concept of the correction coefficient b_1 is to apply the derived values corresponding to the cumulative probability density of about 70% (called fractal values) to the characteristic values. If the *CV*s are 0.6 or higher, the test results are unreliable for performance verification. In such a case, the interpretation of test results shall be revised and, if necessary, the ground modeling shall be reexamined. There may be a case of redoing ground investigations.

Table 2.1.1 Values of Correction Coefficients

Coefficient of variation *CV*	Correction coefficient b_1	
	When it is necessary to correct the characteristic value to a value smaller than the derived values	When it is necessary to correct the characteristic value to a value larger than the derived values
≥ 0, < 0.1	1.00	1.00
≥ 0.1, < 0.15	0.95	1.05
≥ 0.15, < 0.25	0.90	1.10
≥ 0.25, < 0.4	0.85	1.15
≥ 0.4, < 0.6	0.75	1.25
≥ 0.6	Re-investigate the interpretation of the results or the modeling, or re-do the survey	

There are cases of examining the logarithmic distribution of test results when obtaining some geotechnical parameters, such as consolidation yield stress p_c, the consolidation coefficients c_v, and the volume compressibility coefficients m_v. When obtaining the characteristic values of these geotechnical parameters by conducting a large number of soil tests, assuming the object ground is uniform, it is reasonable to examine the variations on logarithmic axes because these geotechnical parameters show logarithmic normal distribution. That is, the *CV* can be expressed by the standard deviations *SD* of $\log a/\log a^*$ with respect to the geotechnical parameter a and, therefore, the values in **Table 2.1.1** can be used directly as the correction coefficient b_1 on the logarithmic axes. In the case of the angles of shear resistance ϕ, the variations of $\tan\phi$, not the variations of ϕ, should be examined by taking their mechanical significance into consideration. However, there is no need to consider *CV* when dealing with the angles of shear resistance of mound materials because the characteristic values to be used for performance verification have already been specified empirically, and the influences of the variations are already incorporated in these values. Here, the *CV* need to be applied to the characteristic values obtained from statistical processing of reported soil test results. In other words, **Table 2.1.1** does not show the required levels of variations that ground investigation and soil test results need to satisfy, but the values corresponding to the variation levels required when evaluating ground investigation and soil test results.

④ **Method of setting correction coefficients with respect to the number of data on derived values**

In ③ Method of setting correction coefficients with respect to derived value variations, the method is based on the availability of sufficient data to conduct statistical processing. However, if there is insufficient data for statistical processing, the correction coefficients b_2 with respect to the number of data on derived values shall be applied based on the concept that statistical results cannot have a certain degree of reliability unless the number of data is 10 or more. The characteristic values shall be corrected by $b_2 = \{1 \pm (0.5/n)\}$ when there is insufficient data. Here, in the formula of b_2, the negative sign is used to correct the characteristic values of geotechnical parameters used in performance verification if they should be smaller than the derived values, and the positive sign is used to correct the characteristic values if they should be larger than the derived values. For

performance verification, there must be two or more data on derived values. However, even in the case where there is only one piece of data on a derived value, the data can still be used for performance verification provided that other parameters (for example SPT-N values or grain size distribution) are available and the distribution of the derived values can be modeled based on the correlation (limited to generally known correlation) between the derived values and the parameters. In such a case, b_1 and b_2 shall be set at 1 and 1 ± 0.5 respectively.

⑤ **Method of setting characteristic values taking into consideration modes of the performance verification**

Soil parameters with respect to consolidation and shear strength are not mutually independent. In performance verification, if these parameters are considered independent, the characteristic values can be obtained by taking the reliability of the respective parameters into consideration. However, the parameters with respect to consolidation need to be closely related to those with respect to shear strength. For example, the stability evaluation needs to consider the effect of consolidation on strength increases. In this case, in the process of obtaining characteristic values from derived values, the respective parameters must be correlated when modeling the distribution of soil test results and obtaining the estimated values. For example, given the relationship of $c_u = m \times \mathrm{OCR} \times \sigma'_{v0}$ derived from the strength increase ratio of $m = c_u/p_c$ and the overconsolidation ratio of $\mathrm{OCR} = p_c/\sigma'_{v0}$ where σ'_{v0} is an effective soil overburden pressure, p_c is consolidation yield stress, and c_u is undrained shear strength, the characteristic values are preferably set through the statistical processing of the variations based on the estimated geotechnical parameters consistent with the relationship.

2.2 Physical Property of Soil

2.2.1 Unit Weight of Soil

(1) The unit weight of soil shall be obtained through laboratory tests by 1) following the **test method for bulk density of soils (JIS A 1225)** after collecting less disturbed specimens and preserving in-situ conditions; 2) directly through in-situ tests following the **test method for soil density by the sand replacement method (JIS A 1214)**; or 3) through indirect tests following the **test method for soil density using a nuclear gage (JGS 1614)**.

(2) The unit weight normally means the weight per unit volume in air, including the wet unit weight and dry unit weight. Also, the unit weight in water (weight per unit volume from which buoyancy is deducted) is referred to as the submerged unit weight. To measure the unit weight, methods of collecting the less disturbed specimens of cohesive soil have been established, and it is possible to obtain specimens representing in-situ conditions. Therefore, the unit weight of cohesive soils can be obtained from laboratory tests. However, the unit weight of sandy soils or sand must be obtained directly in-situ. For the unit weight of sandy soil or sand, there are cases of obtaining it through in-situ density logging or RI cone penetration tests.

The wet unit weight is one of the indices indicating the soil's fundamental properties and is used to calculate soil stiffness, degrees of looseness, the weight of soil masses and void ratios.

① **Wet unit weight**

The wet unit weight is generally expressed by **equation (2.2.1)** as the sum of the weight of soil particles and the weight of water (seawater at sea bottoms) within the voids per unit volume.

$$\gamma_t = \rho_t g = \frac{\rho_s + \frac{S_r}{100} e \rho_w}{1+e} g = \frac{1+\frac{w}{100}}{1+e} \rho_s g \qquad \textbf{(2.2.1)}$$

where

γ_t : wet unit weight (kN/m^3);

ρ_t : bulk density (t/m^3);

ρ_s : soil particle density (t/ m^3);

e : a void ratio;

S_r : the degree of saturation (%);

w : a water content (%);

ρ_w : the density of seawater (t/m^3); and

g : gravitational acceleration (m/s^2).

The approximate values of the unit weight of soil in port and harbor areas in Japan are generally in the ranges shown in **Table 2.2.1** or in the ranges of the histogram shown in **Fig. 2.2.1**. As can be seen in **equation (2.2.1)**, the unit weight has a close relationship with moisture contents. The relationships between the unit weight and moisture contents are shown in **Fig. 2.2.2** and **2.2.3**.

Table 2.2.1 Unit Weight and Water Contents of Typical Soil

	Holocene clays	Pleistocene clays	Sandy soils
Wet unit weight γ_t (kN/m^3)	12–16	16–20	16–20
Dry unit weight γ_d (kN/m^3)	5–14	11–14	12–18
Water content w(%)	150–30	60–20	30–10

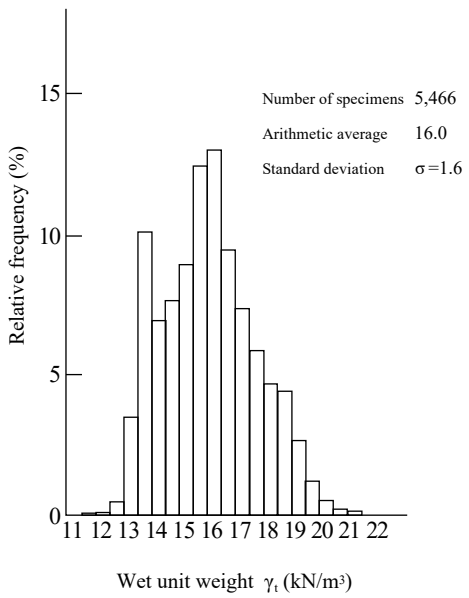


Fig. 2.2.1 Histogram of Wet Unit Weigtht[2)]

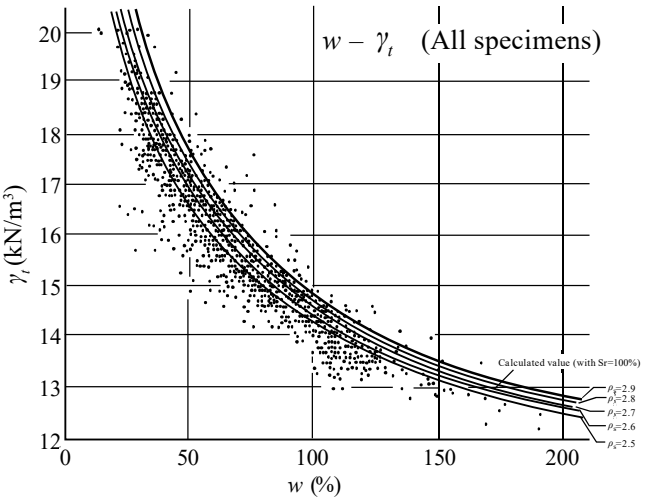


Fig. 2.2.2 Correlation Diagram between Wet Unit Weight and Moisture Contents[3)]

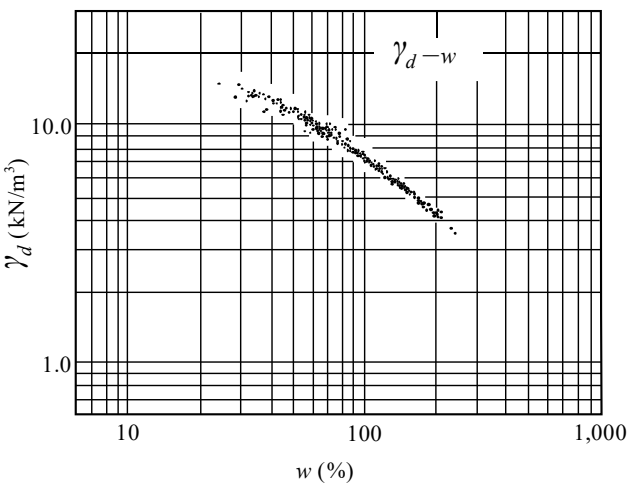


Fig 2.2.3 Correlation Diagram between Dry Unit Weight and Moisture Contents[2)]

② **Dry unit weight**

The dry unit weight considers only the weight of soil particles per unit volume of soil and is expressed by **equation (2.2.2)** which can be obtained by substituting 0 for w and S_r in **equation (2.2.2)**.

$$\gamma_d = \rho_d g = \frac{\rho_s g}{1+e} \tag{2.2.2}$$

where

γ_d : dry unit weight (kN/m^3); and

ρ_d : dry density (t/m^3)

Also, the relationship between wet unit weight γ_t and dry unit weight γ_d is given by the following equation.

$$\gamma_d = \frac{\gamma_t}{1+\frac{w}{100}} \tag{2.2.3}$$

③ **Submerged unit weight**

Given that the voids are fully saturated with water, the submerged unit weight can be expressed by **equation (2.2.4)** taking buoyancy into account.

$$\gamma' = \gamma_{sat} - \gamma_w = \frac{\rho_s - \rho_w}{1+e} g \tag{2.2.4}$$

where

γ' : immersed unit weight (kN/m^3);

γ_{sat} : saturated unit weight (kN/m^3); and

γ_w : the unit weight of water (or seawater at sea bottom) (kN/m^3).

Although the unit weight of water γ_w depends somewhat on salt concentrations and temperatures, correct values are known. Therefore, such dependency does not cause variations of the unit weight of water. Thus, when obtaining the characteristic values of saturated foundations, considering the variations in their unit weight, variations not in γ_{sat} but in γ' can be considered.

The histogram of the density ρ_s of soil particles is shown in **Fig. 2.2.4** for reference.

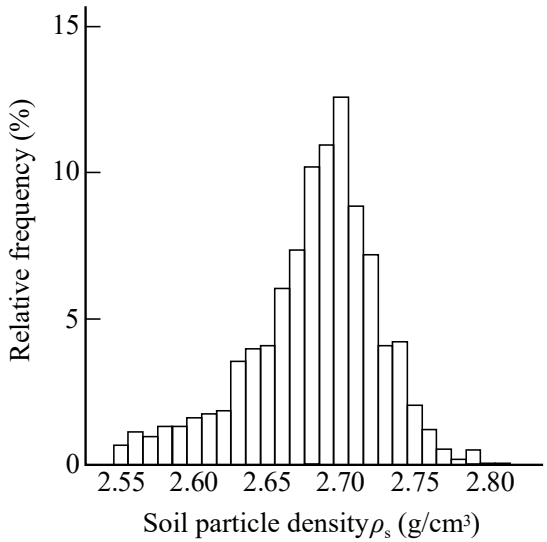


Fig. 2.2.4 Histogram of the Soil Particle Density

(3) Measurement of in-situ unit weight

The methods for directly obtaining the in-situ unit weight include those applicable only to the soil close to ground surfaces and those applicable to soil at deep depths. A simple method, specified in **JIS A 1214** the **test method for soil density by the sand replacement method** (the so called sand replacement method), is an example of the former methods. Also, the **test method for soil density using nuclear gage** specified in **JGS 1614** is an example of the latter methods.

① Sand replacement method

The sand replacement method is mainly used for measuring soil close to land surfaces to manage earth works, but it can be applied to the measurement at a certain depth where pits can be excavated. For the details of the sand replacement method, refer to **JIS A 1214**.

② Method using radioisotope (RI)

In recent years, RI has become available relatively easily. Although there are strict statutory regulations for using RI, such as the **Act on Prevention of Radiation Hazards due to Radioisotopes, etc.** and its related laws and ordinances, there have been many cases of measuring in-situ unit weight with γ-ray densitometers when it is difficult to obtain undisturbed specimens of sand and sandy soil. Also, there are no legal restrictions on using sealed radioactive sources with source intensities of 3.7 MBq (megabecquerel) or less.

As introduced in one of the standards of the Japanese Geotechnical Society, titled the **Test Method for Soil Density Using Nuclear Gauge (JGS 1614)**, there are two types of γ-ray densitometers to which RI has been applied: one is a surface type and the other is an insertion type. Just like the name implies, the surface type is suitable for measuring near the ground surface and is used to manage earth work, as is the case with the sand replacement method. The surface type is further classified into a back scattering system and a transmission system. The back scattering system had been used frequently soon after the development of γ-ray densitometers but, in recent years, the transmission system, which is advantageous in terms of accuracy, has become more popular. On the other hand, the insertion type is suitable for measuring the density distribution in the vertical direction or depth direction. For example, the insertion type is used to examine the density distribution in the depth direction in: foundation ground investigations; determining soil improvement effects through density measurement of replaced sand; and the density measurement of filling sand in caissons.

The RI method is advantageous in that it is nondestructive, capable of measuring in-situ density, and can be implemented with simple measurement operation. However, depending on the level of radiation danger associated with the intensity of radioactive sources, there are several regulations on RI equipment operation. Thus, the RI method cannot be implemented easily. In addition, in surveys associated with port construction, the type of γ-ray densitometer which has been used mainly is the insertion type, requiring access pipes to be inserted into ground. In this respect, measurement accuracy is affected by the materials, quality, and insertion conditions of the access pipes; in particular, the degree of disturbance of the soil around the access pipes when they are inserted and the adhesiveness between the access pipes and soil. Recently, a new type of RI equipment,

called the RI cone penetrometer, has been commercialized. The RI cone penetrometer incorporates RI in a cone probe, so measurements can be made with the cone probe inserted directly into the ground.

(4) Relative density

The degree of compaction (looseness) of sand can be expressed by the relative density defined by **equation (2.2.5)**.

$$D_r = \frac{e_{\max} - e}{e_{\max} - e_{\min}} = \frac{\rho_d - \rho_{d\min}}{\rho_{d\max} - \rho_{d\min}} \frac{\rho_{d\max}}{\rho_d} \qquad \textbf{(2.2.5)}$$

where

D_r : relative density;

$e_{\max}$: the void ratio in the loosest state;

$e_{\min}$: the void ratio in the densest state;

e : the void ratio of a specimen in the present state;

$\rho_{d\min}$: dry density in the loosest state (g/cm^3) or (t/m^3);

$\rho_{d\max}$: dry density in the densest state (g/cm^3) or (t/m^3); and

ρ_d : dry density of a specimen in the present state (g/cm^3) or (t/m^3).

Sand density is greatly affected by the shapes and grain size compositions of particles. The typical parameters expressing sand density as absolute quantities are the unit weight and the void ratios calculated from it. However, the dominant parameter expressing sand's mechanical properties is the relative density D_r which shows the degree of compaction; in other words, the relative value within the void ratio range that object sand can possibly take. The values of e_{max}, e_{min}, ρ_{dmin}, and ρ_{dmax} necessary for obtaining D_r can be measured in accordance with the **Test Method for Minimum and Maximum Densities of Sands (JIS A 1224)**.

Because it is difficult to take undisturbed specimens of sand, there are many cases of indirectly obtaining the relative density through sounding (Refer to **2.3.4(4) Angle of shear resistance of sandy ground**).

2.2.2 Classification of Soil

(1) Soil is classified in terms of the grading for coarse soil and the consistency, which represents the degrees of softness or hardness of soil affected by moisture contents, for fine soil in general case.

(2) The mechanical properties of soil, such as strength and deformation, have close relationships with grading in the case of coarse soil and its consistency in the case of fine soil.

(3) Engineering classification method of subsoil materials (the Japanese unified soil classification system)

The methods for classifying soil and rock and expressing the classification results can conform to the **Engineering Classification Method for Subsoil Material Prescribed (JGS 0051)** (Japanese Unified Soil Classification System). The grain size classifications of subsoil materials and their nominal designations are shown in **Fig. 2.2.5**. Coarse soil consists primarily of the grain components having grain sizes in the range from 75 μm to 75 mm. Fine soil consists primarily of grain components having grain sizes smaller than 75 μm. **Fig. 2.2.6** and **Fig. 2.2.7** show the engineering soil classification system, and **Fig. 2.2.8** shows the plasticity chart used in the classification of fine soil.

