Chapter 3 Geotechnical Conditions

Public Notice

Geotechnical Conditions

Article 13

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

[Commentary]

(1) Geotechnical Conditions
The geotechnical conditions are the various conditions that represent the geotechnical characteristics taken into consideration in the verification of the performance of the facility concerned against the technical standards. In setting the geotechnical conditions, the reliability is determined based on the results of a ground investigation and soil tests carried out by appropriate methods.

(2) Ground investigation
The ground investigation for setting the geotechnical conditions takes into consideration the structure, scale, and importance of the facility that is subject to the technical standards, as well as the nature of the ground close to the location of the facility.

(3) Soil tests
The soil tests for setting the geotechnical conditions uses methods that enable the geotechnical conditions taken into consideration in the performance verification of the facility that is subject to the technical standards to be appropriately set.

[Technical Note]

1 Ground Investigation

1.1 Methods of Determining Geotechnical Conditions

The geotechnical conditions necessary for the performance verification and the construction planning include depth of the bearing strata, depth of the engineering foundation strata, thickness of weak strata, and other stratigraphical conditions of the ground, water levels (residual water level), the density (degree of compaction), physical characteristics, shear characteristics, consolidation characteristics, hydraulic conductivity, liquefaction characteristics, etc. Soil is a material that is strongly stress-dependent, and its characteristics can change greatly due to consolidation with time, or changes in overburden, etc. Therefore when necessary a new ground investigation should be carried out. However, the size of ground investigations is limited, so past ground information (including databases, etc.) obtained from document surveys should be positively utilized. In this case, it is important to confirm that the geotechnical conditions have not changed due to changes in overburden or consolidation, or to take into consideration that the geotechnical conditions have changed.

1.2 Position, Spacing, and Depth of Ground Investigation Locations

(1) The location of a ground investigation, and the spacing and the depth should be determined in accordance with the size of the facility, the stress distribution in the ground caused by the weight of the facility, and the uniformity of the stratigraphy of the ground. However, there is also the problem of the construction cost and importance of the facility, so it is not possible to categorically regulate the number of survey points and their depth. In determining the number of survey points the uniformity or non-uniformity of the ground is the most important aspect. It is effective to check the uniformity or non-uniformity of the ground from the results of past investigations, the topography of the land, and geophysical exploration methods such as sonic wave and surface wave exploration methods. Automatically determining the spacing of ground investigation points should be avoided as much as possible, but for reference **Table 1.2.1** shows the spacing of ground investigation points for boring and sounding surveys.

The depth of the ground investigation shall be sufficient to confirm strata with sufficient bearing capacity. Whether a stratum has sufficient bearing capacity or not varies depending on the shape and scale of the facility, so it cannot be categorically determined. However, as a guide, for comparatively small scale facilities or when the foundations are not end bearing piles, the investigation may be terminated if stratum of a few meters thickness is confirmed with the N-value obtained from the standard penetration test is 30 or higher, or for a large scale facility where end bearing piles are anticipated the investigation may be terminated if stratum of a few meters thickness is confirmed with the *N* value is 50 or higher. Also, for performance verification of seismic-resistance, the investigation should continue until a stratum of engineering rock with a shear wave velocity of 300m/s or more is confirmed.

Table 1.2.1 Guideline for Investigation Location and Spacing for Boring and Sounding Investigations

① In the case where the stratigraphical conditions are comparatively uniform both horizontally and vertically (Units: m)

<table>
<tr><td colspan="2" rowspan="3"></td><td colspan="2">Face line direction</td><td colspan="4">Perpendicular to face line direction</td></tr>
<tr><td colspan="2">Spacing layout</td><td colspan="2">Spacing layout</td><td colspan="2">Distance from face line (maximum)</td></tr>
<tr><td>Boring</td><td>Sounding</td><td>Boring</td><td>Sounding</td><td>Boring</td><td>Sounding</td></tr>
<tr><td rowspan="2">Preliminary survey</td><td>Wide area</td><td>300–500</td><td>100–300</td><td rowspan="2">50</td><td rowspan="2">25</td><td colspan="2" rowspan="3">50–100</td></tr>
<tr><td>Small area</td><td>50–100</td><td>20–50</td></tr>
<tr><td colspan="2">Detailed survey</td><td>50–100</td><td>20–50</td><td>20–30</td><td>10–15</td></tr>
</table>

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

<table>
<tr><td rowspan="3"></td><td colspan="2">Face line direction</td><td colspan="4">Perpendicular to face line direction</td></tr>
<tr><td colspan="2">Spacing layout</td><td colspan="2">Spacing layout</td><td colspan="2">Distance from face line (maximum)</td></tr>
<tr><td>Boring</td><td>Sounding</td><td>Boring</td><td>Sounding</td><td>Boring</td><td>Sounding</td></tr>
<tr><td>Preliminary survey</td><td>50 or less</td><td>15–20</td><td>20–30</td><td>10–15</td><td colspan="2" rowspan="2">50–100</td></tr>
<tr><td>Detailed survey</td><td>10–30</td><td>5–10</td><td>10–20</td><td>5–10</td></tr>
</table>

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) Investigation methods that are the most suitable for the survey objectives are selected taking into consideration the extent of the survey, the importance of the facility and economics.

(2) **Table 1.2.2** shows the survey methods for each survey objective, and the ground information obtained from them.

Table 1.2.2 Survey Methods according to Survey Objectives

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 (structural disturbance is possible for all except γ_t)	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	Unconfined compressive strength Shear strength Angle of shear resistance Relative density	q_u τ_f ϕ D_r
	Consolidation characteristics	Undisturbed sampling	Compression index Compression curve Coefficient of consolidation Coefficient of volume compressibility	C_c e-log p c_v m_v
	Compaction characteristics	Disturbed sampling also applicable In- situ tests	Maximum dry density Optimum water content CBR	$\gamma_{d\max}$ w_{opt}
	Dynamic characteristics	Undisturbed sampling In- situ tests	Shear modulus Attenuation coefficient Liquefaction characteristics	G h_P

References

1) Port and Harbour Bureau, Ministry of Transport: Guideline for Port Surveys. Japan Port Association, 1987
2) The Japan Geotechnical Society: Methodology and Commentary of Soil Survey, 2004

2 Ground Constants

2.1 Estimation of Ground Constants [1)]

(1) General

The ground constants/parameters used in the performance verification are generally estimated in accordance with the flow shown in **Fig. 2.1.1.** However, if there is a rational reason based on the characteristics of the ground investigation and the soil tests, derived values may be used as the characteristic values. For example, as a method of estimating the derived values from measured values of the *N*- value obtained from standard penetration tests, empirical equations or correlation equations have been proposed that take into consideration the variation in the measured values, so the derived values can be used as characteristic value as they are. Also, for the shear wave velocity measured by geophysical exploration, the measured values evaluate the complex conditions and characteristics of the in- situ ground, and the subject being evaluated differs with each measurement location, and there are cases where the use of statistical processing of many measurement results is not appropriate. In this case the derived values may also be used as they are as characteristic values.

Partial factors that are multiplied by the characteristic values to calculate design values may be set based on the variability of the ground parameters and the sensitivity to the verification result of the parameter. Therefore, partial factors are set for each performance verification method for each facility. Also, for each individual performance verification, it is difficult to separately take into account the extent of variation of the ground parameters that depends on ground investigations or the soil test methods. Therefore, the characteristic values are calculated by applying a correction corresponding to the reliability of the soil test method. This approach is a device for simplifying the performance verification method by making the partial factors set for each performance verification method for each facility independent of the ground investigation methods and soil test methods. However, it is slightly different from the concepts of JGS4001 that makes "the characteristic value is the average value of the derived values" as a principle.