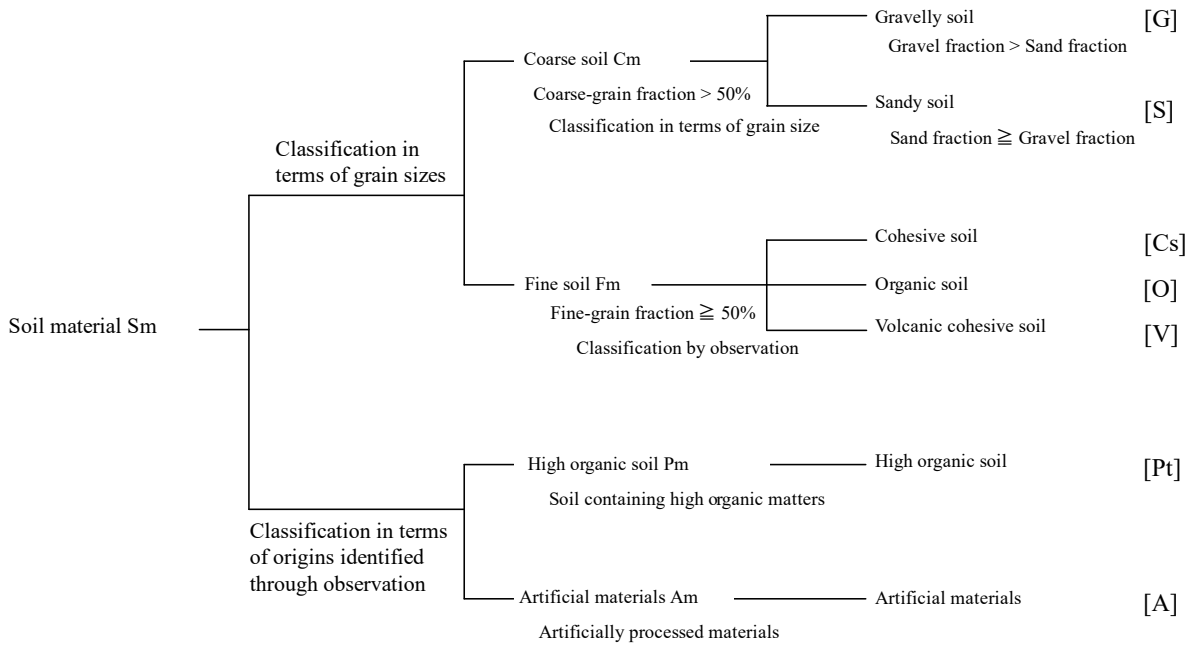
Particle Diameter (mm)

0.005	0.075	0.250	0.850	2	4.75	19	75	300

Clay	Silt	Fine sand	Medium sand	Coarse sand	Fine gravel	Medium gravel	Coarse	Cobble	Boulder
		Sand			Gravel			Stone	
Fine grain fraction		Coarse grain fraction						Stone fraction	

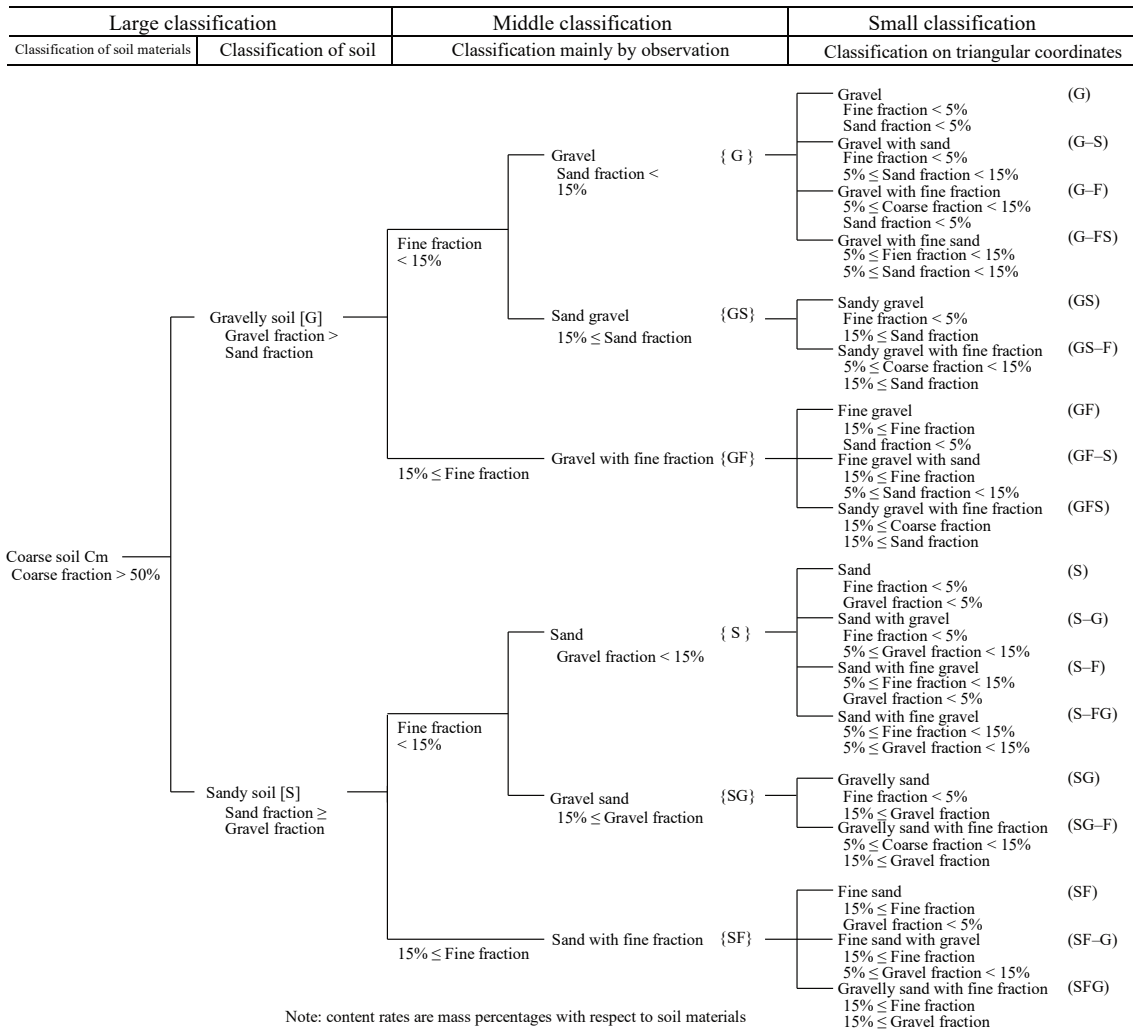
(Note) The word "particle" is affixed when referring to a constituent particle belonging to a particular category; and the word "fraction" is affixed when referring to a component belonging to a particular category.

Fig. 2.2.5 Grain Size Classifications and their Nominal Designations (JGS 0051)

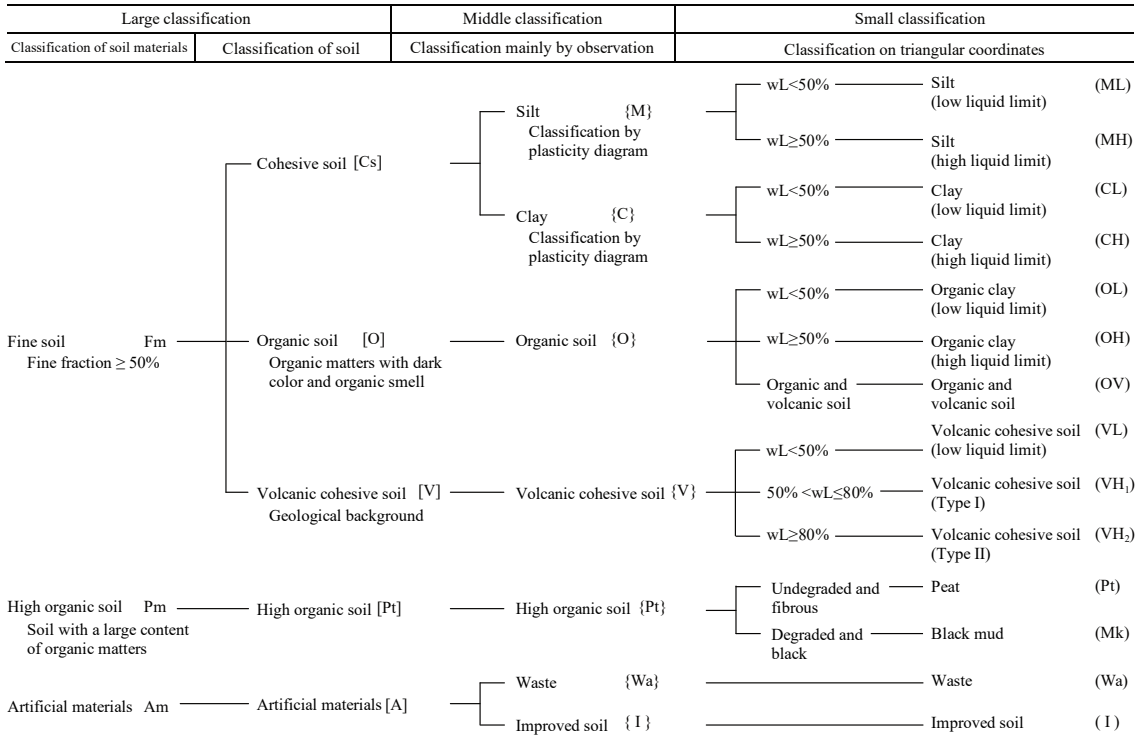


Note: content rates are mass percentages with respect to soil materials

Fig. 2.2.6 Engineering Classification of Subsoil Materials (Large Classification) (JGS 0051)



(a) Engineering Classification System of Coarse Soil



(b) Engineering Classification System of Major Fine Soil

Fig. 2.2.7 Engineering Classification Method for Subsoil Material (JGS 0051)

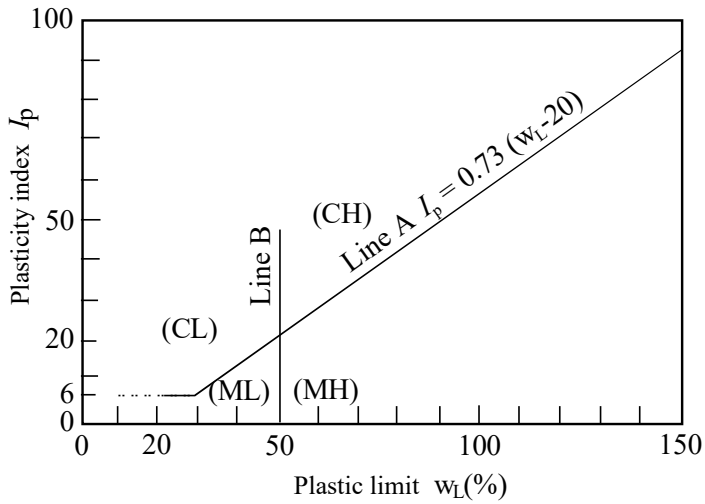


Fig. 2.2.8 Plasticity Chart Used for the Classification of Fine Soil (JGS 0051)

(4) Classification by grain size

Grain sizes have particularly close relationships with the engineering properties of coarse soil. The classification by grain size is to classify several types of soil particles constituting soil in terms of the content ratios of certain ranges of grain diameters. Generally, the content ratios of fine particles with grain sizes smaller than 75 μm are measured through the sedimentation analysis method, using hydrometers. Coarse particles with grain sizes of 75 μm or more are measured through the sieving analysis method. These measuring methods are specified in the **Test Method for Particle Size Distribution of Soils (JIS A 1204)**. The analysis results are expressed as grain diameter accumulation curves to obtain uniformity coefficients U_c, coefficients of curvature U'_c, and effective grain diameter D_{10}.

The uniformity coefficient U_c is the numerical expression of grain diameter distribution of sandy soil as shown in **equation (2.2.6)**.

$$U_c = D_{60} / D_{10} \qquad \textbf{(2.2.6)}$$

Where

U_c : an uniformity coefficient;

D_{60} : a grain diameter corresponding to 60% by mass passing (counting from smaller diameters) (mm); and

D_{10} : a grain diameter corresponding to 10% by mass passing (mm).

Large uniformity coefficients U_c mean that soil has wide and uniform grain size distribution, which is called "well graded." In contrast, small uniformity coefficients U_c mean that soil has narrow grain size distribution with large concentrations of similarly sized particles, which is called "poorly graded." In the Japanese Unified Soil Classification System, coarse soil with fine-grain contents lower than 5% is further classified into "well graded soil" and "poorly graded soil" in terms of U_c as follows.

Well graded soil: $10 \leq U_c$

poorly graded soil with classified grain distribution: $U_c < 10$

(5) Role of cohesive soil particles

Cohesive soil characteristics are the result of the activations of colloid particles with diameters of 1 to 2 μm or smaller and completely different from those of soil with larger particle diameters in terms of mineral textures and physicochemical properties. Cohesive soil has unique engineering properties obtained through the sedimentary environment formed together with coexisting materials (such as amorphous colloid and salts). Thus, when examining soil's engineering characteristics, it is necessary to focus on the qualitative and quantitative analyses with respect not to cohesive soil particles with diameters of 5 μm or smaller, but to colloid particles with diameters of 1 μm or smaller and coexisting materials. However, commonly used existing grain size tests have accuracy

problems for particles with diameters of 5 μm or smaller in terms of the appropriateness of dispersing particles and stabilizing dispersed particles, appropriateness of applying the Stokes' law to the sedimentation method, and errors associated with the hydrometer method.

In contrast, because the consistency tests show particle activation, including the influences of coexisting matters, for the purpose of the engineering classification of fine particles, it is rational to use a classification method based on the results of the consistency test, which can be carried out more easily than the grain size test.

(6) Relationship between consistency and engineering properties of cohesive soil

Consistency is one of the physical properties with which the engineering properties of cohesive soil have a close relationship. Consistency is generally expressed in the forms of liquid limit w_L or plastic limit w_p measured through the **Test Method for Liquid Limit and Plastic Limit of Soils (JIS A 1205)**; and plasticity index I_p (= w_L-w_p) or liquidity index I_L (= w_n-w_p)/I_p which can be obtained from those limits and natural water content w_n. For further information on the relationship between the engineering properties and consistency of cohesive soil, reference can be made to the literature by Ogawa and Matsumoto[3)].

2.2.3 Coefficient of Permeability of Soil

(1) When the seepage flow in completely saturated ground can be considered as a steady laminar flow, the coefficient of permeability can be calculated in accordance with the Darcy's law.

(2) The coefficient of permeability k is calculated by **equation (2.2.7)** in accordance with the Darcy's law with the measured values of a cross-sectional area of soil A, a hydraulic gradient i and the volume of seepage flow in unit time q.

$$k = \frac{q}{iA} \tag{2.2.7}$$

where

k : the coefficient of permeability (cm/s) or (m/s);

q : the volume of water flow in a soil layer in unit time (cm^3/s) or (m^3/s);

i : a hydraulic gradient, $i = \dfrac{h}{L}$

h : a head loss (cm) or (m);

L : the length of a seepage path (cm) or (m); and

A : a cross-sectional area (cm^2) or (m^2).

The measurement methods for determining k include a laboratory permeability test of undisturbed soil specimens sampled from sites and an in-situ permeability test.

(3) Scope of application of the Darcy's law

Darcy's law needs to be applied to laminar flows which are defined by the critical Reynolds numbers R_e. That is, when effective diameters D_{10} are used, laminar flows need to have the critical Reynolds numbers in the range of $3 < R_e < 10$. This range is considered to correspond to the types of soil with grain sizes smaller than those of medium and coarse sand. It is expressed by the effective diameter D_{10} as shown in **Table 2.2.2**.[4)]

Table 2.2.2 Scope of Application of the Darcy's Law (D_{10}: Grain diameter corresponding to 10% by mass passing)

Material	Gravel			Sand gravel	Sand		Sandy soil, silt
	Large	Coarse	Medium		Coarse	Fine	
D_{10} (mm)	75	26.5	9.5	2.0	0.6	0.25	0.075–0.02
State of seepage flow	Normally disturbed flow in reality			Laminar flow when i < 0.2 to 0.3 for loose sand, and i < 0.3 to 0.5 for dense sand			Normally laminar flow with general values of i

(4) Laboratory permeability tests

There are two types of laboratory permeability tests: the constant head permeability test and the falling head permeability test. These tests can be conducted in accordance with the **Test Methods for Permeability of Saturated Soils (JIS A 1218)**. When conducting the laboratory permeability tests with undisturbed specimen's sampled in-situ, it is necessary to clarify the installation directions of specimens before measuring because natural soil has been deposited in layers, to some extent, and its permeability differs depending on the specimens' installation directions. In principle, undisturbed specimens are used for laboratory permeability tests but when it is difficult to obtain them (in the case of soil with low fine fraction contents), laboratory permeability tests can be conducted by necessity using disturbed specimens with different densities to obtain the permeability corresponding to the appropriate density. When using disturbed specimens which are subjected to compaction in a laboratory for adjusting density, there may be cases where the measured permeability largely differs from in-situ permeability because the in-situ skeleton structures of soil cannot be restored in these specimens.

(5) In-situ permeability test

Because laboratory test conditions are different from natural ones, in-situ tests are conducted when needed. The in-situ permeability test is conducted in a manner that installs one or two observation wells and measures ground water level fluctuations with water injected into or pumped out of one of the wells. The in-situ permeability test method can be conducted with reference to the **Method for Determining an Aquifer's Hydraulic Properties in Single Borehole (JGS 1314)**.

(6) Approximate values of the coefficients of permeability

Hazen figured out that coefficients of permeability of sand are correlated with effective diameters, and the coefficients of permeability k can be calculated by **equation (2.2.8)** in the case of the sand with relatively uniform density using the uniformity coefficient $U_c < 5$ and the effective diameter $D_{10} = 0.1$ to 0.3 mm.[5)]

$$k = CD_{10}^{\ 2} \qquad \textbf{(2.2.8)}$$

where

k : a coefficient of permeability (cm/s) or (m/s);

C : a coefficient ($C = 100$ (1/cm/s)) or ($C = 10000$ (1/m/s); and

D_{10} : a grain size called effective diameter corresponding to 10% by mass passing (cm)

Terzaghi argued that **equation (2.2.8)** can also be applied to cohesive soils by using $C \fallingdotseq 2$ (or 200) but it is necessary to carefully determine such applicability. The approximate values of the coefficients of permeability are listed in **Table 2.2.3**.[5)]

Table 2.2.3 Approximate Values of the Coefficient of Permeability[5)]

Type of soil layer	Sand layer	Silt layer	Cohesive layer
Coefficient of permeability	10^{-2} cm/s (10^{-4} m/s)	10^{-5} cm/s (10^{-7} m/s)	10^{-7} cm/s (10^{-9} m/s)

2.3 Mechanical Properties of Soil

2.3.1 Elastic Constants

(1) When analyzing soil as elastic materials, the elastic constants shall be appropriately set taking into consideration the fact that soil is material showing nonlinearity.

(2) The elastic constants generally used when analyzing soil as elastic materials are the deformation moduli and Poisson's ratios. In addition to stress dependency, the deformation moduli depend strongly on strain. Therefore, when conducting elastic analyses of ground, it is necessary to appropriately set elastic moduli taking strain levels of object ground into consideration.

(3) Strain dependency of deformation moduli

The stress-strain relationship of soil shows strong nonlinearity. **Fig. 2.3.1** shows one example of the relationship between deformation moduli and strain of the Pleistocene cohesive soil in Osaka Bay, obtained through a monotonous loading triaxial test.[6] In the figure, NCs are the triaxial test results of specimens which are subjected to anisotropic consolidation to in-situ effective overburden pressure with K_{0NC} kept at 0.5. OCs are the triaxial test results of specimens which are subjected to consolidation to consolidation yield stress with K_{0NC} kept at 0.5, followed by unloading to the level of OCR of 1.2 with K_{0OC} kept at $K_{0NC}OCR^{\sin\phi}$ to reproduce consolidation history. Also, K_{0NC} is the coefficient of earth pressure at rest in a normal consolidation region, ϕ is the angle of shear resistance, and OCR is an overconsolidation ratio. As can be seen in the figure, the deformation moduli are almost constant at their maximum values in the range where strain levels are 10^{-5} or less (0.001% or less). The maximum value E_{max} in this case corresponds to the value measured in dynamic tests, such as the elastic wave exploration, and is called a dynamic elastic modulus. As the strain level increases, the shear modulus of elasticity decreases. The secant modulus E_{50}, obtained from conventional unconfined compression tests or triaxial compression tests, is considered the deformation moduli when the strain is in the order of 10^{-3} (0.1%). When conducting elastic analyses of soil, it is necessary to set elastic constants appropriate for the strain level of object ground.

(4) Measurement of deformation moduli through in-situ tests

In many cases, the deformation moduli are obtained through the following in-situ tests:

① In-situ elastic wave exploration;
② PS logging;
③ Surface wave exploration;
④ Seismic cone test;
⑤ Loading test inside borehole;
⑥ Plate loading test; and
⑦ CBR test.

When using deformation moduli obtained through these tests, it is necessary to give due consideration to the fact that respective in-situ tests are conducted with different strain levels of ground.

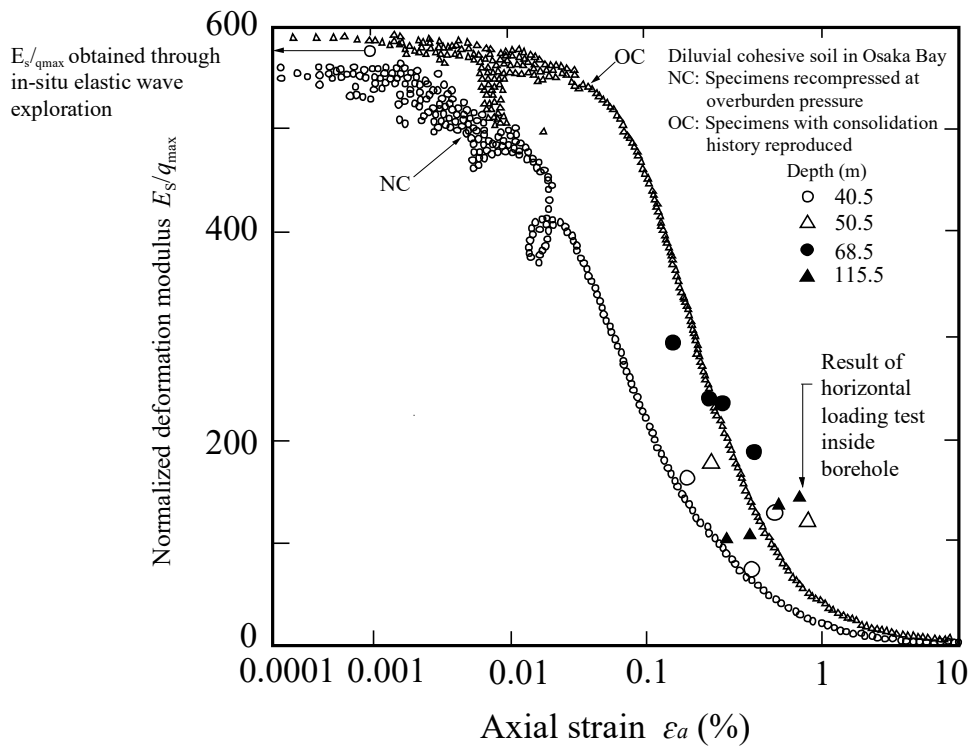


Fig. 2.3.1 Relationship between Strain and Elastic Moduli (Diluvial Cohesive Soil in Osaka Bay)[6]

(5) Relationship between undrained shear strength and deformation moduli

For cohesive soil, the approximate values of initial tangent elastic moduli E_i, and secant elastic moduli E_{50} can be calculated by **equations (2.3.1)** and **(2.3.2)** respectively.[7)]

$$E_i = 210c_u \tag{2.3.1}$$

$$E_{50} = 180c_u \tag{2.3.2}$$

where

E_i : an initial tangent elastic modulus (kN/m^2)

E_{50} : a secant elastic modulus (kN/m^2)

c_u : undrained shear strength (kN/m^2)

The above relationship between E_i and c_u is applicable only to the high-plasticity marine cohesive soil with well-developed structures.