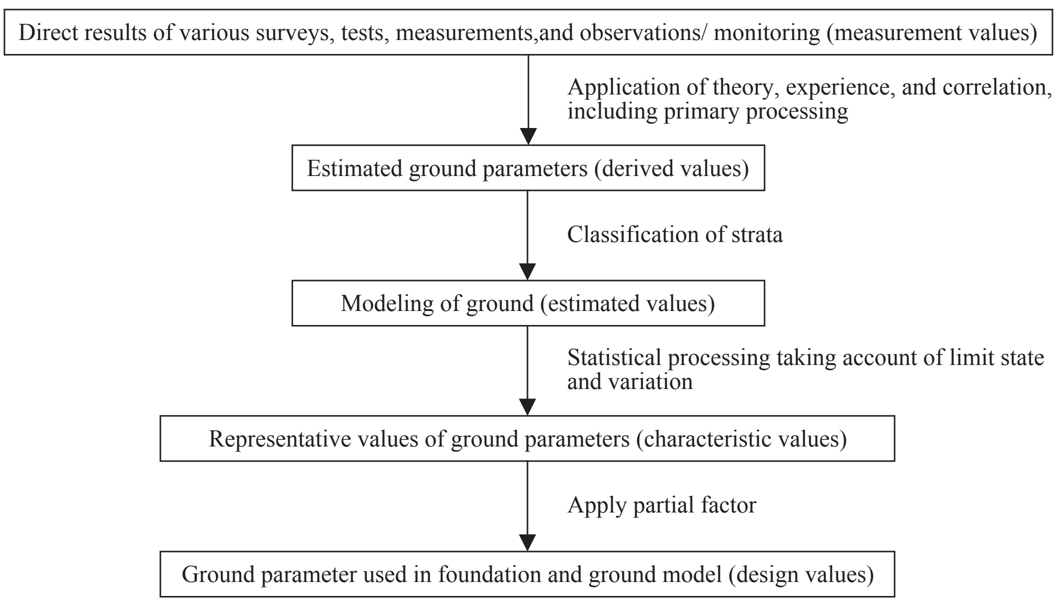

Fig. 2.1.1 Example of Procedure for setting the Design Values of Ground Parameters [1)]

(2) Methods of Estimating the Derived Values

As shown below, methods of estimating derived values include the method of using the measured values as they are as the derived values, the method of applying primary processing only to obtain the derived values, and the method of obtaining the derived values by converting measured values into different engineering quantities.

① The method of using the measurement values as they are as derived values is, literally, direct measurement of the ground parameters.

② Within the method of applying primary processing only to obtain the derived values, the primary corrections are an area correction for shear tests, a correction for the effect of strain rate on the shear strength, and the simple correction corresponds to just multiplying by the coefficients. Also, applying simple processing to test results, such as applying the primary processing to calculate the water content w, the wet density ρ_t, the soil particle density ρ_s, grain size distribution, obtaining the deformation modulus E from the stress-strain relationship, and obtaining the consolidation yield stress p_c from the e-log p relationship, corresponds to this method.

③ The method of obtaining derived values by converting the measured values into different engineering quantities is the method of converting the measured results into engineering quantities based on theoretical or empirical equations, or, obtaining fitting parameters in accordance with theory. Converting N-values into the angle of shear resistance ϕ using empirical equations, and fitting theoretical curves of consolidation to settlement-time curves to obtain the coefficient of consolidation c_v, correspond to this method.

(3) Methods of setting the Characteristic Values

① General
The characteristic values are set generally in accordance with the flow shown in **Fig. 2.1.2**.
If there is a sufficient number of the derived value data to carry out statistical processing, and if the variation in the derived values is small, as a rule, the characteristic value may be calculated as the average value, expected value, of the derived values. Here, if the number of data entries n of the derived values is 10 or more, and the amount of variation is not large, and if the coefficient of variation CV is less than 0.1, it is considered that a certain reliability can be guaranteed for the statistical results, and the average value, expected value, of the derived values may be taken to be the characteristic value. However, if there are an insufficient number of data entries of the derived values to carry out statistical processing and if the variation in the derived values is large, it is necessary to set the characteristic value by correcting the average value, expected value, of the derived values based on the method shown below.

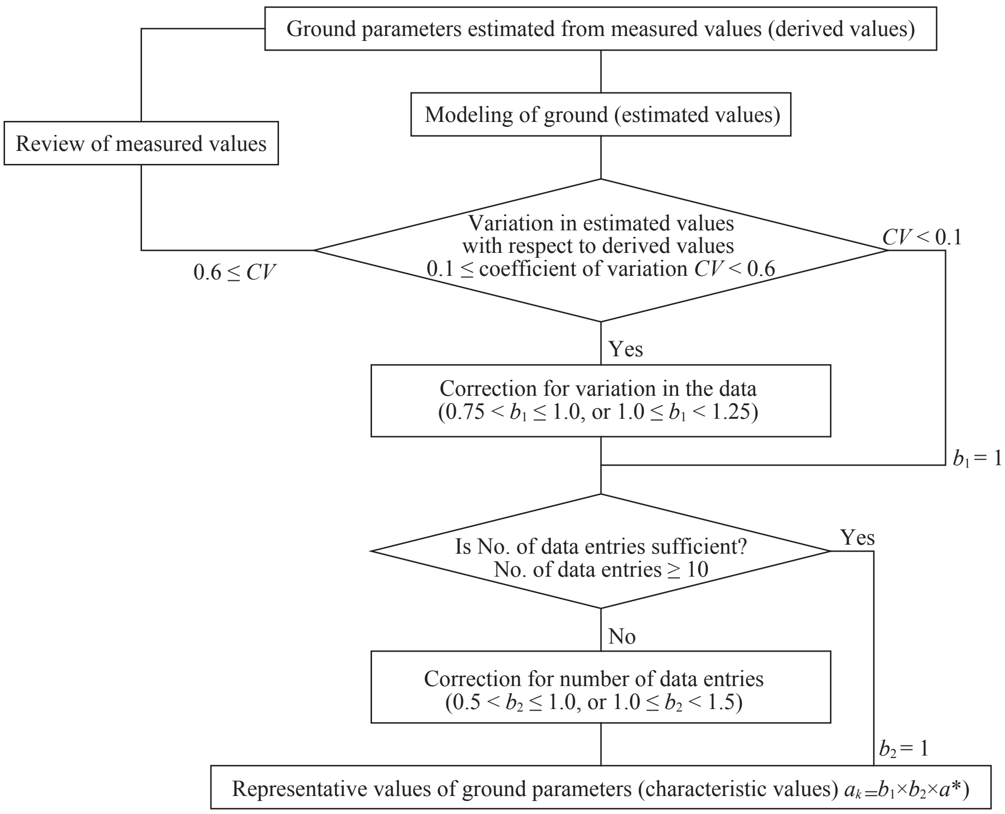

Fig. 2.1.2 Example of Procedure for setting Characteristic Values of Ground Parameters

② Correction of the average value ,expected value, of the derived values
When the number of derived value data entries is limited, or the variation in the derived values is large, the characteristic values cannot simply and automatically be taken to be the average value, expected value, of the derived values, but it is necessary to appropriately set the characteristic values taking into consideration the estimated error of the statistical average values. In this case the following method may be used. The uncertainty factors in the characteristic values include errors in the ground investigation or soil tests, estimation errors in the derived values, and inhomogeneity in the ground itself. Therefore, it is desirable that the ground investigation conditions such as types of survey equipment, soil test conditions such as types of test equipment, test methods and condition of test specimen, the soil stratigraphy, and other soil information need to be carefully examined. The method of correction of the average value ,expected value of the derived values described here is not limited to the values for stability verification of the facility, but it is supposed that it can be generally applied to ground constants, including values used for settlement predictions. In JGS4001 a method of setting the characteristic values in accordance with confidence levels is described, in which a normal distribution is

assumed if the standard deviation of the population is known, and a *t*-distribution is assumed if the standard deviation is not known. However, when dealing with ground parameters, the distribution and variation in the derived values are due to errors in the ground investigation or soil tests, estimation errors in the derived values, and inhomogeneity in the ground itself, and hence, this is different from dealing with quality indices of factory products, and simple statistical processing is hardly applicable.

To obtain the ground parameters, which are obtained by adjusting the average values for the statistical errors, and correspond to the characteristic values, for reliability-based design, it is necessary to obtain a sufficient number of test results for statistical processing. Also, in order to reflect the soil investigation and soil test results in the performance verification, it is necessary to model the distribution in the depth direction of the estimated values a^* of the ground parameter a as constant with depth ($a^*=c_1$), linearly increasing with depth ($a^*=c_1z+c_2$), or as having a quadratic distribution with depth ($a^*=c_1z^2+c_2z+c_3$). Here, c_1, c_2, and c_3 are constants. If a certain range of depth is to be modeled, a sufficient number of tests are 10 or more data entries in order to carry out statistical processing on the ground model. The reliability of ground parameters obtained from different soil test methods such as the undrained shear strength of cohesive soils obtained from triaxial tests and unconfined compression tests differs, so different partial factors should be set accordingly, but it is not known to what extent the factors should differ. However, it is well known that the coefficients of variation of the two test results are significantly different. Based on this the characteristic values are calculated not simply as the arithmetic mean, but by multiplication by a correction coefficient that takes into account the variation of the derived values to the estimated values. However, this is based on the assumption that there is a sufficient number of data entries to carry out statistical processing, so if the number of data entries is insufficient, it is necessary to further set the characteristic values on the safety side , by multiplying by a correction coefficient for the number of data points. In other words, the characteristic values are calculated from the following equation **(2.1.1)** or equation **(2.1.2)**. Here, if it is reasonable to consider the variation on logarithmic axes, equation **(2.1.2)** is used.

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

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

where

a_k : representative value of ground parameter (characteristic value)
b_1 : correction coefficient for variation in the derived values
b_2 : correction coefficient for number of data points of derived values
a^* : model value of the ground parameter (estimated value)

A specific method of setting the correction coefficient is described below. However, when dealing with the unit weight of the in- situ ground in stability analysis, for determining the values at which the action side and the resistance side are substantially in balance, the correction coefficients may be taken to be b_1=1, b_2=1.

③ Method of setting the correction coefficient for variation in the derived values

If the estimated parameter for modeling the distribution of test results is represented by a^*, when considering the variation in test results a, it is convenient to use the standard deviation of (a/a^*) which refers to the coefficient of variation. Here it is assumed that a^* is estimated as the average value of a uniform distribution within a stratum that is modeled, or a distribution in which errors are minimized by the least squares method or similar. It is known that for a uniform ground, the coefficient of variation of the ground parameters obtained as a result of taking undisturbed clay test samples using a fixed piston type thin-walled tube sampler, and carefully carrying out each type of soil test, is 0.1 or less. In other words, even though it is a uniform ground, there is a certain amount of non-uniformity, and there are errors caused by the soil test methods, so this extent of variation in the results is inevitable. However, if the variation is greater, if the non-uniformity in the ground is large, if the disturbance during sampling is large, if the soil test methods are inappropriate, or if the modeling with respect to depth is inappropriate, the estimated values a^* cannot be taken to be characteristic values as they are, but it is necessary to set the characteristic values on the safety side, taking the uncertainty factors into account.

Therefore, the correction coefficient b_1 for variation of the derived values is set corresponding to the coefficient of variation *CV* defined as the standard deviation *SD* of (a/a^*). When the parameter a is contributing to the resistance side such as shear strength, in a performance verification, the correction coefficient $b_1=1-(CV/2)$, and when contributing to the action side such as unit weight of an embankment, and compression index, $b_1=1+(CV/2)$ are set, and the values shown in **Table 2.1.1** should be used in the performance verification. This corrects the value to a value corresponding to about 70% probability of non-exceedance, for use as the characteristic value. If the coefficient of variation is 0.6 or higher, the reliability is poor, so performance verification cannot be carried out, interpretation of the test results must be carried out again, and if necessary the modeling of the ground must be re-investigated. In certain cases it may be necessary to carry out the soil investigation again.

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	1.0	1.0
≥ 0.1, < 0.15	0.95	1.05
≥ 0.15, < 0.25	0.9	1.1
≥ 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	

The ground parameters include parameters whose results are evaluated as logarithmic distributions, such as the consolidation yield stress p_c, the coefficient of consolidation c_v, and the coefficient of volume compressibility m_v. In order to obtain the characteristic values of these parameters, several tests are carried out, and if the ground is to be treated as uniform, these parameters are distributed as log-normal, so it is reasonable to consider the variation on the logarithmic axis. In other words, for the parameter a, if the standard deviation of (log a)/(log a*) is SD, and this becomes the coefficient of variation CV, the values in **Table 2.1.1** can be used as they are as the correction coefficient b_1 on the logarithmic axis. On the other hand, for the angle of shear resistance ϕ, the variation of ϕ itself is not considered, but the variation of tan ϕ is considered. In the case of the angle of shear resistance of a mound material, if the value used in the performance verification is specified based on experience, the specified value already has the effect of variation taken into consideration, so it is not necessary to consider a correction coefficient. The correction coefficients shown here are used after carrying out statistical processing in order to obtain the characteristic values from the reported soil test results. Therefore, it is necessary to be aware that the coefficients of variation in **Table 2.1.1** do not indicate the level of variation obtained from soil investigations or soil test results.

④ Method of setting the correction coefficient for the number of data entries of the derived values
For the **Method of setting the correction coefficient for variation in the derived values** in ③ above, it was assumed that the number of data points is sufficient to carry out statistical processing. However, in the case where the number of data points is insufficient for carrying out statistical processing, the correction coefficient b_2 for the number of data entries of derived values is set as follows. In other words, if the number of data entries n is 10 or more, there will be a certain reliability in the statistical results, but if the number is insufficient the correction coefficient should be set to $b_2=\{1\pm(0.5/n)\}$. Here the negative sign is used when it is necessary to correct the characteristic value of a ground parameter used in performance verification toward smaller values than the derived values, and the positive sign is used when it is necessary to correct the value toward larger values than the derived values. For the performance verification there must be at least two or more data entries. However, even in the case where there is only one data entries, if other parameters for example N- value or grain size distribution have been obtained, and if the distribution in the depth direction is modeled from a correlation with these provided only commonly known correlations are used, then that one data entry may be used in the performance verification. In this case, $b_1=1$, and $b_2=1\pm0.5$ are assumed.

⑤ Method of setting the characteristic values taking the mode of the performance verification into account
The ground constants for consolidation and the ground constants for shear are not mutually independent. In the performance verification, if these constants are considered to be independent, the characteristic values can be obtained taking into consideration the reliability of the respective parameters. However, if a strength increase due to consolidation is expected for stability evaluation, the parameters in respect of consolidation and the parameters in respect of shearing must be closely linked. In these circumstances, in the process of obtaining characteristic values from derived values, the parameters are modeled as mutually linked when modeling the distribution of soil test results to derive estimated values. For example, the characteristic values must be set by statistical processing for the variation, to estimate compatible ground parameters, taking into consideration the relationship $c_u=m\times OCR\times\sigma'_{v0}$ between the effective soil overburden pressure σ'_{v0}, the consolidation yield stress p_c, and the undrained shear strength c_u, using the strength increase ratio $m=c_u/p_c$, and the overconsolidation ratio $OCR=p_c/\sigma'_{v0}$.

(4) Method of Calculating Design Values
In the various calculations when the ground parameters are used in performance verification, design values are obtained by multiplying the characteristic values by a partial factor γ. A value of the partial factor γ may be set for each performance verification method for each facility, but if not specified otherwise, γ may be taken to be 1.0.

2.2 Physical Properties of Soils

2.2.1 Unit Weight of Soil

(1) The unit weight must be obtained by collecting undisturbed samples on site, or directly obtaining it on site.