(6) Poisson's ratio

Despite several proposals, there have been no established test methods for obtaining Poisson's ratio. In general, Poisson's ratio is set at around $\nu = 1/2$ for undrained saturated soil and $\nu = 1/3$ to $1/2$ for other soil.

2.3.2 Compression and Consolidation Characteristics

(1) Soil compression characteristics and coefficients for estimating ground settlement due to consolidation can be calculated from the values obtained based on the **Test Method for One-dimensional Consolidation Properties of Soils Using Incremental Loading (JIS A 1217)**.

(2) Compression is one of the soil characteristics which causes ground settlement, with soil particle structures compressed when the soil is subjected to one-dimensional loading. When voids among soil particles are saturated with pore water, the pore water needs to be drained, which will compress the soil particles due to loading. Highly permeable sandy soil shows compression immediately after loading with pore water smoothly drained. In contrast, cohesive soil ground has significantly low permeability, so compressive settlement occurs very slowly because it takes long time until the pore water is drained. The phenomenon in which cohesive soil ground shows gradual compressive settlement over time is called consolidation.

The consolidation characteristics of soil are used not only to calculate settlement due to loading but also for estimating the increase in shear strength in soil improvement work.

(3) Calculation of final consolidation settlement

When plotting consolidation pressure and void ratios at the end of consolidation with the consolidation pressure (applied for 24 hours) on a semi–logarithmic graph, a compression curve (so called e–log$p$ curve) is obtained as illustrated in **Fig. 2.3.2**. The almost linear section a-b-c in the e–log$p$ curve represents a loading process, and the consolidation states of soil on the linear section a-b-c are called normal consolidation states. When soil is subjected to unloading at point b in the e–log$p$ curve and reaches equilibrium under reduced pressure, the relationship between the void ratios and stress of the soil follows the transition route b-d in the e–log$p$ curve and, when the soil is subjected to reloading, the relationship follows the transition route d-b. The states expressed by the transition routes b-d and d-b are called the overconsolidation state. When conducting consolidation tests, the relationship between stress and void ratios of soil specimens shows the transition route d-b-c because they are initially in overconsolidation states. Then, a boundary point b can be identified from the elastic deformation section, represented by the transition route d-b, and the plastic deformation section, represented by the transition route b-c. The pressure corresponding to the point b is called consolidation yield stress p_c.

The relationship between void ratio e and pressure p in the linear section a-b-c (normal consolidation region) in **Fig. 2.3.2** is expressed by **equation (2.3.3)**.

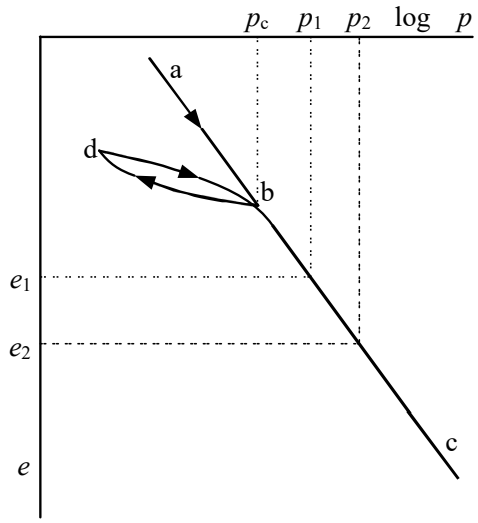


Fig. 2.3.2 *e*–log*p* Relationship in Consolidation

$$e_2 = e_1 - C_c \log_{10} \frac{p_2}{p_1} \tag{2.3.3}$$

Where, C_c is a non-dimensional parameter showing the inclination of the linear section a-b-c and is defined as the decrement of the void ratio in one logarithmic cycle.

The final settlement resulting from the consolidation loads can be obtained through three methods which respectively use: *e*–log*p* curves; C_c; and the coefficients of volume compressibility m_v.

The decrease in void ratio Δe, when the pressure increases from in-situ effective overburden pressure σ'_{v0} to ($\sigma'_{v0} + \Delta p$), can be obtained by directly reading the *e*–log*p* relationship available through consolidation tests. Otherwise, when allowing the consolidation settlement to be overestimated (the settlement in the overconsolidation region evaluated in the same manner as the settlement of normal consolidation region so as to be on the safe side), the final settlement may be calculated by **equation (2.3.4)** derived from **equation (2.3.3)**.

$$\Delta e = e_{\sigma'_{v0}} - e_{\sigma'_{v0}+\Delta p} = C_c \log_{10} \frac{\sigma'_{v0} + \Delta p}{\sigma'_{v0}} \tag{2.3.4}$$

When calculating the final settlement S through the method using the *e*–log*p* curves, the value of Δe either read directly from the *e*–log*p* curve or calculated by **equation (2.3.4)** is plugged into **equation (2.3.5)**.

$$S = h \frac{\Delta e}{1 + e_0} \tag{2.3.5}$$

where

h : the thickness of a stratum.

When calculating the final settlement S through the method using C_c, **equation (2.3.6)** can be used.

$$S = h \frac{C_c}{1 + e_0} \log_{10} \frac{\sigma'_{v0} + \Delta p}{\sigma'_{v0}} \tag{2.3.6}$$

Equation (2.3.6) corresponds to equation obtained by substituting **equation (2.3.4)** into **equation (2.3.5)**.

The coefficient of volume compressibility m_v can be used to estimate settlement because the amount of compression by the increment of a load is proportional to m_v. However, because the method for calculating the final settlement using m_v applies linear approximation to soil which has strong nonlinearity, the method is effective only for small increments in consolidation pressure Δp so that m_v can be assumed to be constant. In the case of the method using m_v, the final settlement S can be calculated by **equation (2.3.7)**.

$$S = m_v \Delta p h \tag{2.3.7}$$

where

m_v : the coefficient of volume compressibility when the consolidation load is （$\sqrt{\sigma'_{v0}(\sigma'_{v0}+\Delta p)}$）

Generally, the value of m_v during consolidation decreases with the increase of effective overburden pressure p. Under the normal consolidation state, the relationship between p and m_v plotted on a double logarithmic graph is almost linear. The value of m_v to be used when calculating settlement by **equation (2.3.7)** shall be the mean value in the period when the effective overburden pressure of ground is changed from σ'_{v0} to (σ'_{v0} + Δp). Normally, the value of m_v corresponding to the geometric mean of the effective overburden pressure . （$\sqrt{\sigma'_{v0}(\sigma'_{v0}+\Delta p)}$）

(4) Settlement rates

The following section describes the settlement rate analysis method based on Terzaghi's one-dimensional consolidation theory, one of the classical consolidation theories. When saturated cohesive soil is subjected to an increment in pressure Δp under undrained condition and excess pore water pressure Δu, equivalent to the increment in pressure Δp, is generated in the soil. The excess pore water pressure gradually dissipates as consolidation progresses with the gradual increase in the effective stress σ' in a manner that balances the sum of the increments of excess pore water pressure Δu and effective stress $\Delta\sigma'$ with the increment in pressure Δp as shown in **equation (2.3.8)**.

$$\Delta p = \Delta\sigma' + \Delta u \tag{2.3.8}$$

In the case of a cohesive stratum having a thickness of $h = (2H)$, placed in between upper and lower sand strata with large permeability, the distribution of $\Delta\sigma'$ and Δu in the depth direction when the cohesive stratum is subjected to the increment in consolidation pressure Δp is as shown in **Fig. 2.3.3**. That is, the state of the sand stratum when consolidation begins (t = 0) can be expressed by the line DC because of $\Delta u = \Delta p$ and $\Delta\sigma' = 0$, and when consolidation is completed, the state can be expressed by the line AB because of $\Delta u = 0$ and $\Delta\sigma' = \Delta p$. The curved line AEB shows the distribution of the excess pore water pressure after the lapse of t_1 since consolidation began. As can be seen in the figure, the portions in the cohesive soil stratum away from drainage layers have relatively slow consolidation rates.

The ratio of the increment in effective stress to the increment in consolidation pressure ($\Delta\sigma'/\Delta p$) at a certain depth z is called a degree of consolidation U_z and the average of the degree of consolidation at respective depths in the entire stratum is called an average degree of consolidation. In **Fig. 2.3.3**, the average consolidation rate can be expressed by the ratio of the area of AEBCD to that of ABCD.

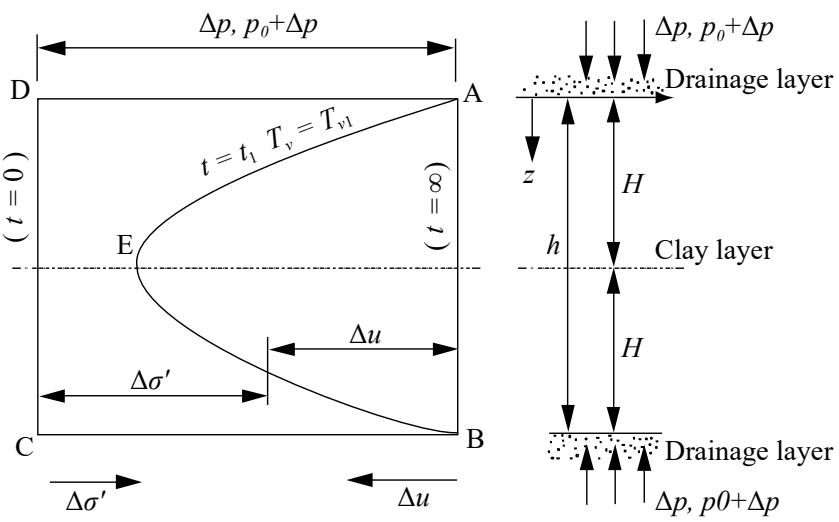


Fig. 2.3.3 Distribution of Pore Water Pressure in the Depth Direction

Consolidation progress in an entire cohesive soil stratum can be expressed by the magnitude of the average degree of consolidation U. A time factor T_v which is a non-dimensional quantity is used as the unit showing time. The relationship between the time factor T_v and actual time t can be expressed by **equation (2.3.9)**.

$$T_v = \frac{c_v t}{H^{*2}} \tag{2.3.9}$$

where

T_v : a time factor;

c_v : the coefficient of consolidation (cm^2/day) or (m^2/s);

t : time after the consolidation is started (day) or (s); and

H^* : the maximum flow distance of pore water (maximum drainage distance) (cm) or (m).

The consolidation coefficient c_v is a soil parameter showing the consolidation progress rate. In the case of a cohesive soil stratum with a thickness of $2H$ placed in between upper and lower drainage strata (called double drainage), H can be used as H^* in **equation (2.3.9)**. In the case of the cohesive soil stratum with only one drainage stratum placed one above or the other (called single drainage), $2H$ can be used as H^* in **equation (2.3.9)**. As indicated by the consolidation isochronous curve in **Fig. 2.3.4**, the degree of consolidation at each depth transitions along with the time factor. **Fig. 2.3.5** shows the relationship between the average degree of consolidation and time factors.

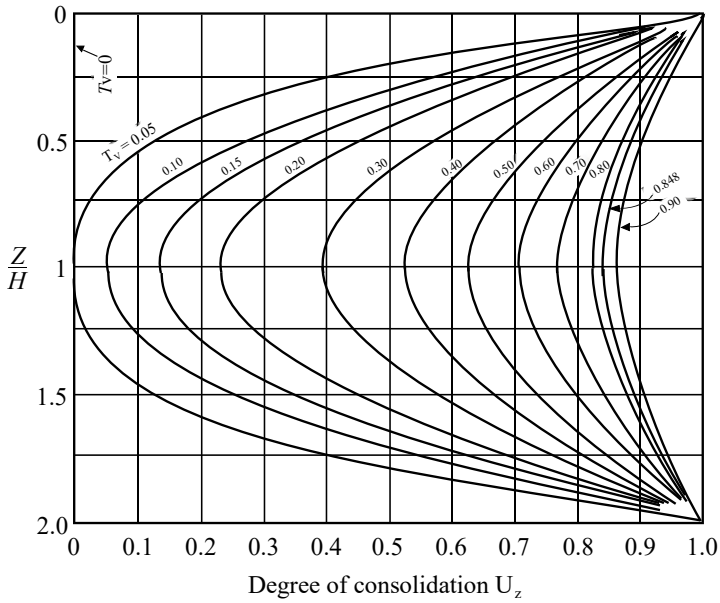


Fig. 2.3.4 Consolidation Isochronous Curve

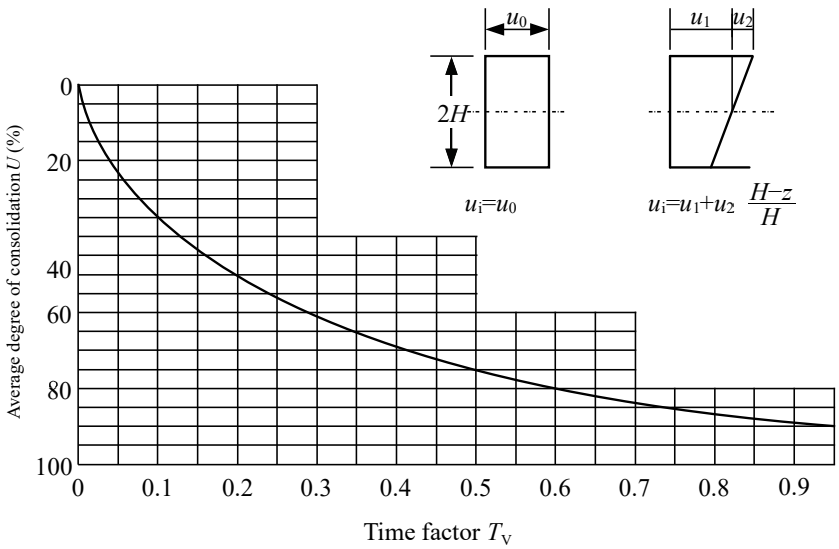


Fig. 2.3.5 Relationship between the Average Degree of Consolidation and Time Factors

Although Terzaghi's consolidation theory has been widely practiced as a simple analysis method, the theory cannot take into account the following four factors:

① the influence of the self-weight of cohesive soil layers;

② the inhomogeneity of the coefficients of consolidation and the compaction property of cohesive soil strata;

③ the secondary consolidation attributable to the viscosity of cohesive soil skeletons (not attributable to the dissipation of excess pore water pressure); and

④ consolidation phenomena in multi- dimension, namely two and three dimensions.

To analyze consolidation settlement with improved accuracy by considering the above factors, it is necessary to implement finite element analysis methods using an elasto-viscoplastic constitutive equation, which has also been widely practiced recently.

(5) Primary and secondary consolidation

According to the relationship between degree of consolidation, measured in consolidation tests, and time (corresponding to the relationship between settlement and time) which can be schematically shown in **Fig. 2.3.6**, the test results do not agree with a theoretical curve in the later consolidation stage. In the relationship between settlement and time, the process where U is up to 100% and test results almost agree with the consolidation theory. This is categorized as the primary consolidation. The process where U is larger than 100% and the test results do not agree with the consolidation theory is categorized as the secondary consolidation. Secondary consolidation is considered to be the creep phenomenon of soil and tends to have a linear relationship between settlement and the logarithm of elapsed time.

In the performance verification of port facilities, there are many cases where the consolidation pressure due to loading reaches several times the consolidation yield stress. In such conditions, the settlement due to the primary consolidation is significantly large, and settlement due to the secondary consolidation is relatively small, thereby allowing most performance verification to be carried out without special consideration to the settlement due to the secondary consolidation. Also, when the settlement is large, there may be cases where no secondary consolidation is apparently observed; the effects of the secondary consolidation are canceled out by the increased effects of buoyancy with the progress of settlement. However, performance verification shall be carried out considering the effects of the secondary consolidation in the following cases:

① when the effect of settlement over time on facilities after their construction is large; and

② when the consolidation pressure due to loading does not significantly exceed the consolidation yield stress of original ground and, therefore, the contribution of the secondary consolidation settlement to the entire settlement is not negligible, as is the case with the deep the pleistocene cohesive soil ground.

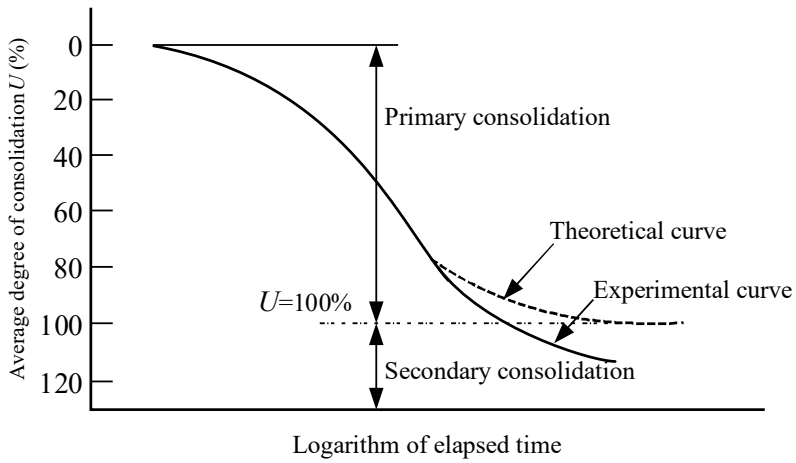


Fig. 2.3.6 Primary and Secondary Consolidation

The long-term consolidation settlement which occurs as residual settlement causes serious issues in design and maintenance when constructing a huge artificial island in a deep sea area or on the cohesive soil seabed. In such a case, a widely used method for estimating final settlement is to add secondary consolidation settlement calculated by using the secondary consolidation coefficient C_α to the completed primary consolidation settlement. However, this method has problems in that: the point to start the calculation of the secondary consolidation settlement is unclear; and the theoretical assumption of a continuous linear relationship between settlement and logarithm of lapsed time in the secondary consolidation cannot consider the actual relationship, which shows a convex-downward curve with the inclination getting gentler as time advances.

According to research on improving the prediction accuracy of long-term consolidation settlement[8)], an isotach concept[9)] focusing on the strain rate dependency of consolidation yield stress is considered an effective approach to continuously deal with the primary and secondary consolidation.

Focusing on viscoplastic strain ε_{vp} obtained by subtracting elastic strain ε_e from total strain ε, the method based on the isotach concept uses **equation (2.3.10)** to model the strain rate dependency of consolidation yield stress p'_c.

$$\ln \frac{p'_c - p'_{cL}}{p'_{cL}} = c_1 + c_2 \ln \dot{\varepsilon}_{vp} \qquad \textbf{(2.3.10)}$$

In Equation, ε_{vp} is a viscoplastic strain rate, p'_{cL}, c_1, and c_2 are constants called isotach parameters, and p'_c converges on the lower limit p'_{cL} when a strain rate reaches an infinitesimal value.

Fig. 2.3.7 shows the relationship of **equation (2.3.10)** normalized with the yield stress p'_{c0} corresponding to the strain rate of $1.0 \times 10^{-7}\mathrm{s}^{-1}$ (equivalent to the strain rate after the lapse time of 24 hours in an incremental loading consolidation test). The isotach parameters of p'_{cL}/p'_{c0} and c_1 are set 0.7 and 0.935, respectively, to be applicable to a variety of cohesive soil. The other isotach parameter c_2 can be automatically determined by the following equation.

$$c_2 = \frac{\ln \frac{p'_c - p'_{cL}}{p'_{cL}} - c_1}{\ln \dot{\varepsilon}_{vp}} = \frac{\ln \frac{1-0.7}{0.7} - c_1}{\ln 1.0 \times 10^{-7}} \qquad \textbf{(2.3.11)}$$

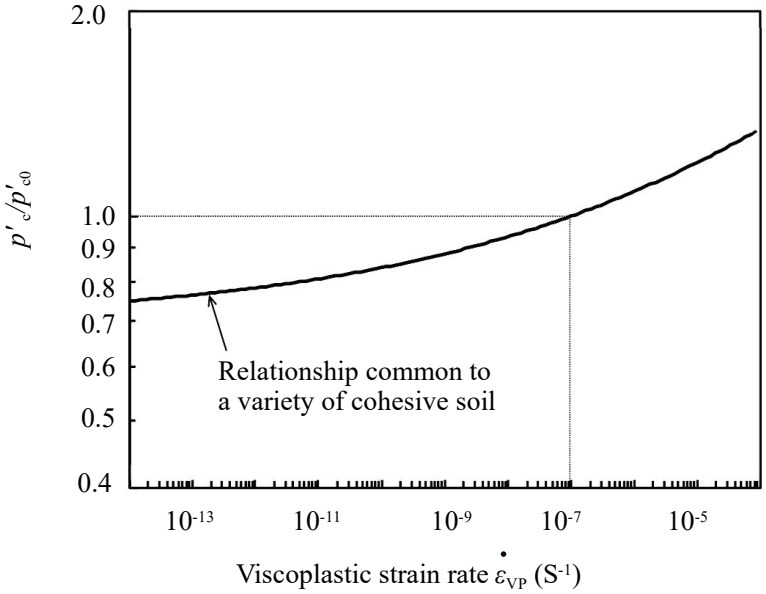


Fig. 2.3.7 Strain Rate Dependency of Consolidation Yield Stress[8)]

The consolidation settlement prediction method, taking a strain rate into consideration, can be explained using **Fig. 2.3.8** assuming that consolidation pressure is increased from the effective overburden pressure σ'_{v0} to the in-situ consolidation pressure p'_1 after loading. The compression curve passing through the point D in the figure corresponds to that obtained through incremental loading consolidation tests. In the laboratory long-term consolidation tests, strain rates show the transit route of A-C-D-E. In contrast, in-situ strain rates show the transit route of A-B-E-F.

By additionally predicting an in-situ strain rate or setting an allowable long-term strain rate for the required performance of facilities, the settlement which corresponds to the strain rate, namely the strain increment ($\Delta\varepsilon_{Field}$) as shown by the section D-E in the figure, can be calculated using **equation (2.3.12)**. Furthermore, consolidation has a possibility to reach the point F in the figure and then the maximum possible settlement corresponding to the strain increment ($\Delta\varepsilon_{ult}$) as shown by the section D-F can be calculated by **equation (2.3.13)**.

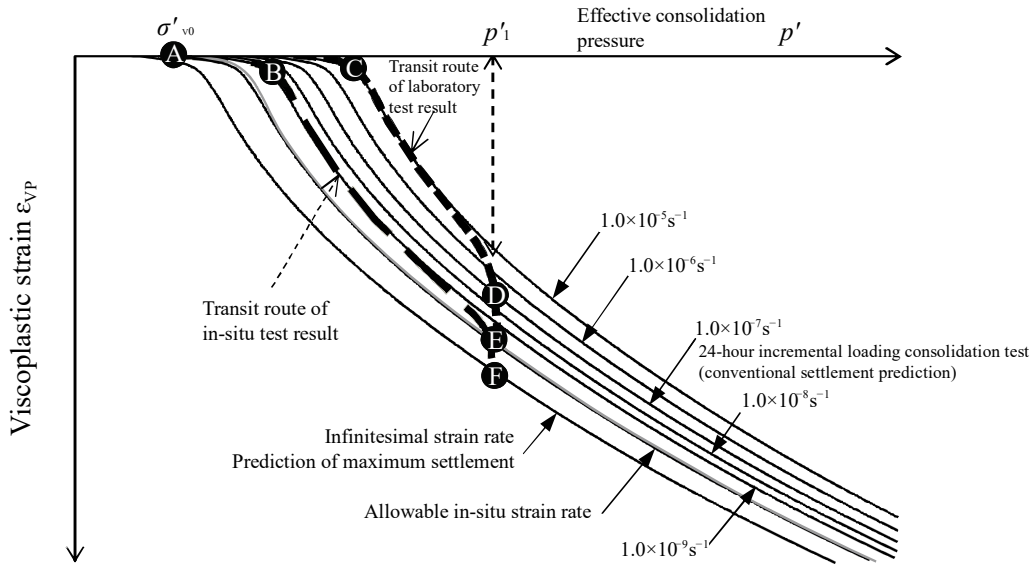


Fig. 2.3.8 Relationship between laboratory test results and in-situ settlement behavior[8)]

$$\Delta\varepsilon_{\mathrm{Field}} = \frac{C_{\mathrm{c}}}{1+e_0}\log\left[\frac{p'_{\mathrm{c0}}}{p'_{\mathrm{cL}}}\left\{\frac{1}{1+\exp\left(c_1 + c_2 \ln\dot{\varepsilon}_{\mathrm{Field}}\right)}\right\}\right] \tag{2.3.12}$$

$$\Delta\varepsilon_{\mathrm{ult}} = \frac{C_{\mathrm{c}}}{1+e_0}\log\frac{p'_{\mathrm{c0}}}{p'_{\mathrm{cL}}} \tag{2.3.13}$$

The relationships expressed by **equations (2.3.12)** and **(2.3.13)** are graphically illustrated in **Fig. 2.3.9** and **2.3.10** respectively.

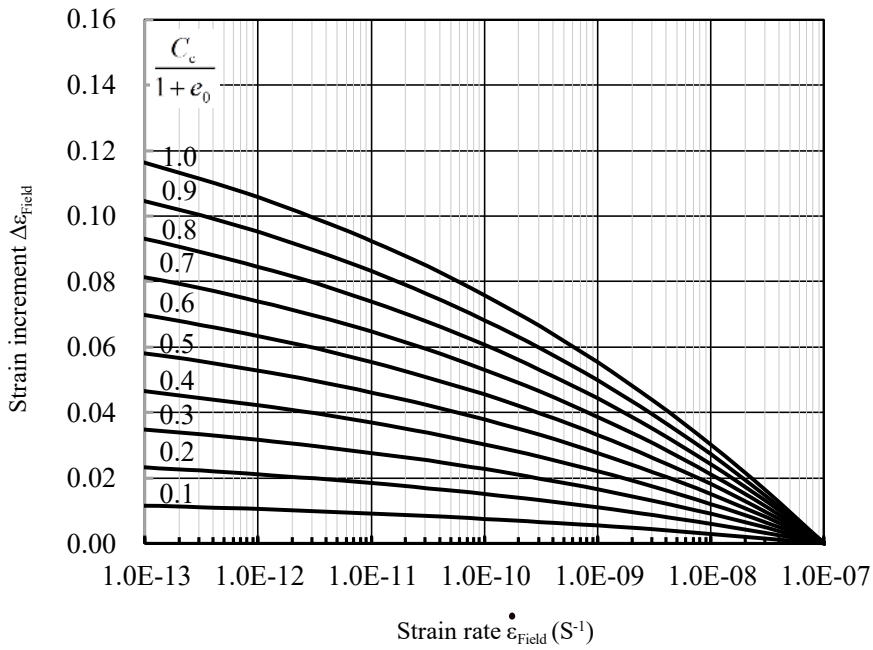


Fig. 2.3.9 Strain Increment Corresponding to In-situ Strain Rate
Strain Increment Additional to Strain Predicted from the Results of Incremental Loading Consolidation Test Result[8)]

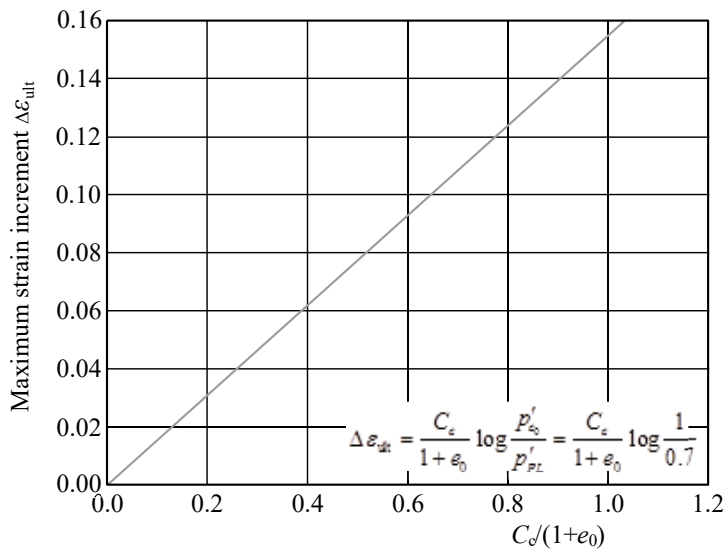


Fig. 2.3.10 Relationship Diagram between Maximum Strain Increment after Strain Rate reaches at $1 \times 10^{-7}s^{-1}$ and $C_c/(1 + e_0)$[8)]

(6) Consolidation settlement of very soft cohesive soil

In landfilling with dredged soil or disposing of sludge, a problem of consolidation settlement of extremely soft ground mey be encountered. The consolidation theories applicable to these problems include Mikasa's[10] consolidation theory, which takes into account the effects of the changes in layer thicknesses during consolidation and the self-weight of cohesive soil strata. Closed solution of this theory on settlement and settlement rates cannot be found, the solution of which are obtained by a finite differential method.

When the reduced thickness of a stratum due to settlement is too large compared to the original thickness to ignore, normal consolidation settlement calculation methods are susceptible to large errors. For example, if the stratum thickness is reduced by 10 to 50%, the difference between the normal calculation method and the calculation method considering the effect of the change in stratum thickness is in the range of 3 to 30%. Also, the effect of self-weight on settlement is maximized when dredged soil is left on a landfill site. In such a case, the effect of self-weight is relatively reduced with the increased surcharge pressure. For example, the effect of self-weight on very soft strata becomes negligibly small when the surcharge pressure on the surface of a landfill site is about twice or more the self-weight of ground at the central depth of the very soft stratum.

A consolidation test method[11] in which specimens are subjected to continuous displacement through constant strain rate loading has been standardized **(JIS A 1227)** to estimate the consolidation properties of very soft cohesive soil. Because of the ability to obtain continuous e–log$p$ curves, the consolidation test method has a wide field of application and is effective, not only for testing consolidation properties of very soft cohesive soil, but also obtaining consolidation yield stress of cohesive soil, including that with a large aging effect, which shows abrupt settlement after reaching the consolidation yield. However, the e–log$p$ curves are strongly affected by strain rates. The e–log$p$ curves obtainable through the above consolidation test method, which generally uses high loading rates, are normally shifted toward a large consolidation pressure side compared to the e–log$p$ curves obtainable through the **Incremental Loading Consolidation Test Specified (JIS A 1217)**. Therefore, it is preferable to set appropriate consolidation yield stress while being aware that the consolidation yield stress based on the consolidation test using a constant rate of strain tends to become larger, and combining the incremental loading consolidation test results.

(7) Correlation between compression and consolidation coefficients and physical properties

Consolidation tests require the longest testing time among all the soil tests. It is advantageous if consolidation properties become available with disturbed specimens and obtainable through physical tests, which can speedily produce test results through relatively simple operation. Skempton proposed **equation (2.3.14)**, which expresses a relationship between the compression index C_c and the liquid limit w_L.

$$C_c = 0.009(w_L - 10) \qquad \textbf{(2.3.14)}$$

Equation (2.3.14) is applicable to cohesive soil re-molded and re-consolidated in laboratories or the young cohesive soil ground immediately after being reclaimed. However, it may underestimate the compression characteristics of naturally deposited cohesive soil.

Fig. 2.3.11 shows the results of surveying the similar relationship using marine cohesive soil in Japan. **Equation (2.3.15)** was also proposed as another relationship between C_c and w_L (%) in the figure.

$$C_c = 0.015(w_L - 19) \qquad \textbf{(2.3.15)}$$

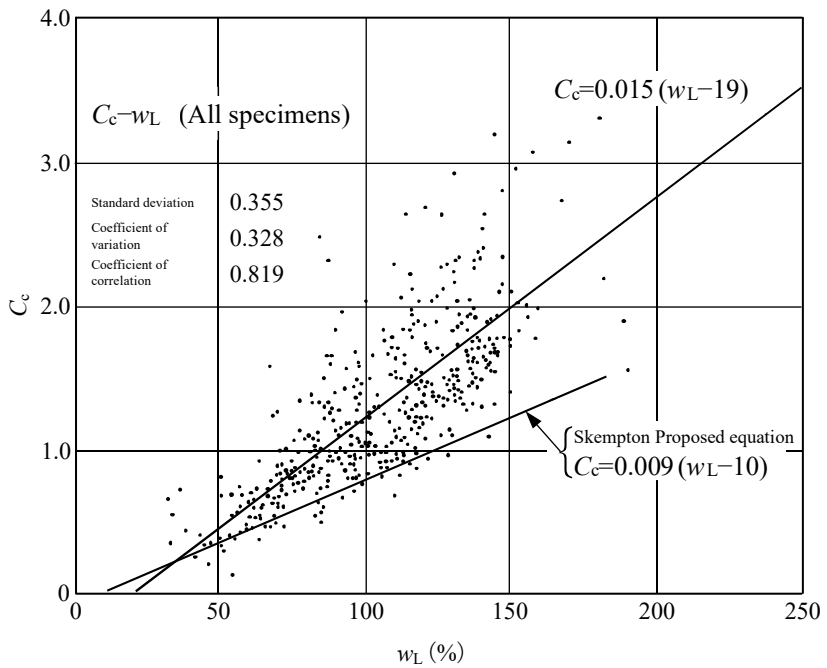


Fig. 2.3.11 Correlation between Compression Index and Liquid Limit[12)]

The reason why natural cohesive soil ground has larger compression index values than young cohesive soil ground is because natural cohesive soil structures, which have formed through the aging effects such as cementation in the sedimentation process over a long period of time, show high compressibility when destroyed with consolidation pressure exceeding consolidation yield stress.

Fig. 2.3.12 and **Fig. 2.3.13** show the relationship among the coefficients of consolidation c_v, coefficients of volume compressibility m_v, and liquid limits w_L which can be obtained through consolidation tests. The values of c_v are those measured when they are almost constant and consolidation processes are already in normal consolidation states. Because m_v linearly changes with respect to the increments in consolidation pressure p on double logarithmic graphs, **Fig. 2.3.13** shows cases when the values of p are 100 kN/m^2 and 1000 kN/m^2 respectively. Although respective relationships show large dispersions, they can be used to obtain approximate values of these coefficients from liquid limits.

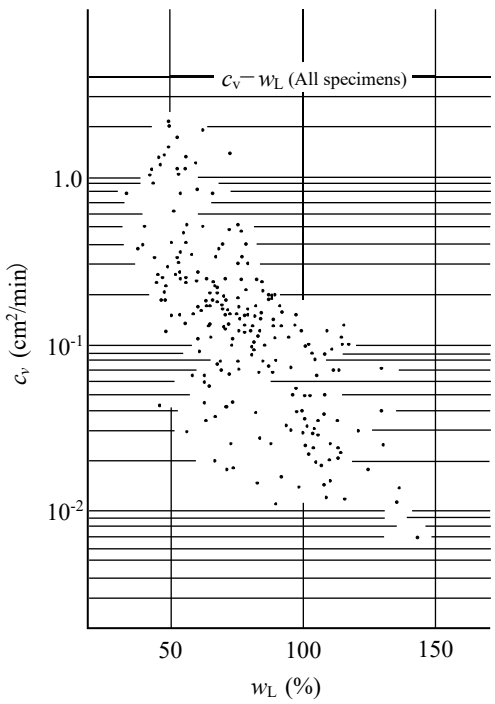
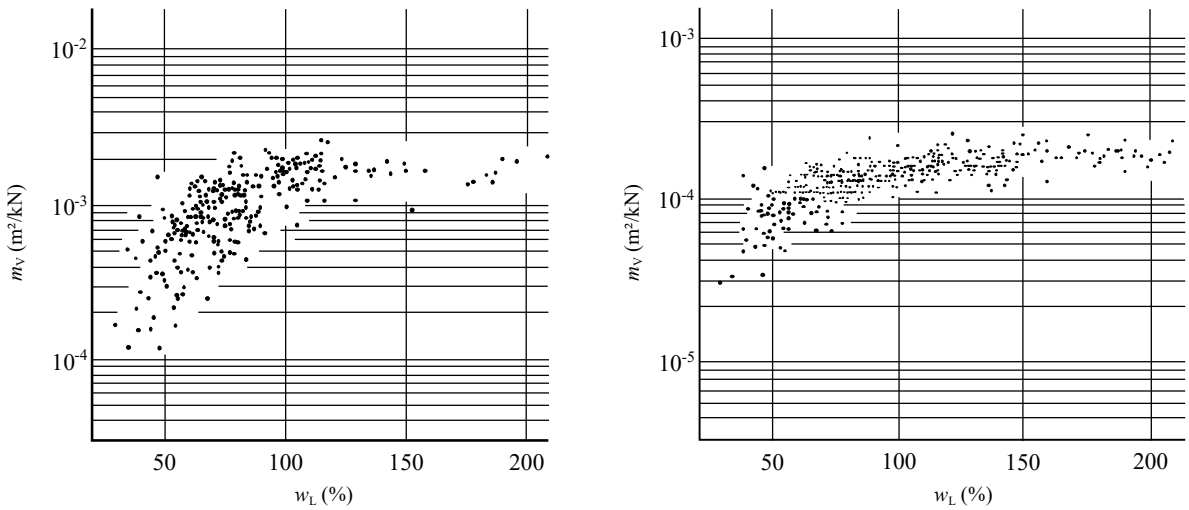


Fig. 2.3.12 Relationship between Coefficient of Consolidation c_v and Liquid Limit[12)]



(a) When p=100 kN/m²

(b) When p=1000 kN/m²

Fig. 2.3.13 Relationship between Coefficients of Volume Compressibility and Liquid Limit[12)]

2.3.3 Shear Characteristics

(1) The shear strength of soil is generally set classifying soil into sandy soil and cohesive soil. Also, the shear strength of sandy and cohesive soil is set under drained and undrained conditions, respectively.

(2) In general, the coefficient of permeability of sandy soil is 10^3 to 10^5 times larger that of cohesive soil. In the case of sandy ground, pore water is considered to have been drained completely during construction. Thus, the shear strength of sandy ground is evaluated with the angles of shear resistance ϕ_D and cohesion c_D under the drained condition. Because c_D is negligibly small in general, the shear strength of sandy ground is evaluated with ϕ_D as the only strength parameter with c_D set at 0.

In contrast, because of low permeability, the shear strength of cohesive ground has been barely changed with the pore water undrained during construction. In the case of cohesive soil ground, the undrained shear strength before construction is used as the strength parameter.

If the ground has intermediate permeability, between that of sandy and cohesive ground, it shall be classified as either sandy or cohesive ground in accordance with the coefficients of permeability and construction conditions, and then shear strength shall be determined through appropriate test methods.

(3) Concept of shear strength

The shear strength of soil τ_f is generally expressed by the following equation.

$$\tau_f = c + \sigma \tan\phi \qquad \textbf{(2.3.16)}$$

where

τ_f : shear strength (kN/m^2);

c : cohesion (or apparent cohesion) (kN/m^2);

ϕ : the angle of shear resistance (or angle of internal friction) (°); and

σ : normal stress on a shear surface (kN/m^2).