(2) The unit weight is normally the weight per unit volume in air, and includes 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 immersed unit weight. For the measurement of the unit weight, methods of collecting undisturbed samples of clay soils have been established, and it is possible to obtain test samples that are representative of the soil in-situ. Therefore the unit weight of clay soils can be obtained from laboratory tests. However, the unit weight of sandy soils or sand must be obtained directly in-situ.

The wet unit weight is one of the indices indicating the fundamental properties of a soil, and is used for recognizing the soil stiffness, and degree of looseness, and for calculating, the weight of a soil mass and the void ratio.

① Wet unit weight

The wet unit weight is generally expressed as shown in equation **(2.2.1)**, by combining both the weight of soil particles per unit volume and the weight of water within the void.

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

where

γ_t : wet unit weight (kN/m³)
ρ_t : bulk density (t/m³)
ρ_s : soil particle density (t/ m³)
e : void ratio
S_r : degree of saturation (%)
w : water content (%)
ρ_w : density of seawater (t/m³)
g : gravitational acceleration (m/s²)

The approximate values of the unit weight of soils normally encountered in harbor areas in Japan are as shown in **Table 2.2.1**.

Table 2.2.1 Unit Weight and Water Content of Representative Soils

	Holocene clays	Holocene 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

② Dry unit weight

Only soil particles are considered in the unit weight, so by putting w=0 or S_r=0, the dry weight per unit volume is expressed by equation **(2.2.2)**.

$$\gamma_d = \rho_d g = \frac{\rho_s g}{1+e} \qquad (2.2.2)$$

where

γ_d : dry unit weight (kN/m³)
ρ_d : dry density (t/m³)

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

$$\gamma_d = \frac{\gamma_t}{1+\frac{w}{100}} \qquad (2.2.3)$$

③ Immersed unit weight
If the void is completely saturated with water, the immersed unit weight is expressed by equation **(2.2.4)** taking buoyancy into account.

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

where
γ' : immersed unit weight (kN/m³)
γ_{sat} : saturated unit weight (kN/m³)

Although the unit weight of water γ_w is somewhat dependent on salt concentration and temperature, its correct value is known. Therefore, when obtaining the characteristic values of a saturated foundation taking the variation in unit weight into account, the variation in γ' should be considered, not γ_{sat}. In other words, when multiplying the characteristic values by a partial factor to obtain the design values, there is no necessity to apply a partial factor to the unit weight of water γ_w, so the immersed unit weight γ' is multiplied by the partial factor, not the saturated unit weight γ_{sat}.

(3) Measurement of Unit Weight In-situ
Methods for directly obtaining the unit weight in- situ include methods in which measurement is only possible near the ground surface, and methods of measurement in firm ground. The former includes for example the so-called sand replacement method, a simple and easy method as prescribed by JIS A 1214 **Method of Soil Density Test by Sand Replacement Method**. Also, the latter includes example methods of measurement using radioisotopes (RI).

① Methods using the sand replacement method
The sand replacement method is mainly applied to measurement on land near the ground surface for control of earthworks, but it can be used down to a certain depth where pits can be excavated. This measurement method is described in JIS A 1214.

② Radioisotopes (RI)
In recent years the use of RI has become comparatively easy, and although there are strict laws and regulations such as **the Law to Prevent Radiological Hazards Caused by Radioactive Isotopes**. (Law No. 167, 1957) and its associated regulations, there have been many cases of measurement using a γ -ray densitometer as an in- situ test where it is difficult to obtain undisturbed samples of sand and sandy soil. Incidentally, these legal restrictions do not apply in the case of sealed radioactive sources whose radiation source strength is 3.7MBq (megabequerel) or less.

There are two types of γ -ray densitometer that use RI: a surface type and an inserted type, and these are described in **Soil Density Test Methods using RI Equipment**, JGS 1614, the Standard of Geotechnical Society of Japan. The surface type is applied to measurement near the ground surface, as implied by its name, and is used for control of earthworks same as the sand replacement method. The surface type is further classified into back scattering types and transmission types. Measuring equipment using the initially developed back scattering method is frequently used, but in recent years equipment using the transmission method has become popular because of its accuracy. On the other hand, the insertion type is applied to measuring the density distribution in the vertical direction, in other words for surveys in the depth direction. For example, it is used for investigating the density distribution in the depth direction for ground surveys, for determining the soil improvement effect by density measurement of replaced sand, and measurement of the density of filled sand in caissons.

The RI method has the advantage that it is a non-destructive test from which the in- situ density can be directly measured. Also, although the measurement operation itself is simple so it has a high usability value, on the other hand because there is danger associated with the radioactive material there are many regulations regarding its handling, so it cannot be simply brought out and used in-situ. In addition, in surveys associated with port construction, the inserted type is mainly used, so there is an operation of inserting the equipment into the access pipe. The measurement accuracy is governed by the material and quality of the pipe, or the insertion accuracy, in other words, the measurement accuracy is governed by the disturbance of the surroundings when the pipe is inserted, and how good the contact between the pipe and the soil is, so caution is necessary. Recently the RI cone penetrometer, which incorporates RI in a cone probe, is being developed as a device capable of directly penetrating into the ground for surveys.

(4) Relative Density
The degree of compaction of sand may be expressed by the relative density using equation **(2.2.5)**.

$$D_r = \frac{e_{\max} - e}{e_{\max} - e_{\min}} = \frac{\rho_d - \rho_{d\min}}{\rho_{d\max} - \rho_{d\min}} \frac{\rho_{d\max}}{\rho_d} \tag{2.2.5}$$

where,

D_r : relative density
e_{max} : void ratio in the loosest state
e_{min} : void ratio in the densest state
e : void ratio in the present state of the test sample
ρ_{dmin} : dry density in the loosest state (g/cm³)
ρ_{dmax} : dry density in the densest state (g/cm³)
ρ_d : dry density in the present state of the test sample (g/cm³)

The density of sand is greatly affected by the shape of the particles and by the grain size composition. So from the unit weights and the void ratios calculated from it, the density of sand cannot be correctly evaluated. Therefore, the relative density is used to indicate the relative value within the range of void ratios that can be taken with this soil. Measurement of e_{max}, e_{min}, (ρ_{dmin}, ρ_{dmax}) for obtaining D_r can be carried out in accordance with Japanese Industrial Standard JIS A 1224 **Method of Measurement of the Minimum and Maximum Density of Sand**.

It is difficult to take undisturbed samples of sand, so the relative density is frequently measured indirectly by sounding. (see **2.3.4(4) Angle of shear resistance of sandy ground**).

2.2.2 Classification of Soils

(1) Soil classification is performed by the grading for coarse soils and by the consistency for fine soils.

(2) Mechanical properties of soil such as strength or deformation have a close relationship with the grading for coarse soils, and with the consistency for fine soils.

(3) Engineering Classification Method for Subsoil Materials (Japanese Unified Soil Classification System)
The classifying method of soil and rock, and their nomenclature should be in accordance with the engineering classification method for subsoil material prescribed by the JGS 0051 **Japanese Unified Soil Classification System** of the Geotechnical Society of Japan. The grain size classifications and their names are shown in **Fig. 2.2.1**. The coarse-grained soil refers to soil composed mainly of coarse fraction with a grain size ranging from 75 μm to 75 mm. Soil consisting of components with a grain size less than 75 μm is called the fine-grained soil. **Fig. 2.2.1** and **Fig. 2.2.2** show the engineering classification system for soil, and **Fig. 2.2.3** shows the plasticity diagram used in classifying fine-grained soil.