When stress is applied to soil, both the effective stress and pore water pressure in the soil are changed. Given that σ, σ' and u denote the stress applied to the soil, the effective stress acting on the soil, and pore water pressure respectively, the following relationship can be established. Here, σ is called total stress as opposed to effective stress σ'.

$$\sigma = \sigma' + u \qquad \textbf{(2.3.17)}$$

The strength parameters such as c and ϕ in **equation (2.3.16)** vary depending on the shear test conditions. Among shear test conditions, the drainage condition has the largest impact on strength parameters. Soil undergoes volume changes when being subjected to shear force (and such behavior of soil is called dilatancy). Thus, the shear strength of soil varies depending on the presence or absence of volume changes (absorption or discharge of water in the case of saturated soil). Generally, the drainage condition is classified into the following three categories, and different strength parameters are used in accordance with each condition:

① Unconsolidated and undrained condition (UU condition);
② Consolidated and undrained condition (CU condition); and
③ Consolidated and drained condition (CD condition).

Fig. 2.3.14 shows the schematic diagram of the results of direct shear tests conducted under different drainage conditions ①, ②, and ③ above.[12)] The diagram shows the cange in shear strength of the soil specimens subjected to first consolidation at p_0 under increased or reduced normal stress σ. As can be seen in the diagram, under the UU condition ①, shear strength is constant and independent of σ; under the CU condition ②, shear strength linearly increases with the increase in σ in the range of $p_0 < \sigma$; and under the CD condition ③, shear strength is overall larger than that under conditions ① and ② because the consolidation and shear force causes reduced void ratios in soft cohesive soil and loose sand. However, when σ is significantly smaller than p_0 (at the critical value of normal stress indicated as σ^* in the diagram), the shear strength under the CD condition is smaller than the strength under the CU condition due to the swelling effect of soil specimens when subjected to shear force. The above relationships are summarized below in terms of the range of σ.

In the range of $p_0 < \sigma$ (namely a surcharge load is larger than the pre-consolidation pressure);
①<②<③

In the range of $\sigma^* < \sigma < p_0$ (namely a surcharge load is slightly smaller than the pre-consolidation pressure;
②<①<③ or ②<③<①

In the range $\sigma < \sigma^*$ (namely a surcharge load is significantly smaller than the pre-consolidation pressure);
③<②<①

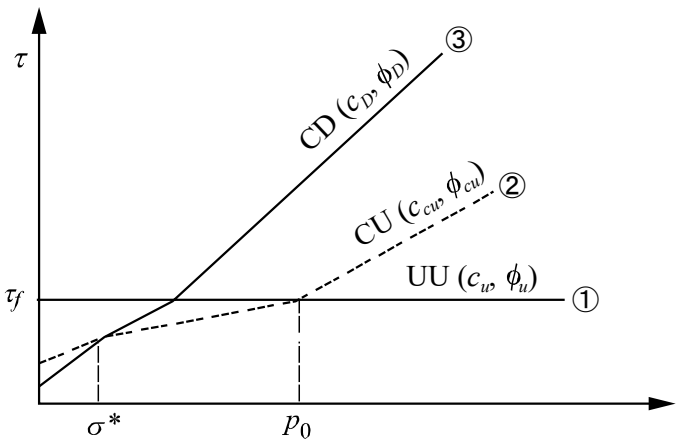


Fig. 2.3.14 Relationship between Drainage Conditions and Shear Strength[13)]

Considering that the shear strength under the most dangerous drainage condition should be used for ground performance verification, the following drainage conditions can be the determining factors of shear strength in respective cases.

(a) Case of rapid loading on cohesive soil ground:

Because shear strength increases as consolidation progresses over time, the ground is in the most dangerous condition immediately after loading, when almost all pore water is undrained (this is called a short-period stability problem). The shear strength τ_f to be used under such a drainage condition is the shear strength c_u which can be obtained through UU tests. The shear strength c_u is also called apparent cohesion, and the analysis using c_u is called $\phi_u = 0$ method. The situations to which the shear strength c_u is applied are the construction of revetments, breakwaters (without involving excavation), landfills, and earth fill on soft cohesive ground.

(b) Case of the ground with high permeability or very slow loading, allowing pore water to be drained completely during construction:

In this case, the strength of the ground is expected to increase with pore water drained while the loading is applied to it and ground performance verification is carried out with the strength parameter c_D or ϕ_D which can be obtained through CD tests. The situations to which c_D or ϕ_D is applied are the construction of revetments, breakwaters, landfills, and earth fill on sandy ground.

(c) Case of expecting strength increase by phased construction:

In phased construction, loading is divided into some phases and conducted stepwise. After each phase of loading, the ground is allowed to have a consolidation period long enough to increase its strength. The load of the next phase is small enough to reliably secure the safe loading operation. Because loading in each phase is implemented in a short period of time, drained condition cannot be applied to the staged construction; however, ground strength increases can be expected during consolidation periods. The shear strength to be used under such a drainage condition is the undrained shear strength c_u which can be obtained through CU tests.

(d) Case of the ground which has poor permeability and is subjected to a load reducing normal stress σ on a shear surface:

In this case, the ground gradually increases the level of danger along with the decrease in shear strength because the ground undergoes swelling due to reduction of the normal stress σ (this is called a long-period stability problem). As can be seen in **Fig. 2.3.14**, when the OCR is small (σ is slightly smaller than p_0), the undrained shear strength c_u is the smallest after swelling. In this case, the shear strength to be used is c_u taking the strength reduction due to swelling into consideration. The situations to which c_u is applied are earth retaining and excavation work in cohesive soil ground and the behavior of cohesive soil ground after the removal of preloading. In contrast, in the case of heavily overconsolidated ground with σ significantly smaller than p_0, the CD shear strength is the smallest and, therefore, c_D or ϕ_D is the strength parameter to use in performance verification. The situations to which c_D or ϕ_D is applied are normally cut earth work and construction work in coastal areas involving the removal of preloading, such as deepening quaywalls and seabed dredging.

The strength parameters to be frequently used when examining the construction conditions of port facilities in the performance verification are: the undrained shear strength under the UU condition as described in (a) above (or the CU condition in (c) above when expecting strength increase through staged construction) for cohesive soil ground and the drained shear strength under the CD condition in (b) above for sandy soil ground. The calculation equations of respective shear strength are as follows.

1) Cohesive soil ground (with the sand content of less than 50%)

$$\tau = c_{\mathrm{u}} \tag{2.3.18}$$

where

τ : shear strength (kN/m^2); and

c_{u} : undrained shear strength (kN/m^2).

2) Sandy soil ground (with the sand content of 80% or more)

$$\tau = (\sigma - u)\tan\phi_{\mathrm{D}} \tag{2.3.19}$$

where

τ : shear strength (kN/m^2);

σ : normal stress on a shear surface (kN/m^2);

u : in-situ steady hydraulic pressure (mainly static hydraulic pressure) (kN/m^2); and

ϕ_{D} : the angle of shear resistance under drained conditions (º).

Soil with 50 to 80% sand content shows properties intermediate between sandy and cohesive soil and, therefore, is called intermediate soil. Because evaluating the shear strength of intermediate soil is difficult compared to sandy or cohesive soil, the shear strength for such soil should be evaluated carefully by referring to the latest research results and the performance records of previous survey, design and construction. With respect to the intermediate soil that can be treated as cohesive soil, it is preferable to evaluate shear strength through triaxial CU tests, etc., rather than unconfined compression tests, which often significantly underestimate the shear strength. As described later, the shear strength shall be carefully set in the case where intermediate soil undergoes dilation when subjected to shear force.

(4) Shear strength of sand

Because sandy soil is highly permeable and considered to be under a completely drained condition, the shear strength of sand can be expressed by **equation (2.3.19)**. The angle of shear resistance under drained condition ϕ_{D} can be obtained through triaxial CD tests. Also, because the ϕ_{D} of sand gets larger with decrease in void ratio or increase in density, it is necessary to obtain accurate void ratios e_0 of ground in a manner that samples less disturbed specimens from the ground and conducts laboratory tests using such specimens as undisturbed ones. Although even sand will show slightly different ϕ_{D} values depending on shear conditions, the ϕ_{D} obtained through triaxial CD tests using undisturbed specimens preserving in-situ conditions under the confined pressure according to the design condition can be used as the characteristic value in stability analyses. However, when examining the bearing capacity problems of foundations susceptible to progressive fracture, bearing capacity may be overestimated if the ϕ_{D} obtained through triaxial CD tests is directly used as the characteristic value.

Generally, compared with the case of cohesive soil, sampling less disturbed sand specimens is technically difficult and also very expensive. Thus, there are many cases of obtaining the angles of shear resistance of sand ground not from laboratory test results but from the SPT-N values measured through the standard penetration tests. For equation to calculate ϕ_{D} from SPT-N values, refer to **Part II, Chapter 3, 2.3.4 (4) Angle of Shear Resistance of Sandy Ground**.

(5) Shear strength of cohesive soil

This section focuses on cohesive soil with a clay and silt content of 50% or more. There are several methods for determining the undrained shear strength c_{u} of cohesive soil as introduced below. It is necessary to select an appropriate strength determination method by comprehensively evaluating the performance records involving the object ground, ground properties, and the importance of facilities.

① **q_u method:**

In this method, the undrained shear strength c_u of cohesive soil for performance verification is calculated by the following equation, which uses unconfined compression strength q_u of undisturbed specimens.

$$c_u = q_u / 2 \tag{2.3.20}$$

The unconfined compression tests shall be conducted as specified in the **Method for Unconfined Compression Test of Soils (JIS A 1216)**. The unconfined compression tests are conducted mainly with high-plasticity cohesive soil. Because specimens are not subjected to confined pressure, test results are susceptible to the disturbance of specimens, and there may be the cases of getting significantly small strength. Unconfined compression tests are particularly difficult to apply to cohesive soil sampled from deep ground, such as the Pleistocene cohesive soil, which is hard and liable to crack. Also, the unconfined compression test shall not be considered applicable to intermediate soil with high sand contents because the test results show too small a shear strength with the effective stress in specimens of such soil easily released during the tests. Thus, in the case of intermediate soil, it is preferable to employ other shear tests where specimens are subjected to consolidation with confined pressure applied to them such as triaxial test or direct shear test.

② **Method using the strength obtained through triaxial tests taking into consideration initial stress and anisotropy:**

When analyzing the stability of embankment on cohesive soil ground with respect to circular slip failures, as shown in **Fig. 2.3.15**, because the soil immediately below the embankment is subjected to shear force due to increased vertical stress, the shear strength of the soil with respect to such shear force can be evaluated through triaxial undrained compression tests (compression shear tests under CU condition). Strictly speaking, there is a difference between shear strength under plane strain condition and axi-symmetric condition, and the former is slightly larger than the latter. In contrast, the shear force is generated on the soil at the end point of the slip circle, or at the foot of a slope, due to increased horizontal stress. Therefore, the shear strength of the soil with respect to such shear force can be evaluated through triaxial undrained extension tests (extension shear tests under CU condition). There is a difference between shearing conditions in the triaxial extension test and the circular slip failure, in addition to the difference between shear strength under plane strain condition and axi-symmetric condition. Horizontal stress increases in the circular slip failure of embankment, whereas axial force is reduction in the triaxial extension test. In the case of the soil at the bottom of the slip circle, which is subjected to almost horizontal shear force unlike in the case of soil subjected to deformation modes due to compression and extension force, the shear strength of the soil can be evaluated through a box shear test or simple shear test.

The undrained shear strength c_u* to be used in the performance verification is generally calculated as the average of the shear strength c_{uc} and c_{ue} obtained through compression and extension tests, respectively, using the following equation.

$$c_u^* = \frac{c_{uc} + c_{ue}}{2} \tag{2.3.21}$$

The shear strength c_{us} obtained through box shear tests may be used as a characteristic value. In many cases of Japanese cohesive soil, the shear strength c_{ue} obtained through triaxial undrained extension tests is about 70% of the shear strength c_{uc} obtained through triaxial undrained compression tests.

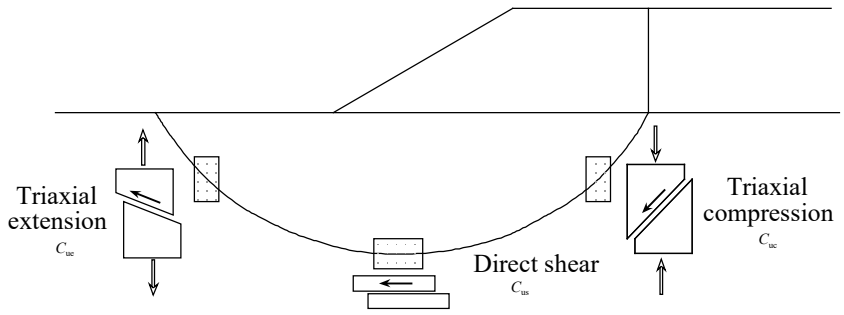


Fig. 2.3.15 Stability Problem and Strength Anisotropy of Earth Fill Constructed on Cohesive Soil Ground

Sampling inevitably causes specimens to have a certain level of disturbance despite continued efforts to minimize it. Although unconfined compression tests are considered unreliable, many performance verification methods have been empirically established besed on the tests and persist without being replaced by other methods. Among the methods for determining undrained shear strength currently proposed, the **recompression method**[14] is considered the most reliable because it is capable of reducing the influence of disturbance of specimens on test results in a manner that allows the specimens to undergo consolidation with their in-situ stress states restored.

The triaxial recompression test shall be implemented in accordance with the standards established by the Japanese Geotechnical Society, titled the **Method for K_0 Consolidated-undrained Triaxial Compression (K_0CUC) Test on Soils with Pore Water Pressure Measurements (JGS 0525)**, and the **Method for K_0 Consolidated-undrained Triaxial Extension (K_0CUE) Test on Soils with Pore Water Pressure Measurements (JGS 0526)**. Soil elements are subject to vertical overburden effective stress σ'_{v0}, and horizontal earth pressure at rest σ'_{h0} ($=K_0\sigma'_{v0}$) *in-situ*. Sampled specimens have no total stress under atmospheric pressure, but do have a certain level of isotropic residual effective stress due to suction. The recompression method allows specimens to undergo consolidation with $\sigma'_1=\sigma'_{v0}$, $\sigma'_3=K_0\sigma'_{v0}$ in a triaxial test equipment (this process is called recompression), thereby enabling undrained shear tests to be conducted with their in-situ stress states restored. The effective overburden pressure σ'_{v0} can be calculated from the unit weight of sampled specimens. The problem here is how to obtain the coefficient of earth pressure at rest K_0. Several methods have been proposed for obtaining it through in-situ tests. The coefficient can also be obtained through a laboratory test called the K_0 consolidation test (a consolidation test which controls the cell pressure σ_3 to avoid changing the cross-sectional areas of specimens when the axial pressure σ_1 or the axial strain ε_1 is increased). The values of K_0 obtained through this method are those in the normally consolidated state (expressed as K_{0NC} in many cases) and close to those of the soil in actual ground in pseudo-overconsolidated states due to the aging effect (the K_0 values of the soil in intensively overconsolidated states is much higher than the K_0 values obtained through the method). Japanese cohesive soil in a normally consolidated state generally has K_0 values in the range of 0.45 to 0.55.

The recompression method can also be implemented in the box shear tests. In this case, because shear rings can restraint the change in specimen diameters, the recompression method can be implemented by simply controlling consolidation pressure to the effective overburden pressure σ'_{v0} without paying special attention to K_0 values.

Although the values of undrained shear strength ($q_u/2$) obtained through unconfined compression tests have wide variations, their averages almost coincide with those of undrained shear strength (averages of c_{uc} and c_{ue}), obtained through triaxial compression and extension tests using the recompression methods which can restore in-situ stress states in specimens. The triaxial compression and extension tests using the recompression method, of which effective stress state of specimens and mechanical rationale are clearer, provide more reliable results than the unconfined compression tests. Thus, it is expected that triaxial tests from which results with small variations can be obtained are advantageous for performance verification based on **Part II, Chapter 3, 2.1 Estimation of Geotechnical Properties**. Also, in the case of intermediate soil, because the unconfined compression tests may lead to uneconomical design through underestimating undrained shear strength, the recompression method is preferable.

The recompression method is used mainly for obtaining in-situ undrained shear strength of naturally deposited cohesive soil. It can also be used to evaluate the undrained shear strength if specimens undergo consolidation considering the increment in in-situ stress due to loading.

When extension tests cannot be conducted, extension strength can be estimated from compressive strength using the ratio of undrained extension strength to undrained compressive strength as the anisotropy of shear strength c_{ue}/c_{uc} (about 0.7 in many cases of Japanese cohesive soil).

③ **Method using strength obtained through box shear test:**

This method uses the strength τ_{DS} obtained through box shear tests after undisturbed specimens of cohesive soil are subjected to one-dimensional consolidation with in-situ effective stress. The box shear tests can be conducted in accordance with the standards of the Japanese Geotechnical Society, titled the M**ethod for Consolidated Constant-volume Direct Box Shear Test on Soils (JGS 0560)**. The undrained shear strength, c_u, to be used in performance verification is given by the following equation:

$$c_{\mathrm{u}} = 0.85\tau_{\mathrm{DS}} \tag{2.3.22}$$

In this equation, 0.85 is a correction factor with respect to a shear rate effect. Thus, in this method, derivative values are obtained through primary processing of measured values.

④ **Method combining the strength obtained through unconfined compression tests and triaxial compression tests:**

The q_{u} method, which uses unconfined compression strength, has a problem with low reliability when evaluating soil with no past record because unconfined compression strength is susceptible to specimen disturbance when they are sampled. In order to solve the problem, in the method described in this section, strength is determined through evaluating quality of specimen in a manner that combines q_{u} and strength obtained from triaxial CU tests using undisturbed specimens. In this method, the specimens are subjected to first isotropic consolidation with in-situ mean effective stress (equivalent to $2\sigma'_{\mathrm{v0}}/3$ when K_0=0.5) for two hours and then undrained compression through triaxial CU tests. The undrained shear strength, obtained through the above processes, is finally multiplied by 0.75 as empirical correction. That is, like the boxshear test, this method requires obtaining derivative values through primary processing of measured values. This triaxial test is also called a simplified triaxial test (simplified CU test) and has been used as an alternative to the triaxial test with the recompression method. Also, this method is used for natural ground and cannot be applied to unconsolidated ground such as ground just after reclamarion. This method does not require measuring pore water pressure. For more details, see **references 15)** and **16)**.

⑤ **Method for obtaining shear strength through in-situ vane shear test**

The vane shear test can be conducted in accordance with the standards of the Japanese Geotechnical Society, titled the M**ethod for Field Vane Shear Test (JGS 1411)**. The average values of the shear strength $c_{\mathrm{u(v)}}$ obtained through the vane shear tests can be used for performance verification as the undrained shear strength c_{u}.[17] The in-situ vane shear test is advantageous in that it can be conducted by flexibly changing positions in survey areas and accurately measure the shear strength of cohesive soil, even when it is too soft to sample self-standing specimens for unconfined compression tests. Thus, the vane shear test is suitable for construction management of, for example, soil improvement work through the vertical drain method. Although the test method and principle are simple, it must be conducted with attention to friction on a rod, which may affect test results and, therefore, requires measures to calibrate the influence of friction on test results or to reduce friction. However, the vane shear test has rarely been used in actual port development works.

⑥ **Method for obtaining shear strength through electric cone penetration test:**

The electric cone penetration test can be conducted in accordance with the standards of the Japanese Geotechnical Society, titled the **Method for Electric Cone Penetration Test (JGS 1435)**. In the test, the following three parameters are measured: cone penetration resistance q_{c}; skin friction f_{s}; and pore water pressure u. Measurements of cone penetration resistance q_{c} are affected by the influence of pore water pressure and, therefore, reported as tip resistance q_{t} after the correction of such influence. In the case of cohesive soil, the shear strength c_{u} can be estimated by the following equation.

$$c_{\mathrm{u}} = \frac{q_{\mathrm{t}} - \sigma_{\mathrm{v0}}}{N_{\mathrm{kt}}} \tag{2.3.23}$$

In the equation, σ_{v0} is overburden pressure as total stress, and N_{kt} is a constant called cone coefficient. The values of the cone coefficient N_{kt} differ according to ground characteristics and are in the range of 8 to 16. It is necessary to preliminarily set an appropriate value of N_{kt} of object ground by comparing the calculated values to undrained shear strength additionally obtained through a reliable method, such as the triaxial recompression test with respect to undisturbed specimens.

In the electric cone penetration test, measuring at short intervals is conducted and provides practically continuous data in depth direction. Therefore, thin strata typical in heterogeneous ground can be detected. In contrast, in the case of gravelly ground, gravel which is hit by a cone may cause a test result to have excessively high tip resistance or may prevent the cone from penetrating further into the ground. New penetration equipment, which has been developed and commercialized recently, has a rotary boring mechanism so that equipment operation can be swiftly switched from cone penetration to rotary boring when a cone hits gravel or hard strata and switched back to cone penetration when rotary boring reaches a measurable depth.

In order to set a cone coefficient Nkt, it is necessary to implement either one of the methods described in ① to ④ requiring laboratory tests or the method described in ⑤ requiring direct in-situ measurement through the vane shear test. Also, there may be a necessity of additional surveys at many locations to interpolate the data between major survey points. The electric cone penetration test, which can be conducted easily, is considered to be effective in such a case.

These methods described in ① to ⑥ above have their distinctive characteristics and, therefore, it is necessary to select and implement the method appropriate for the purposes and object soil as needed.

The undrained shear strength cu of cohesive soil is increased with the progress of consolidation. Also, the greater consolidation loads, the greater cu after consolidation. Because the overburden pressure gets larger with increasing depth, the soil at deeper depth is subjected to larger consolidation pressure. Thus, the cu of normally consolidated ground gets larger with increasing depth in general. Based on this, the cu to be used in the performance verification can be normally given by the following equation.

$$c_{\mathrm{u}} = c_{\mathrm{u0}} + kz \qquad \textbf{(2.3.24)}$$

where

c_{u} : the undrained shear strength at depth z from the surface of a cohesive stratum;

c_{u0} : the undrained shear strength on the surface of a cohesive soil stratum;

k : an increase rate of c_{u} at depth z; and

z : the depth from the surface of a cohesive soil stratum.

(6) Increase in strength of cohesive soil due to consolidation

The undrained strength of cohesive soil increases with the progress of consolidation. Thus, in the soil improvement methods, such as the vertical drain method, which improves the strength of cohesive soil by accelerating pore water drainage due to consolidation, a strength increase ratio c_{u}/p by consolidation is an important parameter. Naturally deposited cohesive ground may be in a slightly overconsolidated state (actually pseudo-overconsolidated state as explained below) or, even in a normally consolidated state in terms of stress history, appear to be overconsolidated with consolidation yield stress p_{c} larger than effective overburden pressure σ'_{v0} due to aging effect. For this reason, by normalizing undrained shear strength c_{u} not by effective overburden pressure σ'_{v0} equivalent to consolidation pressure but by consolidation yield stress p_{c} ($m=c_{\mathrm{u}}/p_{\mathrm{c}}$), the strength increase ratio can be a parameter specific to cohesive soil. The larger the value of $c_{\mathrm{u}}/p_{\mathrm{c}}$, which is a soil property used when increasing soil strength through, for example, the vertical drain method, the higher the strength increase ratio will be, thereby enabling higher soil improvement effects to be achieved. Based on the performance records and existing survey results with respect to Japanese marine cohesive soil, the value of $c_{\mathrm{u}}/p_{\mathrm{c}}$ lies in a range shown by the following equation regardless of its plasticity.

$$c_{\mathrm{u}}/p_{\mathrm{c}} = 0.2 \sim 0.25 \qquad \textbf{(2.3.25)}$$

Considering that overconsolidation ratios OCR (= $p_{\mathrm{c}}/\sigma'_{\mathrm{v0}}$) of naturally deposited cohesive soil are generally in the range of 1.0 to 1.5, and the effective overburden pressure can be expressed by $\sigma'_{\mathrm{v0}}=p_{\mathrm{c}}/\mathrm{OCR}$, the data (in **Fig. 2.3.16**)[18] exemplifies the credibility of **equation (2.3.25)**.

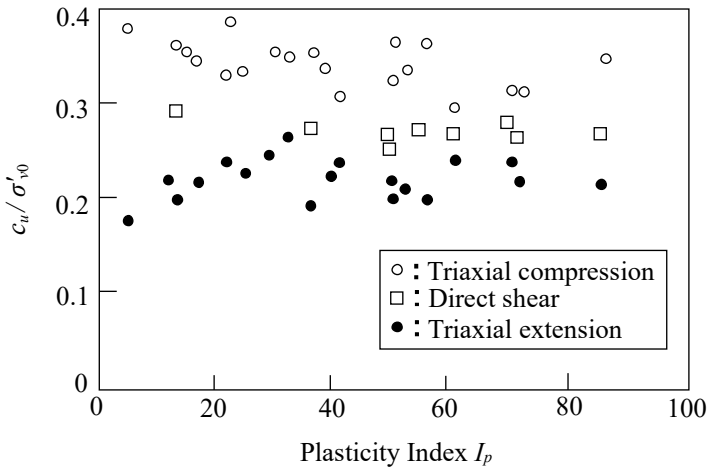


Fig. 2.3.16 Relationship between Plasticity Indexes and c_u/σ'_{v0}[18)]

(7) Decrease in strength of cohesive soil due to swelling

When a load is partially removed after consolidation, cohesive soil undergoes swelling with time, thereby reducing the c_u. In addition, the time required for the swelling is considerably shorter than the time required for consolidation. The drainage condition in this case corresponds to the CD condition in an overconsolidated state, as described in **Part II, Chapter 3, 2.3.3 Shear Characteristics** and, therefore, the strength should be evaluated considering its reduction after swelling.[19)] Examples of situations corresponding to this case are: removal of loads at the end of consolidation in soil improvement works, such as the vertical drain method or the preload method; excavation for earth retaining structures[20)]; and dredging to deepen the sea bottom.

(8) Strength of intermediate soil

Soil with sand content of 50 to 80% is classified as intermediate soil with properties intermediate between sandy and cohesive soil.[21)] The shear strength of intermediate soil is calculated assuming it as either sandy or cohesive soil, depending on the permeability and design conditions. In the case of intermediate soil with high sand content or coral gravel soil (limited to that with low coral gravel contents), it is preferable to devise test methods and compare the laboratory test results of the coefficients of permeability with those obtained through in-situ permeability tests or the electric cone penetration test.[22)] This is because consolidation tests have a high possibility of underestimating coefficients of permeability due to limited test conditions. When the coefficients of permeability determined through the methods described above are $1\text{x}10^{-4}$ cm/s ($1\text{x}10^{-6}$ m/s) or more, the ground can be regarded permeable and the ϕ_D can be determined through the electric cone penetration test or triaxial CD test with c_D=0. According to previous research on Japanese intermediate soil, the values of ϕ_D are not less than 30º in many cases.[23), 24), 25), 26) and 27)] However, there may be cases where the electric cone penetration test cannot be applied to the ground with gravel. For the ground surveys on the soil with coral gravel, refer to the **Survey and Design Manual of Coral Gravelly Soil**.

When the coefficients of permeability are not more than $1\text{x}10^{-4}$ cm/s ($1\text{x}10^{-6}$ m/s), intermediate soil is considered cohesive in performance verification. However, because the influence of releasing confined pressure during sampling on less disturbed specimens is much greater in intermediate soil than in cohesive soil, the shear strength obtained through the unconfined compression test (q_u) method is underestimated. Although there have been proposed methods for obtaining shear strength by correcting the unconfined compressive strength of specimens with high sand contents using cohesive soil contents or plasticity indexes[28)], the triaxial test using the recompression method is preferably used to evaluate the shear strength of intermediate soil ground. Alternative methods equivalent to the triaxial test using the recompression method include comparing shear strength obtained through both unconfined compression and triaxial tests and using the strength obtained through the constant-volume box shear test.[29)]

In contrast, the intermediate soil with high silt or coarse particle contents is likely to undergo dilation during shearing. Thus, in the triaxial CU test, such intermediate soil shows a strong tendency of strain hardening where shear strength is increased with the progress of shearing without expressing peak strength, thereby making it difficult to set shear strength. This is because negative excess pore water pressure is generated along with dilation. Since actual soil conditions do not correspond to the definite undrained conditions of triaxial tests, such negative

excess pore water pressure may be too large for soil to have under actual conditions and thereby causing overestimation of shear strength. Thus, as a measure to avoid the overestimation problem, there is a method for setting undrained shear strength with that obtained through the triaxial CD test as the upper limit.

2.3.4 SPT-N value Interpretation Methods

(1) The angle of shear resistance of sandy soils can be calculated by the following equation from the values obtained through the standard penetration test.

$$\phi = 25 + 3.2\sqrt{\frac{100N}{\sigma'_{v0} + 70}} \qquad \textbf{(2.3.26)}$$

where

ϕ : the angle of shear resistance of sand (°);

N : the SPT-N value obtained through the standard penetration test; and

σ'_{v0} : the effective overburden pressure at the depth where the standard penetration test is performed (kN/m^2).

(2) SPT-N values have been associated with a wide variety of geotechnical properties so far. When using related equations of SPT-N values, it is necessary to confirm their scope of application by examining the backgrounds that led to the proposal and establishment of related equations and ground conditions. Many related equations, such as Dunham's equation, which has been commonly used for many years, calculate the ϕ directly from SPT-N values without considering effective overburden pressure σ'_{v0}. However, because the relative density D_r varies depending on the σ'_{v0} as can be seen in **Fig. 2.3.17**, it is necessary to consider the σ'_{v0} when calculating D_r from SPT-N values. This concept has been incorporated in the liquefaction determination,[30] where liquefaction resistance of ground is examined with the equivalent SPT-N values $(N)_{65}$ which are converted values under the effective overburden pressure is σ'_{v0}=65 kN/m^2. It is also known that, even ground with identical ϕ, SPT-N values get larger with increasing effective overburden pressure. Therefore, the influence of the σ'_{v0} shall be taken into account when calculating ϕ from SPT-N values.

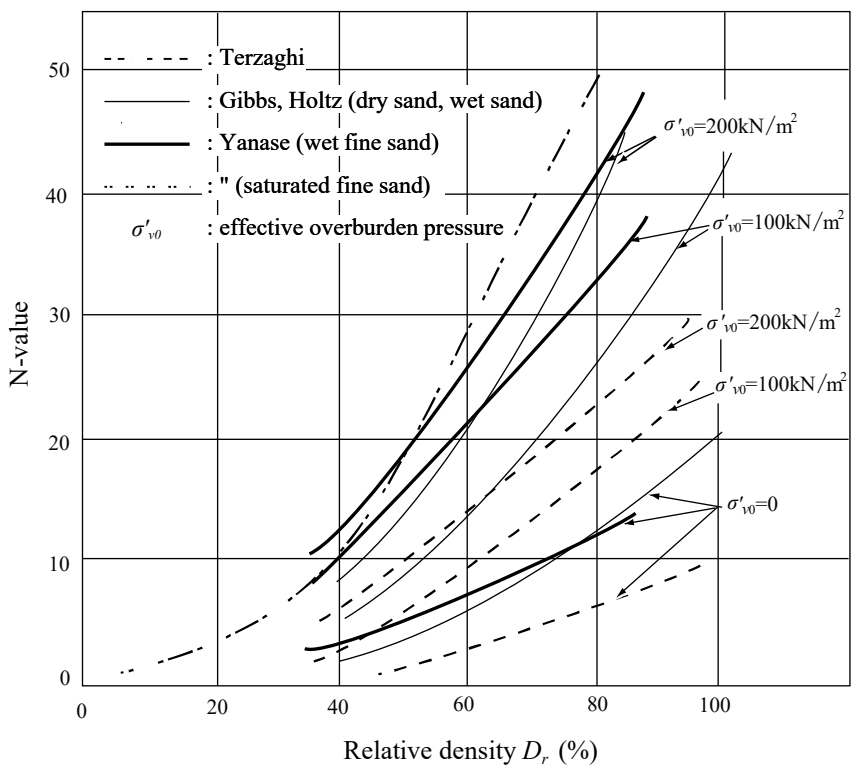


Fig. 2.3.17 Influence of Effective Overburden Pressure and Relative Density on SPT-N values[30]

(3) Factors affecting SPT-N values

Because the factors influencing SPT-N values mutually interact, methods for quantitatively correcting SPT-N values with respect to these factors have not yet been established. The factors and the extent of influences important to better understand SPT-N values are summarized below.

① **Density**

Particularly in the case of sandy ground, SPT-N values get larger with the increase in soil density (relative density).

② **Water contents**

Apart from well compacted fine sand and silty sand, the SPT-N values of saturated sand, dry sand, and wet sand get larger in this order.

③ **Effective overburden pressure**

SPT-N values get larger with the increase in effective overburden pressure σ'_{v0}.

④ **Influence of groundwater**

When groundwater levels fluctuate, effective overburden pressure and degrees of saturation vary and so do the SPT-N values.

⑤ **Other influencing factors**

SPT-N values vary depending on the shapes, grain size distributions, and mineral compositions of soil particles.

⑥ **Influencing factors attributable to test methods**

The self-weight of a boring rod increases as its length is extended with increasing depths. Thus, in the case of loose sand, SPT-N values tend to be lower than actual ones because the tip of the standard penetration test equipment sinks under the self-weight of the equipment. In contrast, in the case of dense sand, SPT-N values tend to be larger than actual ones because hammer impact efficiency is reduced by rod deflection or wobbling. In the standard penetration test, the SPT-N values to be measured differ depending on hammer dropping methods. To reduce the influence of the different hammer dropping methods on impact efficiency, the International Society for Soil Mechanics and Geotechnical Engineering has set an impact efficiency of 60% as the international standard. That is, SPT-N values shall be measured on the basis that 60% of the free falling energy of a hammer of 63.5 kg dropped from a predetermined height (76 cm) is transmitted to the tip of a rod.[31)] For reference, the SPT-N values measured through the trigger method which is one of the hammer dropping methods and has an impact efficiency of about 80%. It is about 30% lower than those measured through the internationally standardized method.[31)] Thus, caution shall be taken when referring to foreign literature and applying the information in the literature to domestic projects.

Recently, a new method using an automatic hammer dropping device has been promoted as substitute for the trigger method. The new method enables accurate tests to be conducted easily. When simply dropping a hammer with a rope hanging on a cone pulley, without using the automatic hammer dropping device or a trigger, the hammer connected to the rope is inevitably subjected to some resistance, preventing a complete free fall and thereby causing the number of hammer blows, namely SPT-N values, to be overestimated. It shall be particularly noted that overestimating SPT-N values results in the evaluation of bearing capacity and deformation moduli, as well as the determination of liquefaction, on the dangerous side.

(4) Angles of shear resistance of sandy ground

Angle of shear resistance ϕ is an important constant in performance verification of foundations, as is the case with undrained shear strength of cohesive soil. However, because the angle of shear resistance ϕ is affected by many interacting factors, even the same soil does not show constant value of ϕ. Thus, it is necessary to investigate sufficiently the background to establish performance verification methods, including the conditions on which the performance verification methods using ϕ were based.

The ground for deriving **equation (2.3.26)** is as follows. First, reference was made to **equation (2.3.27)** proposed by Meyerhof[32)] to express the relationship between SPT-N values and D_r. In this equation, D_r is in the unit of %.

$$D_r = 21\sqrt{\frac{100N}{\sigma'_{v0} + 70}} \qquad \textbf{(2.3.27)}$$

Equation (2.3.27) has been used in the **Specification for Highway Bridges (by the Japan Road Association)** when correcting SPT-N values with effective overburden pressure to examine liquefaction. Then, reference was additionally made to **equation (2.3.28)** proposed by Meyerhof[33] to express the relationship between D_r and ϕ_D.

$$\phi_D = 28 + 0.15D_r \qquad \textbf{(2.3.28)}$$

On the basis of **equations (2.3.27)** and **(2.3.28)**, ϕ_D can be calculated from SPT-N values. However, the values of ϕ_D calculated by using these equations are slightly larger than those calculated using Dunham's equation. Thus, to be consistent with the Dunham's equation (in the case of well-graded angular particles), **equation (2.3.26)** where ϕ_D becomes 25° when SPT-N value is 0 was determined to be the equation to calculate ϕ_D from SPT-N values.

(5) SPT-N values in cohesive ground

Compared to sandy ground, SPT-N values of cohesive soil are too small to be used as a reliable parameter. According to past experience and test results, SPT-N values of cohesive soil are difficult to measure unless q_u is 100kN/m^2 or more. In the case of cohesive soil with q_u not more than 100 kN/m^2, SPT-N values cannot be used to determine mechanical properties such as strength although implementing standard penetration tests still has significance in confirming whether or not object cohesive soil is soft through observing specimens taken by samplers, as is the case with preliminarily surveys; and determining the physical properties of object cohesive soil. In the case of the Pleistocene cohesive soil with high strength, repeated changes in deposition environments and stress history in the past have often caused such soil to have heterogeneous properties, even in identical strata, or to be in overconsolidation states regardless of effective overburden pressure. Therefore, there may be cases where slight differences in positions or depths cause significant differences in SPT-N values and ground properties. Also, sampling hard cohesive soil is technically difficult because hard cohesive soil specimens are easily cracked. In Japan, the strength of cohesive soil has been evaluated frequently using q_u values, but the q_u values are very easily affected by the quality of specimens. Thus, the relationships between q_u values and SPT-N values normally have large dispersion. The relationship between q_u values and SPT-N values which has been conventionally used is shown in **Fig. 2.3.18**.

Similar to rock ground, to be described in (6) below, the problem with hard cohesive soil is often not strength but other parameters, such as deformation moduli. These parameters have been associated with SPT-N values or q_u values based on past test results. Thus, it is necessary to interpret SPT-N values with due consideration of object facilities and the background of establishing empirical equations, without mechanically obtaining parameters using empirical equations associated with SPT-N values.

(6) SPT-N values of rock ground

The scope of application in which SPT-N values can be used reliably is sandy ground containing a small amount of gravel with the size of 10 mm or less. There is a risk of overestimating SPT-N values when the ground has higher gravel contents. Thus, SPT-N values are certainly too unreliable to be applied to rock ground. **Fig. 2.3.19** shows the relationship between SPT-N values and q_u values in the case of soft rock, called mudstone. As can be seen in the figure, no significant relationship cannot be found between them. When the object ground happens to be rock ground of this sort, and still measurement is required for deformation moduli and compressive strength with high accuracy, it is preferable to measure shear wave velocities or conduct horizontal loading test inside borehole or laboratory tests, using quality specimens without cracks.

(7) Interpretation of SPT-N values of the ground having SPT-N values of 50 or more

The analyses using SPT-N values have problems with dense sand and gravel, gravel, and rock ground. In ground of these types, penetration depths do not reach the predetermined 30 cm even after the number of hammer blows exceeds 50. Generally, in ground where SPT-N values exceed 50, the vibrations on rods and rebounds of hammers prevent proper penetration of samplers, thereby making accurate measurement of SPT-N values difficult. Therefore, SPT-N values are generally expressed by exact figures until 50 and, when exceeding 50, they are simply expressed by N>50. However, because there are cases of overestimating SPT-N values when tips of samplers are clogged with gravel or rock fragments, estimating strength and deformation moduli on the basis of simply determined N>50 has a risk of the overestimation of SPT-N values. Thus, it is necessary to determine N>50 comprehensively by, for example, observing specimens in samplers.