Particle Diameter

5μm | 75μm | 250μm | 425μm | 850μm | 2mm | 4.75mm | 19mm | 75mm | 300mm

Clay	Silt	Fine sand	Medium sand	Coarse sand	Fine gravel	Medium gravel	Coarse gravel	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. 2.2.1 The Grain Size Classifications and their Names (JGS 0051)

(4) Classification by Grain Size
The uniformity coefficient is an index showing the grain size characteristics of sandy soil and is defined by equation (**2.2.6**).

$$U_c = D_{60}/D_{10} \tag{2.2.6}$$

where

U_c : uniformity coefficient
D_{60} : grain size corresponding to 60 % passing by mass in grain size distribution curve (mm)
D_{10} : grain size corresponding to 10 % passing by mass in grain size distribution curve (mm)

A large uniformity coefficient means that the grain size is broadly distributed, and such a soil is called "well graded". In contrast, a small value of *Uc* means that the grain size distribution is narrow or the grain size is uniform. Such a soil is called "poorly graded". In the Japanese Unified Soil Classification System, coarse soil where fine contents are less than 5% of the total mass is further divided into "broadly-distributed soil" and "uniformed soil".

Broadly-distributed soil : $10 \leq U_c$
Uniformed soil : $U_c < 10$

2.2.3 Hydraulic Conductivity of Soil

(1) When the seepage flow in a completely saturated ground is a steady laminar flow, the hydraulic conductivity shall be estimated by using Darcy's law.

(2) The hydraulic conductivity *k* is calculated by equation (2.2.7), taking into account of the measurement of cross-sectional area of soil A, hydraulic gradient *i* and volume of seepage flow in unit time.

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

where

k : coefficient of permeability (cm/s)
q : volume of water flow in soil in unit time (cm^3/s)
i : hydraulic gradient, $i = \frac{h}{L}$
h : head loss (cm)
L : length of the seepage path (cm)
A : cross-sectional area (cm^2)

The measurement for determining coefficient of permeability *k* includes a laboratory permeability test of undisturbed soil samples taken in-situ, or a in-situ permeability test.

(3) Approximate values of the coefficient of permeability
Hazen showed that the effective grain size D_{10} and the permeability of sand k are related, and gave equation (2.2.8) to calculate *k* of relatively uniform sand with the uniformity coefficient of U_c less than 5, and the effective grain size D_{10} from 0.1 mm to 0.3 mm.[5)]

$$k = CD_{10}^{\ 2} \tag{2.2.8}$$

where

k : coefficient of permeability (cm/s)
C : constant (C=100 (1/cm · s))
D_{10} : grain size called as the effective grain size corresponding to 10 percentage passing of mass in grain size distribution curve (cm)

Terzaghi has pointed out that equation (**2.2.8**) can also be applied to cohesive soils by using $C \fallingdotseq 2$. The approximate values of the coefficient of permeability are listed in **Table 2.2.2**).[5)]

Table 2.2.2 Approximate Values of Coefficient of Permeability [5)]

Soil	Sand	Silt	Clay
Hydraulic conductivity	10^{-2}cm/s	10^{-5}cm/s	10^{-7}cm/s

2.3 Mechanical Properties of Soil

2.3.1 Elastic Constants

(1) When analyzing soil behavior as an elastic body, the elastic constants are determined with due consideration for the nonlinearity of stress-strain relation of soils.

(2) When analyzing soil behavior as an elastic body, the deformation modulus and Poisson's ratio are normally used as the elastic constants. Because of the strong nonlinearity of stress-strain relationship of soil, the elastic constants in analysis must be determined by considering the strain level of the ground to be analyzed.

(3) Strain Dependency of Deformation Modulus
The stress-strain relation of soil usually shows a strong nonlinearity. When the strain level is within a range of 10^{-5} or less namely 0.001% or less, the deformation modulus is largest and nearly constant. This maximum value E_{max} is corresponding to the measured value in the dynamic testing methods such as the elastic wave exploration, and is called the dynamic elasticity modulus. As the strain level increases, the elasticity modulus decreases. The secant modulus E_{50}, determined from a conventional unconfined compression test or a triaxial compression test, is considered as the deformation modulus when the strain is of the order of 10^{-3} (0.1%). When conducting an elastic analysis of soil, it is necessary to determine the elastic constant by considering the strain level of the soil.

(4) Relationship between Undrained Shear Strength and Deformation Modulus
For cohesive soils, the approximate values for the initial tangent elastic modulus E_i, and the secant elastic modulus E_{50} can be determined by using equation (**2.3.1**) and equation (**2.3.2**).[7)]

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

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

where

E_i : initial tangent elastic modulus (kN/m^2)
E_{50} : secant elastic modulus (kN/m^2)
c_u : undrained shear strength (kN/m^2)

The equation (**2.3.1**) is applicable only for highly structured marine cohesive soil with high plasticity.

(5) Poisson's Ratio
For determining Poisson's ratio of soil, there is no established method currently, although a number of methods have been proposed. Practically, ν = 1/2 is used for undrained conditions of saturated soil, and ν = 1/3 – 1/2 is used for many other situations.

2.3.2 Compression and Consolidation Characteristics

(1) Compression characteristics of soil and the coefficients for estimating settlement of foundations due to consolidation can be calculated from the values obtained based on JIS A 1217 **Test Method for Consolidation Test of Soils Using Incremental Loading**.

(2) When soil is loaded one-dimensionally, compression of the structure with the soil particles which causes settlement is referred to as compression. If the voids of the soil are saturated with water, it is necessary for the pore water to be drained in order to contact the structure with the soil particles. For sandy soils with high hydraulic conductivity, drainage is fast, so contraction occurs immediately after loading and is soon completed. However, for cohesive soil ground the hydraulic conductivity is very low, so a long period of time is needed for drainage, and compression settlement occurs slowly. This phenomenon in which compression settlement in cohesive soil ground occurs over a long period of time is referred to as consolidation.
The consolidation characteristics of soils are used not only for calculating the settlement due to loading, but also for estimating the increase in shear strength of soils in soil improvement work.

(3) Calculation of the final settlement due to consolidation
When the consolidation pressure and the void ratio when consolidation is completed at that pressure (after 24 hours) in a consolidation test are plotted on semi-logarithmic graph, the so-called e–log p curve or compression curve is obtained, as shown in **Fig. 2.3.1**. The "abc" portion of the e–log p curve indicates the loading process, and is virtually linear. The consolidation state indicated by the "abc" portion is referred to as the normal consolidation state. On the other hand, if the soil is unloaded from the state at point "b", the relationship between the void ratio and the pressure when the equilibrium state is reached under the reduced pressure describes the path "bd". If the pressure is increased again, the path "db" is described. The state represented by "bd" and "db" is referred

to as the overconsolidation. When a consolidation test is carried out, the path "d→b→c" is described, the point "b" is obtained at the boundary of "d→b" indicating the elastic deformation and "b→c" indicating the plastic deformation, and the pressure corresponding to this boundary is referred to as the consolidation yield stress.

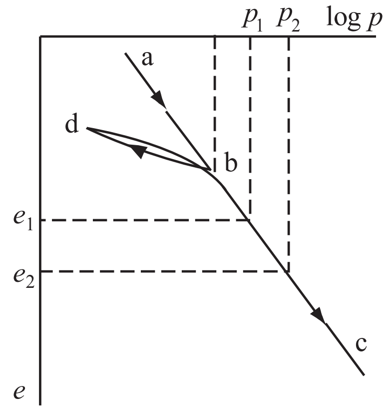

Fig. 2.3.1 *e*–log *p* Relationship during Consolidation

The relationship between the void ratio e and the pressure p for the segment "abc", normal consolidation domain in **Fig. 2.3.1** is expressed by equation **(2.3.3)**.

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

where

C_c is a non-dimensional number showing the degree of inclination of segment "abc" and is called the compression index.