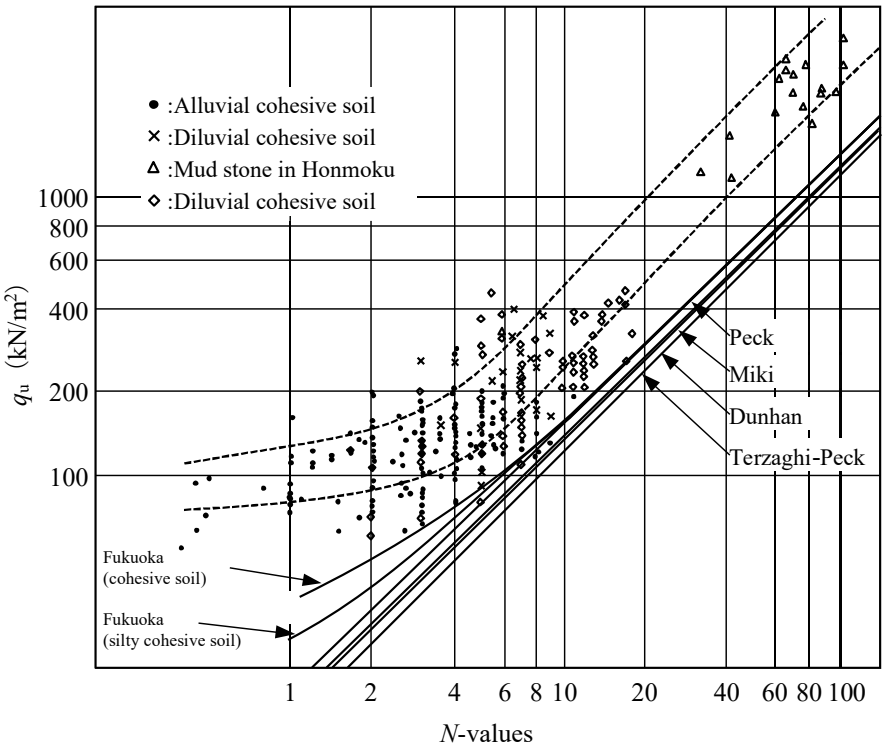


Fig. 2.3.18 Relationship between q_u Values and SPT-N values of Cohesive Soil

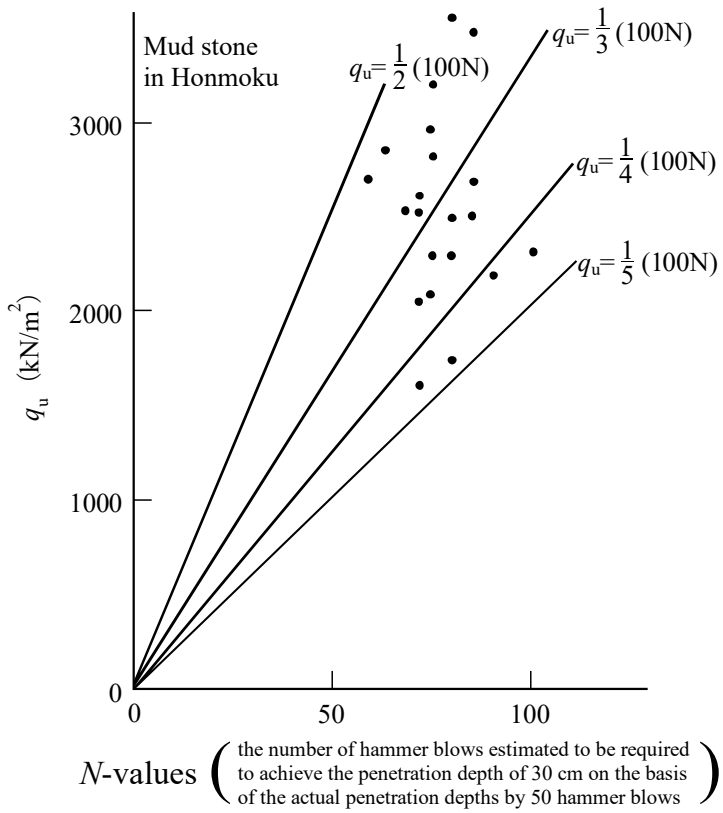


Fig. 2.3.19 Relationship between q_u Values and SPT-N values of Soft Rock

2.4 Dynamic Analyses

2.4.1 Dynamic Deformation Moduli

(1) In seismic response analyses, it is necessary to set appropriate values of dynamic deformation moduli of soil which defines the relationship between the shear stress and shear strain of soil.

(2) The performance verification methods with respect to seismic resistance can be broadly classified into static and dynamic performance verification methods. Static performance verification methods, represented by the seismic coefficient method, analyze the stability of ground and facilities through balancing force with actions, due to earthquake ground motions, applied to the ground and facilities as static inertia force. In contrast, dynamic performance verification methods analyze the stability of ground and facilities through calculating amplification ratios and amplified values of acceleration, velocities, and deformation with respect to the ground above foundations of facilities and the foundations themselves. As for the seismic response analysis methods, they are also classified as the time domain analysis method and the frequency domain analysis method. Both methods require the relationship between shear stress and shear strain of soil, which constitutes ground.

Normally the relationship between the shear stress and shear strain of soil subjected to dynamic loading is described separately by a skeleton curve and a hysteresis curve, as shown in **Fig. 2.4.1 (a)**. The skeleton curve shows remarkable nonlinearity as the shear strain amplitude becomes larger. Since the dynamic deformation moduli define the relationship between shear stress and shear strain, they shall be input appropriately when conducting seismic response analyses.

(3) Relationship between dynamic shear stress and shear strain of soil

There are many models introducing shear stress and shear strain curves of soil into analyses, including a hyperbolic model (Hardin-Dornevich model) and the Ramberg-Osgood model.[34)]

(4) Methods for displaying deformation moduli in equivalent liner model

To reasonably estimate the behavior of ground during earthquakes, it is necessary to appropriately evaluate and model relationship nonlinearity between dynamic stress and strain of soil with respect to a wide range of shear strain amplitude. When replaced with an equivalent linear model, the relationship between dynamic stress and strain of soil is expressed by two parameters: shear moduli of elasticity and damping constants (which are alternatively called moduli of rigidity and damping ratios respectively). As shown in **Fig. 2.4.1 (b)**, the shear moduli of elasticity G and the damping constants h are defined by **equations (2.4.1)** and **(2.4.2)**, respectively, with respect to the shear strain amplitude. The scope of application of the equivalent linear model is considered up to strain levels of 10^{-3}. When strain levels exceed 10^{-3}, due consideration shall be given to interpreting the calculation results.

$$G = \frac{\tau}{\gamma} \tag{2.4.1}$$

$$h = \frac{\Delta W}{2\pi\, W} \tag{2.4.2}$$

where

G : a shear modulus of elasticity (kN/m^2);

τ : shear stress amplitude (kN/m^2);

γ : shear strain amplitude;

h : a damping constant;

W : strain energy (kN/m^2); and

ΔW : damping energy (kN/m^2).

The values of the shear modulus of elasticity G and the damping constant h, with respect to arbitrary shear strain amplitude γ, vary depending on the values of γ as shown by a G/G_0-γ curve and a h-γ curve in **Fig. 2.4.2**. In the figure, G_0 is the shear modulus of elasticity corresponds to $\gamma \fallingdotseq 10^{-6}$.

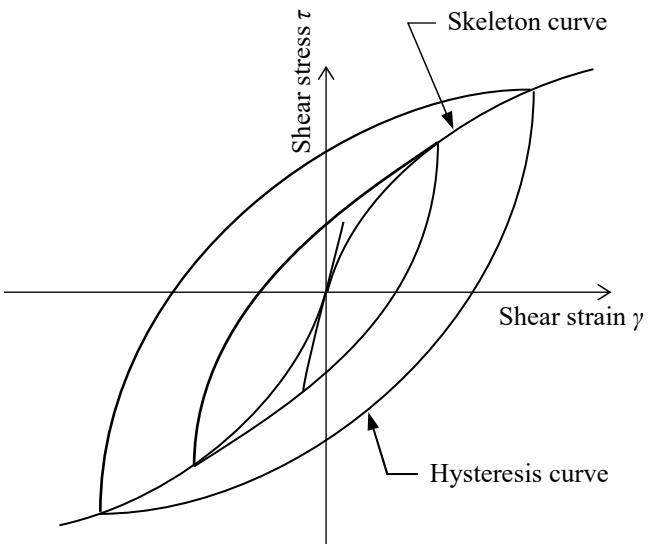


Fig. 2.4.1 (a) Stress-strain Curve

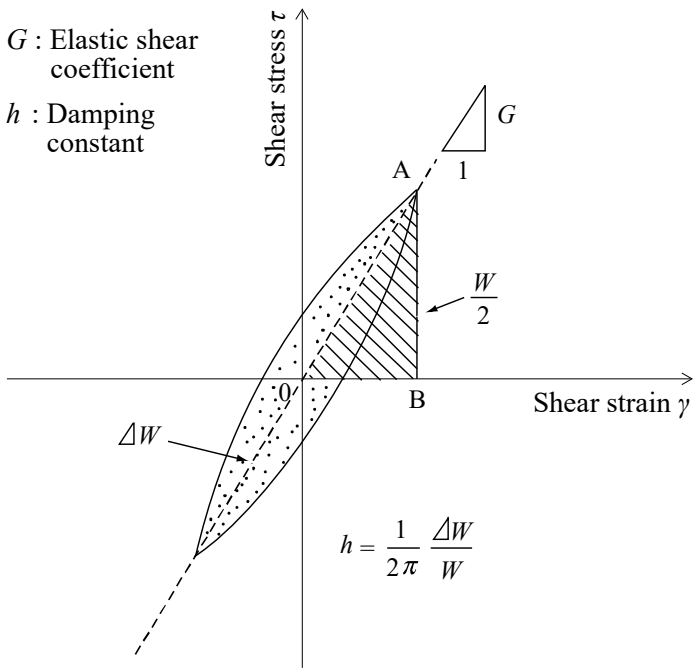


Fig. 2.4.1 (b) Shear Modulus of Elasticity and Damping Constant

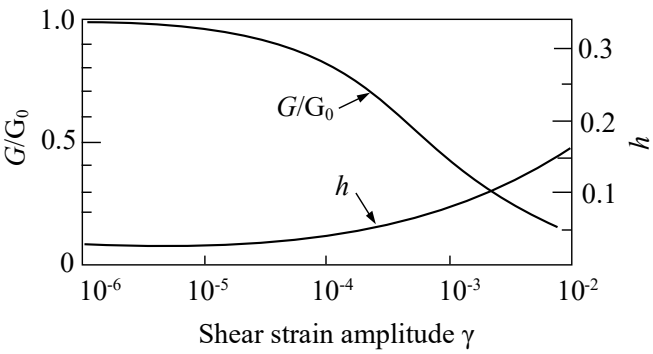


Fig. 2.4.2 Shear Modulus of Elasticity, Damping Constant and Shear Strain Amplitude

(5) Measurement of shear moduli of elasticity and damping constants

The shear moduli of elasticity and damping constants shall be obtained through laboratory tests, such as the cyclic triaxial test and in-situ tests using elastic waves such as the PS logging and the cross hole velocity methods. Laboratory tests require undisturbed specimens sampled in-situ and have a wide scope of applications in that they can be used for the measurement of the shear moduli of elasticity and damping constants corresponding to a wide range from shear strain amplitude of 10^{-6} to failure of specimens. They can also measure change of dynamic deformation moduli caused by construction of facilities. In the cyclic triaxial test, **equation (2.4.3)** can be used to obtain shear moduli of elasticity by assuming Poisson's ratios ν.

$$G = \frac{\sigma_a}{2\varepsilon_a(1+\nu)} \tag{2.4.3}$$

where

σ_a : axial stress amplitude (kN/m^2); and

ε_a : axial strain amplitude.

The value of ν is 0.33 under a drained condition or 0.45 under an undrained condition.

The damping constants can be calculated by **equation (2.4.2)** with W and ΔW obtained from the stress-strain curve similar to **Fig. 2.4.1 (b)**.

Regarding the use of in-situ tests for the measurement of shear moduli of elasticity and damping constants, in-situ tests have been available only for shear moduli of elasticity corresponding to shear strain amplitude in the level of about 10^{-6}. In-situ tests capable of measuring both shear moduli of elasticity and damping constants in the range of wide shear strain amplitude has not been commercialized. However, in-situ tests can directly measure in-situ values directly. They are also useful in that their results can be referred to when correcting shear moduli of elasticity obtained through laboratory tests whose results are affected by specimen disturbance. The elastic constants of ground can be obtained by **equations (2.4.4)** to **(2.4.6)** using the elastic wave velocities measured through the elastic seismic exploration using boreholes.

$$G_0 = \rho V_s^2 = \frac{\gamma_t}{g} V_s^2 \tag{2.4.4}$$

$$E_0 = 2(1+\nu)G_0 \tag{2.4.5}$$

$$\nu = \frac{\left(\frac{V_p}{V_s}\right)^2 - 2}{2\left\{\left(\frac{V_p}{V_s}\right)^2 - 1\right\}} \tag{2.4.6}$$

where

V_p : a longitudinal wave velocity (m/s);

V_s : a transverse wave velocity (m/s);

G_0 : a shear modulus of elasticity (kN/m^2);

E_0 : Young's modulus (kN/m^2);

ν : Poisson's ratio;

ρ : density (t/m^3);

γ_t : wet unit weight (kN/m^3); and

g : gravitational acceleration (m/s^2).

There are various items requiring attention when applying the elastic wave exploration to the soft seabed. These items include: methods for exciting and receiving elastic waves (longitudinal and transversal waves); accuracy in reading waveforms; and methods for protecting borehole walls.

(6) Simple estimation of shear moduli of elasticity and damping constants

In cases where it is difficult to directly measure the shear moduli of elasticity and the damping constants of soil through laboratory tests or in-situ tests, there are methods for estimating them from plasticity indexes, void ratios, unconfined compressive strength, and SPT-N value.[35] However, it should be noted that the method for estimating them from SPT-N values produces large variations in estimation results, with a variation coefficient of about 0.2. For example, **Fig. 2.4.3** shows the study result of the estimation errors in measuring *S* wave velocities of Holocene sandy and cohesive soil based on the variations of SPT-N values and *S* wave velocities for respective types of ground measured by Imai.[36] The horizontal axis shows the ratio of the *S* wave velocities converted from the SPT-N values to the actual velocities. In the case of the Holocene sandy soil, the average and standard deviations of the ratios of converted *S* wave velocities to actual ones are 1.12 and 0.29 respectively. In the case of the Holocene cohesive soil, the average and standard deviations are 0.95 and 0.32 respectively. The probability distribution of both cases is considered log-normal distribution.[37]

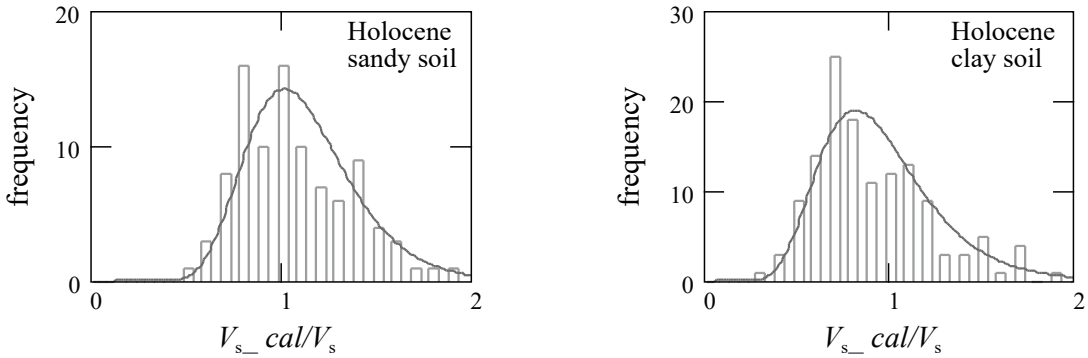


Fig. 2.4.3 Estimation Accuracy of S Wave Velocities[36]

2.4.2 Dynamic Strength Properties

(1) The soil strength with respect to dynamic actions is normally determined through laboratory tests. In this case, it is necessary to set the characteristics of the actions and ground conditions appropriately.

(2) The typical dynamic actions in ports and harbors are seismic movements and wave actions. The seismic movements are characterized by short periods and few cyclic repetitions. In contrast, the wave forces are characterized by long periods and many cyclic repetitions. These dynamic actions are generally converted into static actions, as in the seismic coefficient method; however, there are cases where they need to be analyzed as dynamic actions. Examples include predicting liquefaction during earthquake movements and examining reductions in the strength of foundation ground of structures subjected to wave actions based on dynamic strength properties obtained through cyclic triaxial tests. Depending on the purposes, the cyclic triaxial tests can be conducted with reference to the **Method for Cyclic Undrained Triaxial Test on Soils (JGS 0541)** and the **Method for Cyclic Triaxial Test to Determine Geomaterials' Deformation Properties (JGS 0542)**.

(3) Types of dynamic actions

The characteristic of dynamic actions distinctly different from those of static actions are twofold: ① dynamic actions have shorter durations; and ② dynamic actions are repetitive but static ones are not. Some dynamic actions, like earthquake movements, have both the above characteristics and others, like impulsive force of blasting which is a one-shot action with very short duration, have either of the above characteristics. Dynamic actions may also include those repetitive actions with relatively slow loading velocities, such as wave force in the board sense. **Fig. 2.4.4** shows the classifications of dynamic and static problems in terms of loading time and cyclic repetitions.[38] According to **Fig. 2.4.4**, the general wave force based on the period of around 10 seconds is considered the problem positioned around the border between dynamic and static problems.

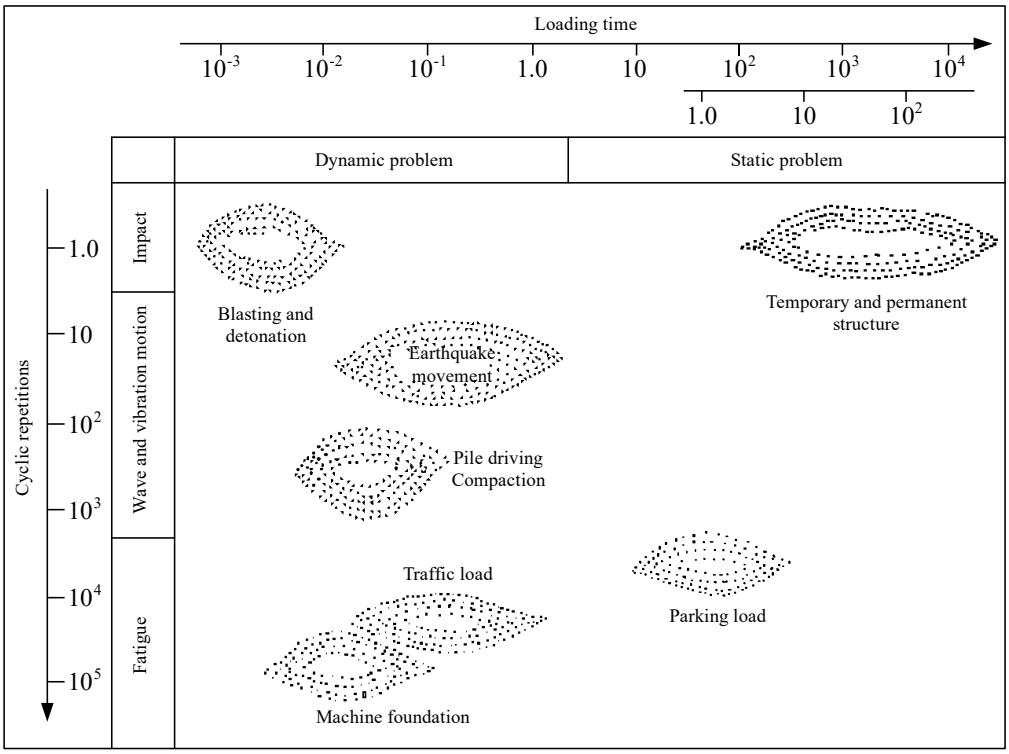


Fig. 2.4.4 Classification of Dynamic Problems in Terms of Loading Time and Recurrence Rates[38)]

(4) Test methods

Laboratory dynamic tests include cyclic triaxial, cyclic simple shear, and torsional shear tests, and they have their own characteristics. Among them, the cyclic triaxial test has been used most heavily because it produces consistent test results less influenced by individual variations in test engineers' skills. When implementing laboratory dynamic tests, reference can be made to the **Method for Cyclic Undrained Triaxial Test on Soils (JGS 0541)**, the **Method for Cyclic Triaxial Test to Determine Geomaterials' Deformation Properties (JGS 0542)**, and the **Method for Cyclic Torsional Shear Test on Hollow Cylindrical Specimens to Determine Deformation Properties of Soils (JGS 0543)**.

Dynamic strength can be examined through laboratory and in-situ tests, and there have been many cases of using laboratory tests. Undisturbed specimens are used in principle except when examining the strength of landfilling materials, for which specimens prepared by remolding disturbed soil can be used. The cyclic shear strength of soil largely differs depending on the two characteristics (duration and cyclic repetitions) with respect to dynamic actions, even though physical properties and stress states of soil are identical; and drain conditions relative to the velocities of actions. Therefore, when soil strength with respect to dynamic actions is required, due consideration shall be given to the characteristics of dynamic actions and ground conditions.

Implementing laboratory tests also requires loading conditions where dynamic actions are replaced by appropriate loads. The characteristics of actions to be considered in such a case are: waveforms (amplitude and periods); cyclic repetitions; loading velocities; and irregularity in waveforms. Thus, it is preferable to set the loading conditions which can reproduce these characteristics as precisely as possible.

The shear strength of soil differs depending on drain conditions. When subjected to dynamic actions, such as earthquake ground motions, ground is in an undrained condition because the ground's drainage velocities are relatively smaller than loading velocities. Thus, laboratory tests for such ground shall be conducted under undrained conditions. Determining whether to examine soil strength with respect to wave force under drained or undrained condition depends on the wave characteristics and stratal organization. Thus, drain conditions shall be set to evaluate the strength and deformation on the safe side. In the case of cohesive soil, laboratory tests shall be implemented under undrained conditions.