The final settlement resulting from the consolidation load can be calculated using three methods: the e-log p curve method, the C_c method, and the coefficient of volume compressibility m_v method.
The decrease in void ratio Δe when the pressure increases from the overburden pressure in- situ p_0 to $(p_0+\Delta p)$ can be determined by directly reading the e–log p relationship curve obtained from consolidation tests. Otherwise, if the settlement is expected to be overestimated to the safe side, it can also be evaluated by equation **(2.3.4)** using equation **(2.3.3)**.

$$\Delta e = e_{p_0} - e_{p_0+\Delta p} = C_c \log_{10} \frac{p_0 + \Delta p}{p_0} \tag{2.3.4}$$

In the e-log p curve method, the settlement S is calculated by the following equation using Δe either read directly or determined from equation **(2.3.4)**:

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

where

h : thickness of layer

In the C_c method, the settlement S is calculated by the following equation **(2.3.6)**:

$$S = h \frac{C_c}{1 + e_0} \log_{10} \frac{p_0 + \Delta p}{p_0} \tag{2.3.6}$$

This equation corresponds to that whereby equation **(2.3.4)** is substituted in equation **(2.3.5)**.
The coefficient of volume compressibility m_v is used for estimating settlement and the amount of compression by a load is proportional to m_v. However, this is effective only with small increases in consolidation pressure such as where m_v can be assumed to be constant, because it would linearize the soil with strong nonlinearity. Equation **(2.3.7)** is used to calculate the settlement S using m_v.

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

where

m_v : coefficient of volume compressibility when the consolidation pressure is $\sqrt{p_0 \times (p_0 + \Delta p)}$

Generally, the value of m_v during consolidation decreases with the increase of effective overburden pressure. Under normally consolidated state, the relationship between p and m_v plotted on a double logarithmic graph would almost be a straight line. The m_v used in equation **(2.3.7)** for calculating settlement is the mean value during the change in effective overburden pressure of the ground from p_0 to ($p_0+\Delta p$). Usually, this would be the m_v for the geometric mean of the effective overburden pressure ($\sqrt{p_0 \times (p_0 + \Delta p)}$).

(4) Settlement Rate

In Terzaghi's theory which is a classical theory of consolidation, the method of analyzing the settlement rate is as follows: When a pressure increment p is added to a saturated cohesive soil under undrained conditions, an excess pore water pressure equal to the magnitude of p is generated. As consolidation progresses, this excess pore water pressure gradually dissipates, and at the same time the stress σ' acting between soil particles increases. This stress is referred to as the "effective stress". However, the sum of the excess pore water pressure u and the increment of stress σ' between soil particles is always equal to the increment of loading pressure p, so equation **(2.3.8)** is established.

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

Consider the case where highly permeable sand layers exist above and beneath a clay layer of thickness $2H$. When a consolidation pressure increment p is applied, the distribution with depth of σ' and u are as shown in **Fig. 2.3.2**. In other words, at the time of start of consolidation (t=0), the state is indicated by the line DC with u=p, σ'=0, and when consolidation is completed the state is as indicated by the line AB, with u=0, σ'=p. The curve AEB is the pore water pressure distribution at the time t_1 after start of consolidation. This curve is called "isochrone". As shown in the figure, the parts of soil distant from the drainage layers have relatively slow rate of consolidation.

The ratio of the effective stress increment to the consolidation pressure increment (σ'/p) at a certain depth z is referred to as the degree of consolidation U_z at that depth. The degree of consolidation at each depth averaged over the whole layer is referred to as the average degree of consolidation U. The average consolidation is the ratio of the area of AEBCD to the area ABCD in **Fig. 2.3.2**.

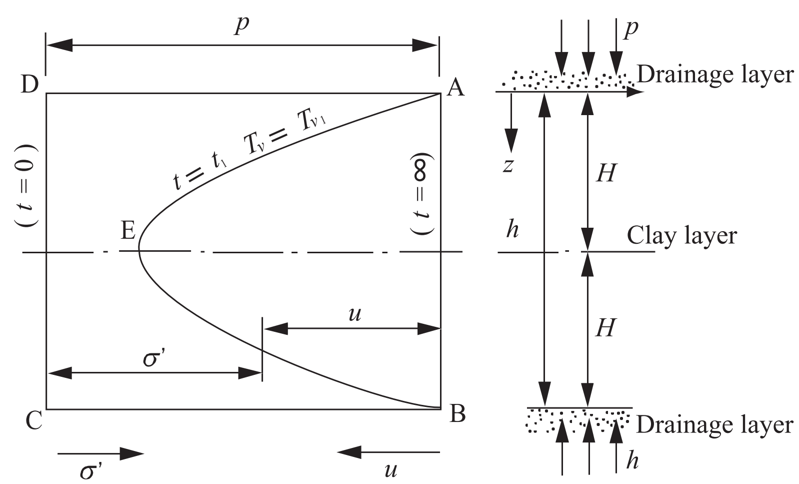

Fig. 2.3.2 Distribution of Pore Water Pressure with Depth

The consolidation is the time-dependent settlement phenomenon. The rate of consolidation for an entire cohesive soil layer is represented with the parameter U for the average degree of consolidation. The relationship between U and the non-dimensional time factor T_v is obtained by the theory of consolidation. The relationship between the non-dimensional time factor T_v and the actual time t is shown by the following equation:

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

where

T_v : time factor

c_v : coefficient of consolidation

t : time after the consolidation starts

H^* : maximum drainage distance

When the permeable layer exist at both sides of the cohesive soil layer, the maximum drainage distance H^* is the same as H. However, when the permeable layer only exists on one side, H^* is equal to $2H$. The degree of consolidation at each depth is shown by the consolidation isochrones in **Fig. 2.3.3**. Furthermore, **Fig. 2.3.4** shows the theoretical relationship between the average degree of consolidation and the time factor.

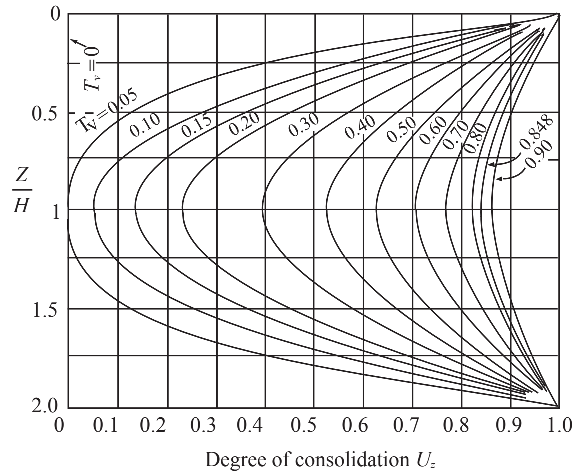

Fig. 2.3.3 Consolidation Isochrones

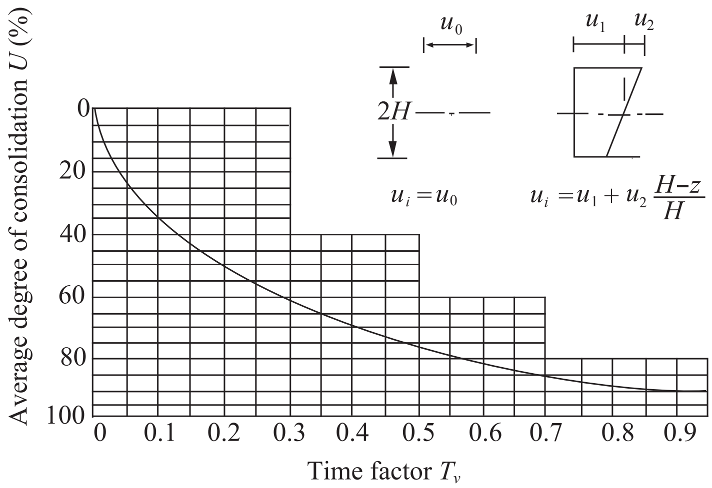

Fig. 2.3.4 Theoretical Relationship between Average Degree of Consolidation and Time Factor

(5) Primary Consolidation and Secondary Consolidation

If the relationship between amount of settlement and time measured in a consolidation test is shown as degree of consolidation against time, **Fig. 2.3.5** is obtained. However, as shown in the figure, at the final stage of consolidation, the test curve does not coincide with the theoretical curve. Consolidation until U=100% as determined by the time settlement relationship virtually agreeing with consolidation theory is referred to as “primary consolidation”, and the part in which U>100% and consolidation is not in accordance with consolidation theory is referred to as “secondary consolidation”. Secondary consolidation is considered to be a creep phenomenon, and in this case the settlement tends to occur linearly with respect to the logarithm of time.

In the performance verification of port facilities, normally the consolidation pressure due to loading reaches several times the consolidation yield stress of the ground. Under these conditions, the amount of settlement due to primary consolidation is large, and the amount of settlement due to secondary consolidation is comparatively small, so in most cases secondary consolidation is not considered when carrying out the performance verification. Also, if the settlement is large, the effect of the increase in buoyancy with settlement cancels out the effect of secondary consolidation, so apparently secondary consolidation is not seen. In the following cases, secondary consolidation must be taken into consideration at the performance verification.

① The advancement in ground settlement with elapse of time subsequent to construction is having serious effects on the facility.

② As in the case of deep Pleistocene clayey ground, when the consolidation pressure does not exceed the consolidation yield stress of the soil layer significantly, the contribution of secondary consolidation in an entire settlement can not be neglected.

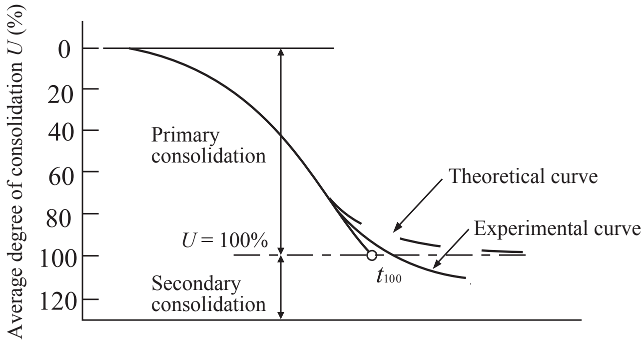

Fig. 2.3.5 Primary Consolidation and Secondary Consolidation

(6) Consolidation Settlement of Very Soft Cohesive Soils

When the landfill is carried out with dredging or disposed sludge, it is necessary to predict the consolidation settlement of extremely soft deposits. Mikasa's [8] consolidation theory which takes into account the effect of self weight of clay layer and the changes in layer thickness during consolidation can be applied to analyze this problem. . In this case, the amount and speed of settlement cannot be determined analytically; it must be calculated with the finite differential method.

When the reduction in thickness of a layer due to settlement compared with the original thickness is so large that it cannot be ignored, the errors in the normal consolidation settlement calculation method become large. For example, if the reduction in the layer thickness is 10 to 50%, the difference between the normal calculation method and a calculation that takes into consideration the effect of the change in layer thickness is in the region 3 to 30%. Also, the effect of dead weight is largest after allowing to stand after dredging and filling, and as the load increases, the effect is reduced relatively. For loading that is twice or more the average own weight of a weak layer, the effect of self weight becomes very small, and can be virtually ignored.

In order to estimate the consolidation parameters of very soft cohesive soil, there is a constant rate of strain consolidation test in which displacement is continuously applied as stipulated in JIS A 1227 **Test Method for One-dimensional Consolidation Properties of Soils using Constant Rate of Strain Loading**. For cohesive soils with a large aging effect, or for cohesive soils of which settlement can be suddenly seen after the consolidation yield stress, the constant rate of strain consolidation test from which a continuous e–log p curve can be obtained is a very useful method for obtaining the consolidation yield stress [9] However, the e–log p curve is strongly affected by the strain rate, and the e–log p curve obtained from this test is normally greatly shifted to the large consolidation pressure side compared with the e–log p curve obtained from an icremental loading consolidation test as stipulated in JIS A 1217. Therefore it is necessary to be aware that the consolidation yield stress becomes larger.

(7) Correlation between the Compression and Consolidation Coefficients and the Physical Properties

Of all the soil tests, the consolidation test requires the longest amount of time. If the results of consolidation test can be estimated from physical test results, which require only disturbed test samples, and is a comparatively simple test method, and moreover whose results can be quickly obtained, this would be very useful. Skempton has proposed the correlation equation **(2.3.10)** as the relationship between the compression index C_c and the liquid limit w_L.

$$C_c = 0.009(w_L - 10) \tag{2.3.10}$$

Equation **(2.3.10)** is applicable to clay that is re-molded and re-consolidated in the laboratory, or young clay ground formed by artificial filling, but it tends to either over or underestimate the compression characteristics of naturally deposited clays.

The reason why natural cohesive soil grounds have larger compression index values than young clay is because in the process of sedimentation which occurs over many years, a structure is formed due to aging effects such as cementation. When this structure is destroyed as a result of the consolidation pressure exceeding the consolidation yield stress, high compressibility is demonstrated.

2.3.3 Shear Characteristics

(1) The shear strength parameters of soil are determined by classifying soil into sandy soil and cohesive soil. The shear strength of sandy soil is determined under drained conditions, while the shear strength for cohesive soil is determined under undrained conditions.

(2) In general, the hydraulic conductivity of sandy soil is 10^3 – 10^5 times that of cohesive soil. For sandy soil layer, the excess water in pores is considered to be completely drained during construction. For cohesive soil layer, on the other hand, almost no drainage is expected during construction because the hydraulic conductivity is significantly low. Thus in many cases for sandy soil layer the shear strength is evaluated using the angle of shear resistance in drained condition ϕ_D and the cohesion in drained condition c_D. Because the value of c_D is usually very small, practically c_D is ignored and only ϕ_D is used as the strength parameter.

In the case of saturated cohesive soil layer, the shear strength of the layer undergoes almost no change between before and after construction, as the drainage cannot take place during construction. The undrained shear strength before construction is therefore used as the strength parameter. For intermediate soil that has the permeability somewhere between those of sandy soil and cohesive soil, the soil should be viewed as sandy soil or cohesive soil based on the coefficient of permeability and construction conditions.

(3) Considerations on Shear Strength
The shear strength τ_f of a soil is generally expressed by the following equation.

$$\tau_f = c + \sigma \tan\phi \quad (2.3.11)$$

where

τ_f : shear strength
c : cohesion or apparent cohesion
ϕ : angle of shear resistance (°)
σ : normal stress on the shear surface

When a stress is applied to a soil, the stress acting on the skeletal structure of the soil particles, referred to as the effective stress, and the pore water pressure,) both change. If the total stress applied to the soil denotes σ, the effective stress denotes σ', and the pore water pressure denotes u, the following relationship can be established.

$$\sigma = \sigma' + u \quad (2.3.12)$$

$$\sigma' = \sigma - u \quad (2.3.13)$$

In equation **(2.3.11)**, the strength constants such as cand ϕ, vary depending on the conditions during the shear tests, but the condition that has the greatest effect is the drainage condition of the soil. Because soil has the tendency of changing volumes which is known as “dilatancy” while being sheared, shear strength of soil is greatly dependent upon whether a volume change takes place during the shear or not. The drainage condition is classified into the following three categories and different strength parameters are used for each case:

① Unconsolidated, Undrained condition (UU condition)

② Consolidated, Undrained condition (CU condition)

③ Consolidated, Drained condition (CD condition)

In **Fig. 2.3.6**, pattern diagrams are shown for the shear strength when direct shear tests are carried out under the drainage condition ①, ②, ③.[10] In the figure, the change in shear strength under increased or reduced normal stress σ is shown on the soil samples consolidated in advance to the pressure p_0. As shown in the figure, under the unconsolidated undrained condition ①, the strength is constant and does not depend on σ. In the case of the consolidated undrained condition ②, within the range $p_0 < \sigma$ the strength increases linearly as σ increases. Under the consolidated drained condition ③, the strength is overall greater than ①, ②, and this is because the void ratio is reduced by consolidation or shearing in the case of weak cohesive soil or loose sand. However, when σ is significantly smaller than p_0 (in the figure this limit of the normal stress is indicated as σ^*), the strength under the consolidated drained condition is smaller than the strength under the consolidated undrained condition due to the effect of swelling during shearing. Summarizing this relationship for the range of σ the following is obtained.