(5) Application of test results

To obtain dynamic strength, laboratory tests do not directly apply dynamic actions to specimens but apply loading conditions in which dynamic actions are simplified, to some extent. Thus, due consideration shall be given to the

relationship between the test conditions and characteristics of dynamic actions, as well as ground conditions, when using the dynamic strength obtained through laboratory tests.

When applying cyclic triaxial test results to predict ground liquefaction during earthquakes, refer to prediction and determination of liquefaction in the **Handbook of Liquefaction Countermeasures for Reclaimed Areas** (revised edition).[39)]

[References]

1) Watabe, Y., M. Tanaka, S. Sassa and Y. Kikuchi: New Determination Method of Soil Parameters for Performance Based Design, Report of the Port and Airport Research Institute, Vol.48 No.2, pp.123-143, 2009. (in Japanee)

2) Matsumoto, K. and F. Ogawa: Correlation of the Mechanical and Index Properties of Soils in Harbor Districts (First Report), Report of the Port and Harbour Research Institute, No.71, 1969. (in Japanese)

3) Ogawa, F. and K. Matsumoto: The Correlation of the Mechanical and Index Properties of Soils in Harbour Districts, Report of the Port and Harbour Research Institute, Vol. 17, No. 3, pp.3-89, 1978. (in Japanese)

4) Japanese Geotechnical Society: Japanese Geotechnical Society Standards, p.456, 2010. (in Japanese)

5) Terzaghi, K. and P.B. Peck.: Soil Mechanics in Engineering Practice, New York John Wiley and Sons Inc., p.44, 1948.

6) Mukabi., J.N., F. Tatsuoka, Y. Kohata, T. Tsuchida and N. Akino: Small strain stiffness of Pleistocene clays in triaxial compression, Proceedings of International Symposium on Prefailure Deformation Characteristics of Geomaterials, pp.189-195, 1994.

7) Nakase, A., M. Kobayashi and A. Kanechika: Undrained Shear Strength and Secant Modulus of Clays, Report of the Port and Harbour Research Institute, Vol. 11, No. 2, pp.243-259, 1972. (in Japanese)

8) Watabe, Y and T. Kaneko: Interpretation of Long-term Consolidation Behavior of Worldwide Clays on the basis of the Isotache Concept, Report of the Port and Airport Research Institute, Vol.54 No.1, pp.3-30, 2015. (in Japanese)

9) Šuklje, L.: The analysis of the consolidation process by the isotache method, Proceedings of the 4th International Conference on Soil Mechanics and Foundation Engineering, London, Vol.1, pp.200-206, 1957.

10) Mikasa, M.; Consolidation of soft clay, Kajima Publications, 1966. (in Japanese)

11) Umehara, Y.: Study on the consolidation characteristics of soils and consolidation test methods, Technical Note of the Port and Airport Research Institute, No.469, 1983. (in Japanese)

12) Ogawa, F. and K. Matsumoto: The Correlation of the Mechanical and Index Properties of Soils in Harbour Districts, Report of the Port and Harbour Research Institute, Vol. 17, No. 3, pp.3-89, 1978.

13) Japanese Geotechnical Society: Strength Parameters in Design- c, Φ and N Values, Soil Mechanics and Foundation Engineering Library No.32, pp.15-68, 1988. (in Japanese)

14) Beere, T. and L. Bjerrum: Shear strength of normally consolidated clays, Proceedings of the 8th International Conference on Soil Mechanics and Foundation Engineering, pp.39-49, 1973.

15) Tsuchida, T., J. Mizukami, K. Oikawa and Y. Mori: New Method for Determining Undrained Strength of Clayey Ground by Means of Unconfined Compression Test and Triaxial Test, Report of the Port and Harbour Research Institute, Vol. 28, No. 3, pp.81-145, 1989. (in Japanese)

16) Tsuchida, T.: Study on Determination of Undrained Strength of Clayey Ground by Mean of Triaxial Test, Technical Note of the Port and Airport Research Institute, No. 688, 1990. (in Japanese)

17) Tanaka, H. and M. Tanaka: Determination of Undrained Shear Strength of Clayey Ground Measured by Vane Shear Tests, Report of the Port and Harbour Research Institute, Vol. 33, No. 4, pp.1-17, 1994. (in Japanese)

18) Hanzawa, H. and H. Tanaka: Normalized undrained strength of clay in the normally consolidated state and in the field, Soils and Foundations, Vol.32, No.1, pp.132-148, 1992.

19) Nakase, A., M. Kobayashi and M. Katsuno: Change in Shear Strength of Saturated Clays through Consolidation and Rebound, Report of the Port and Harbour Research Institute, Vol. 8 No. 4, pp.103-143, 1969. (in Japanese)

20) Tanaka, H.: Behavior of Braced Excavations in Soft Ground, Technical Note of the Port and Airport Research Institute, No. 734. (in Japanese)

21) Kurata, S. and T. Morishita: Study on engineering characteristics of Sand-clay mixed soil, Report of Transport Technical Research Institute, Vol. 11 No. 9, 1961. (in Japanese)

22) Tanaka, H., M. Sakakibara, K. Goto, K. Suzuki and K. Fukazawa: Properties of Japanese Normally Consolidated Marine Clays Obtained from Static Piezocone Penetration Test, Report of the Port and Harbour Research Institute, Vol. 31 No. 4, pp. 61-92, 1992. (in Japanese)

23) Kobayashi, M., T. Tsuchida and Y. Kamei: Intermediate soil -Sand or clay ?- Geotech-Note 2, Japanese Society of Soil Mechanics and Foundation, 1992. (in Japanese)

24) Tsuchida, T., M. Kobayashi, S. Ifuku and I. Fukuda: Engineering properties of coral soils in Japanese south western islands, Proceedings of the International Conference on Calcareous Sediments, Vol.1, pp.137-144, 1988.

25) Tsuchida, T: Consolidation, Compression and Permeability Properties of Intermediate Soil and Mixture Soil, Soil and Foundation, TSUCHI-TO-KISO, Vol. 41, No. 7, pp.5-10, 1993. (in Japanese)

26) Watabe, Y, T. Kaneko and S. Sassa: Mechanical properties of coral-graveled soil- a parametric study using reconstituted samples, Report of the Port and Airport Research Institute, Vol.55 No.2, pp.61-74, 2016. (in Japanese)

27) Watabe, Y., T. Kaneko and S. Sassa: Mechanical properties of coral-graveled soil- an integrated governing parameter for undisturbed samples, Report of the Port and Airport Research Institute, Vol.55 No.3, pp.3-19, 2016. (in Japanese)

28) Nakase, A., M. Katsuno and M. Kobayashi: Unconfined compression strength of soils of intermediate grading between sand and clay, Report of the Port and Harbour Research Institute, Vol. 11, No. 4, 1972. (in Japanese)

29) Tanaka, H., M. Tanaka, and T. Tsuchida: Strengthening characteristics of undisturbed intermediate soil, Jour. JSCE No.589/ III-42, pp.195-204, 1998. (in Japanese)

30) Iai, S., K. Koizumi and H. Tsuchida: Affiliation A New Criterion for Assessing Liquefaction Potential Using Grain Size Accumulation Curve and N-value, Report of the Port and Harbour Research Institute, Vol. 25, No. 3, pp.125-234, 1986. (in Japanese)

31) Iwasaki, T.: Problems related to Standard Penetration Test, Foundation, Vol. 18, No. 3, pp.40-48, 1990. (in Japanese)

32) Meyerhof, G.G.: Discussion on soil properties and their measurement, Discussion 2, Proceedings of the 4th International Conference on Soil Mechanics and Foundation Engineering, Vol.3, p.110, 1957.

33) Meyerhof, G.G.: Compaction of sands and bearing capacity of piles, Journal of the Soil Mechanics and Foundations Division, ASCE, Vol.85, No.SM6, pp.1-29, 1959.

34) Ishihara, K: Fundamentals of Soil Dynamics. Mechanics, Kajima Publishing, pp.7-34, 1976. (in Japanese)

35) Coastal Development Institute of Technology (CDIT): Handbook of Countermeasure against Liquefaction of Reclaimed Land (Revised Edition), CDIT, pp.60-77, 1997. (in Japanese)

36) Imai, T.: P- and S- wave velocities of the ground in Japan, Proceedings of the 9th International Conference on Soil Mechanics and Foundation Engineering, Vol.2, pp.257-260, 1977.

37) Nagao, T., Y. Kikuchi, M. Fujita, M. Suzuki and T. Sanuki: Reliability-based design method for pile-supported wharves against the level-one earthquake ground motion, Journal of Structural Engineering, Vol.A52, pp.201-208, 2006. (in Japanese)

38) Refer to Reference 34), pp.1-6

39) Refer to Reference 35), pp.114-136

3 Groundwater Levels and Seepage

(1) General

In the performance verification of port facilities, it is necessary to give proper consideration to the groundwater levels of sand coasts, seepage flow velocities, and seepage flow rates in permeable ground, or inside facilities, as needed.

(2) Groundwater levels in coastal areas

The depths of underground saltwater surfaces in coastal areas can be estimated by **equation (3.1.1)** (refer to **Fig. 3.1.1**).[1), 2), and 3)]

$$h^2 = h_0^2 + \left(h_l^2 - h_0^2\right)\frac{x}{L} \qquad \textbf{(3.1.1)}$$

where

$$h_0 = \frac{\rho_1}{\rho_2 - \rho_1}\zeta_0 \quad , \quad h_l = \frac{\rho_1}{\rho_2 - \rho_1}\zeta_l$$

h : the depth below sea level of an interface between fresh and saltwater at a distance x (m);

h_0 : the depth below sea level of an interface between fresh and saltwater at x = 0 (m);

h_l : the depth below sea level of an interface between fresh and saltwater at x = L (m);

ρ_1 : density of fresh water (g/cm^3);

ρ_2 : density of salt water (g/cm^3);

ξ_0 : the height from sea level of fresh water surface at a shoreline (x = 0) (m);

ξ_l : the height from sea level of fresh water surface at x = L (m);

L : the distance from a shoreline (x = 0) to an observation point (m); and

x : the landward distance from a shoreline (m).

Equation (3.1.1) cannot be applied to the ground where impermeable strata exist close to ground surfaces or underground. In such cases, reference can be made to **reference 2)**. **References 3)** and **4)** can be used for tidal influences on groundwater in coastal areas. Also, for increases in groundwater levels and beach deformation due to run-up waves, refer to **Part II, Chapter 2, 7.4.9 Relationship between the Topographic Changes of Foreshore and Groundwater Levels**.

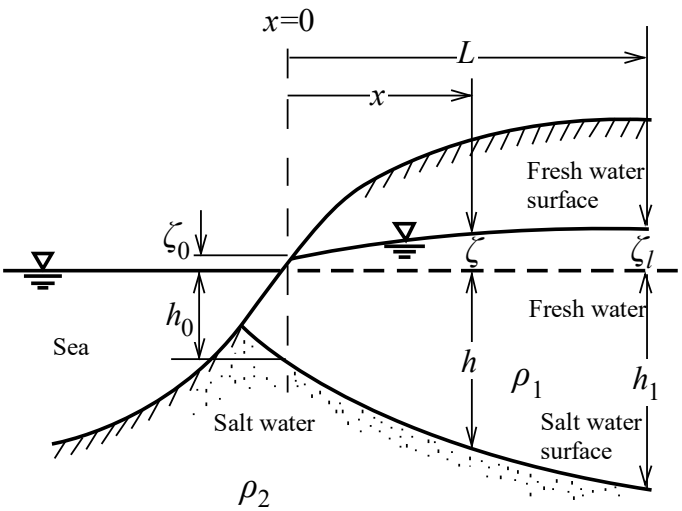


Fig. 3.1.1 Schematic Drawing of Groundwater in a Costal Area

(3) Seepage flows inside foundations and facilities

① Equation to calculate permeating flow rates

In the case of steady laminar flows in permeable strata, the seepage flow rates can be calculated by Darcy's formula, as shown below.

$$q = kiA \tag{3.1.2}$$

where

q : the flow rate of water flowing in a permeable stratum per unit time (cm^3/s);

k : a coefficient of permeability (cm/s);

i : a hydraulic gradient $i = \dfrac{h}{L}$

h : a head loss (cm);

L : the length of a seepage flow passage (cm); and

A : a cross-section area (cm^2).

The formula applicability limit is governed by the grain sizes of soil particles, constituting permeable strata, and the Reynolds numbers related to seepage flow rates. However, considering there are no sufficiently united opinions about applicability, therefore, the calculation results should be verified with actual measurements.[5)] For the details on the applicability and coefficient of permeability, reference can be made to **Part III, Chapter 3, 2.2.3 Coefficient of Permeability of Soil**.

② Seepage in permeable ground

The seepage flow rate in permeable ground can be obtained by drawing a flow net.

The flow net is a pattern of equipotential lines and flow lines, orthogonal to each other, drawn in a manner that can be locally seen as a square grid (refer to **Fig. 3.1.2**).[6)] In the flow net, the flow rate in a flow tube between two flow lines next to each other is constant, and the head loss in each square is also constant. Thus, when a flow net, as shown in the figure, can be drawn in a flow field, a total flow rate can be calculated by **equation (3.1.3)**.

$$q = kh\frac{F}{N} \tag{3.1.3}$$

where

q : a seepage flow rate per unit width ($cm^3/s/cm$);

k : the coefficient of permeability (cm/s);

h : a total water head difference (cm);

N : the number of segments on a flow line separated by equipotential lines; and

F : the number of segments on an equipotential line separated by flow lines.

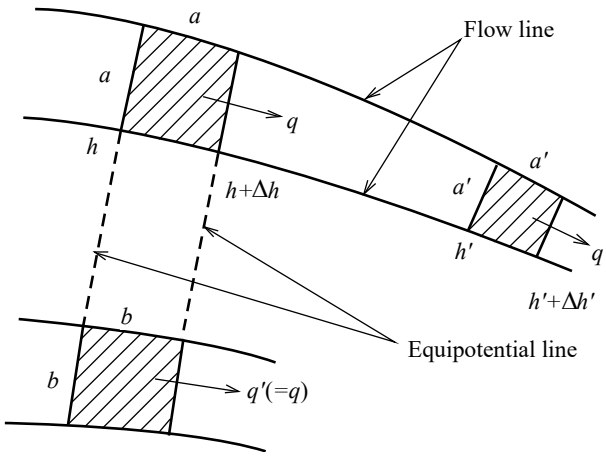


Fig. 3.1.2 Explanatory Diagram of a Flow Net

③ **Seepage of sheet pile walls**

Rates of seepage flow through the sheet pile walls are not determined only by the permeability of the walls; the dominant determination factor is the soil permeability behind the walls. Shoji et al.[7)] conducted comprehensive seepage tests by varying conditions of tension applied to interlocking joints and sand filling in the joint section, and based on the test results, proposed an empirical equation (refer to **equation (3.1.4)**).

$$q = Kh^n \tag{3.1.4}$$

where

q : a seepage flow rate per unit length in the depth direction of an interlocking joint ($cm^3/s/cm$);

K : the coefficient of permeability at a joint section (cm^{2-n}/s);

h : the pressure head difference between upstream and downstream sides of an interlocking joint (cm);

n : a coefficient determined by the state of an interlocking joint;

n≒0.5 when not filled with soil

n≒1.0 when filled with soil

The value of K is set at 7.0×10^{-4} (cm/s) when sand is filled on both sides of a sheet pile with tensile force applied to an interlocking joint. However, because the calculation result of a seepage flow rate using this value was 30 times the actual measurement, the reason for such a large difference has been investigated. Thus, when using the equation, it is necessary to give due consideration to the differences between the states of the sheet pile walls used in tests vs. the actual sheet pile walls.

④ **Seepage flows inside rubble mounds**

The seepage flow rates inside rubble mounds for gravity type structures can be calculated by **equation (3.1.5)**.

$$q = UH$$
$$U = \sqrt{\frac{2gd}{\zeta}\frac{\Delta H}{\Delta S}} \tag{3.1.5}$$

where

q : a seepage flow rate per unit width ($cm^3/s/cm$);

U : an average flow velocity in the cross section of a rubble mound (cm/s);

H : the height of a permeable layer (cm);

d : a grain size of rubble (cm);

g : gravitational acceleration (cm/s^2);

$\frac{\Delta H}{\Delta S}$: hydraulic gradient; and

ξ : a coefficient of resistance.

Equation (3.1.5) was established on the basis of the seepage tests using eight types of rubble samples with uniform grain sizes in the range of 5 to 100 mm. In the equation, the virtual flow length ΔS can be a distance obtained by adding 0.7 to 0.8 times the height of a permeable layer to the bottom width of a caisson. Also in the equation, the coefficient of resistance can be set in accordance with **Fig. 3.1.3** or set at $\xi \fallingdotseq 20$ when Reynolds number R_e (= Ud/ν) is larger than 10^4.

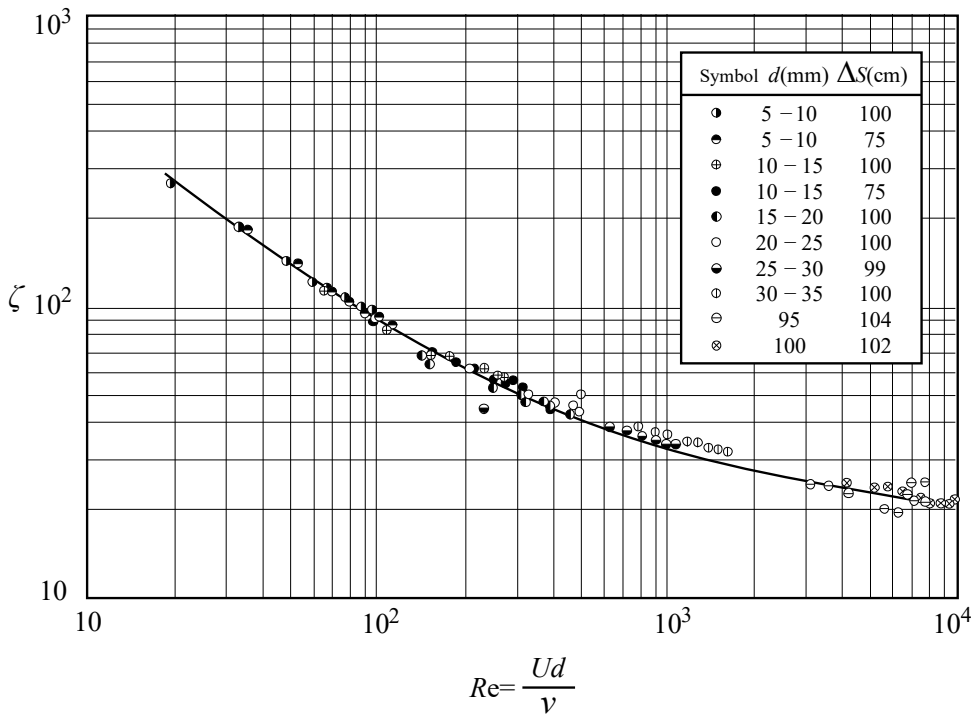


Fig. 3.1.3 Relationship between Coefficient of Resistance and Reynolds Number[8) and 9)]

[References]

1) Todd, D.K.: Groundwater hydrology, John Wiley & Sons, Inc., 1963.

2) Japan Society of Civil Engineers: The Collection of Hydraulic Formulae (1985 Edition), 1985. (in Japanese)

3) Ishihara, T. and H. Honma: Applied Hydraulics (Vol.II), Maruzen, 1966. (in Japanese)

4) Sakai, G: Geohydrology, Asakura Publishing, 1965. (in Japanese)

5) Iwasa, Y.: Hydraulics, Asakura Publishing, p.226, 1965. (in Japanese)

6) Yamaguchi, H.: Soil Mechanics, Gihodo Shuppan, p.76, 1969. (in Japanese)

7) Shoji, Y., M. Kumeda and Y. Tomita: Experiments on Seepage through Interlocking Joints of Sheet Pile, Report of the Port and Harbour Research Institute, Vol.21 No.4, pp.41-82, 1982. (in Japanese)

8) Ministry of Transport, Second Port Development Bureau: Report on the experiment of water penetration over storm surge barrier using hydraulic model of Nagoya Port, First report, 1962. (in Japanese)

9) Ministry of Transport, Second Port Development Bureau: Report on the experiment of water penetration over storm surge barrier using hydraulic model of Nagoya Port, Second report, 1963. (in Japanese)