In the range $p_0 < \sigma$, namely the applied loading is larger than the pre-consolidation pressure ; ①<②<③

In the range $\sigma^* < \sigma < p_0$, namely the applied loading is somewhat smaller than the preceding consolidation

pressure ; ②<①<③ or ②<③<①
In the range $\sigma < \sigma^*$,namely the applied loading is significantly smaller than the preceding consolidation pressure ; ③<②<①

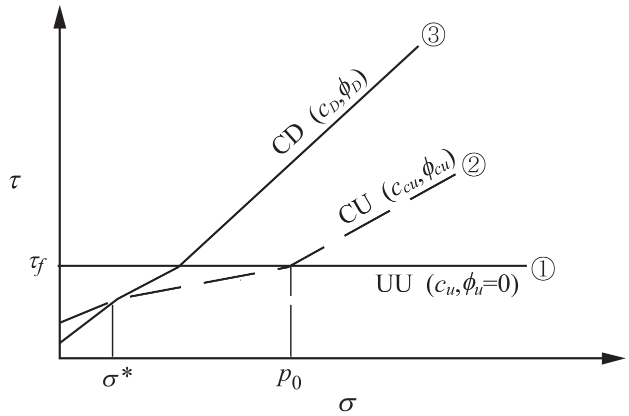

Fig. 2.3.6 Relationship between Drainage Conditions and Shear Strength

The shear strength used for the performance verification of ground should be the shear strength for the most dangerous drainage conditions expected under the given load. The drainage condition and shear strength are then as in the following:

(a) When loading takes place rapidly on the cohesive soil ground:
Because consolidation progresses and shear strength increases with the elapse of time, the most dangerous time will be immediately after the loading when almost no drainage has occurred. This is called the short-period stability problem. The shear strength τ_f at this time is the shear strength c_u that is determined from unconsolidated undrained (UU) tests using the sample before loading. The parameter c_u (undrained shear strength) is also called the apparent cohesion, and the analysis using c_u is also called the " ϕ =0 method". Constructions of seawalls or breakwaters without excavation, landfill, and embankments on soft cohesive soil ground fall in this category.

(b) When ground permeability is large or when drainage from consolidated layer is almost completed during construction period because the loading is carried out very slowly:
Because drainage from the layer occurs simultaneously with loading and an increase in strength of the layer is expected along with the loading, the performance verification of structures should be carried out using c_D and ϕ_D determined under consolidated and drained (CD) conditions. Constructions of seawalls or breakwaters, landfill and embankments on sandy soil belong to this category.

(c) When the hydraulic conductivity of the ground is poor and the load is removed to decrease the normal stress σ on the shear plane:
In this case, the most dangerous situation is after a long time has elapsed, when the soil absorbs water, expands, and loses its shear strength, this is called the long-term stability problem. As shown in **Fig. 2.3.6**, undrained shear strength becomes the lowest after water absorption and soil expansion when the over-consolidation ratio is small, in other words, σ is a little less than p_0. In this situation, therefore, the c_u value should be used with consideration of soil swelling. Earth retaining and excavation in clayey ground or removal of preloading on cohesive soil ground belongs to this category. On the other hand, in the case of heavily over-consolidated ground where σ is very small compared to p_0, the parameters c_D and ϕ_D are used for performance verification because the shear strength under consolidated, drained condition is the smallest. Usually, this often applies to cases where cut earth methods are employed but it also applies to construction works in coastal areas such as works to deeper quaywall depth and dredging works on seabed soil.
In almost all cases for normal construction conditions of port facilities, the undrained strength in UU conditions of (a) is used in the performance verification for cohesive soils and the strength parameter in the CD conditions of (b) is used for sandy soils. The following equations show the strength calculation methods respectively:

1) For cohesive soil with the sand content is less than 50%

$$\tau = c_u \tag{2.3.14}$$

where
τ: shear strength
c_u: undrained shear strength

2) For sandy soil with the sand content is higher than 80%

$$\tau = (\sigma - u)\tan\phi_D \tag{2.3.15}$$

where
τ : shear strength
σ : normal stress to shear plane
u : hydraulic pressure at the site
ϕ_D : angle of shear resistance for drained conditions (º)

Furthermore, because soil with a sand fraction ranging from 50% – 80% displays intermediate characteristics between sandy soil and cohesive soil, it is called the intermediate soil. The evaluation of shear strength of intermediate soil is difficult compared with that of sandy soil or cohesive soil. Hence, the shear strength for such soil should be evaluated carefully by referring to the most recent research results. With respect to intermediate soil that can be treated as cohesive soil, it is preferable to utilize results of triaxial CU tests etc. rather than evaluate shear strength from unconfined compressive strength.

(4) Shear Strength of Sand
Because sandy soil has high hydraulic conductivity and is regarded in completely drained condition, the shear strength of sand is represented by equation (**2.3.15**). The angle of shear resistance ϕ_D for drained conditions can be determined using a triaxial CD test under consolidated and drained condition. Because the value of ϕ_D becomes large when sand's void ratio becomes small and its density becomes high, the void ratio e_0 in- situ should be accurately determined. Therefore, it is best to take and test an undisturbed sample. Although the ϕ_D values of sand with the same density will vary a little with the shear conditions, the value of ϕ_D determined by a triaxial CD test, which is conducted with the consolidation pressure corresponding to design conditions with undisturbed sample, can be used as the design parameter for stability analysis. However, in the case of bearing capacity problem for foundation, which is much influenced by progressive failure, the bearing capacity is over-estimated in some cases if the value of ϕ_D determined by a triaxial CD test is directly used as the design parameter.

Compared with the case of cohesive soil, sampling of undisturbed sand samples is technically difficult and also very expensive. This is the reason that the shear strength for sandy soil is frequently determined from the N-value of standard penetration test rather than from a laboratory soil test. For the equation to determine ϕ_D from N-values, refer to **2.3.4 (4) Angle of shear resistance of sandy ground**.

(5) Shear Strength of Cohesive Soil
Here, soil of which the clay and silt fraction by percentage is greater than 50% is regarded as cohesive soil. There are several methods, as presented below, to determine the undrained shear strength c_u of cohesive soil. An appropriate method should be chosen in consideration of such factors as the past experiences, the subsoil characteristics and the importance of the structures.

① q_u method:
This method uses the average value of unconfined compressive strength determined from undisturbed samples. The undrained shear strength c_u used for the performance verification is given by the following equation:

$$c_u = \overline{q_u}/2 \tag{2.3.16}$$

In this equation, $\overline{q_u}$ is the average value of unconfined compressive strength. In unconfined compression tests, confining pressure is not applied on the test sample and therefore, the strength result obtained may be remarkably small due to disturbance of the sample. Application is particularly difficult on clayey soil sampled from depth such as stiff Pleistocene clayey soil in which cracks can appear easily. Caution is also needed for application on intermediate soil with high sand content as effective stress may not be maintained in the test sample and consequently, a remarkably small shear strength may be obtained. In this case, it is preferable to employ other test methods such as triaxial test or direct shear test.

② Method of using strength by triaxial tests taking initial stress and anisotropy into consideration:
Consider the stability analysis of a embankment on the clayey ground using a circular slip, as shown in **Fig. 2.3.7**. Directly below the embankment shearing is caused by the increase in vertical stress, so it is possible to evaluate the shear strength corresponding to this by the triaxial consolidated undrained compression test (CUC test), although strictly speaking there are differences in the plane strain and axial symmetry. On the other hand, shearing occurs at the end point of the circular arc, in other words near the base of the slope, due to the increase in horizontal stress, so it is possible to evaluate this by the triaxial consolidated undrained extension test (CUE).

Of course, there are differences in the plane strain and axial symmetry, and there is the major difference that in contrast to the triaxial extension test in which the axial force reduces, in the failure of an embankment the horizontal stress increases. Near the bottom of the circular arc, the deformation mode is not compression nor extension., but virtually horizontal shearing is produced. Therefore, it is possible to evaluate this by a direct shear test or a simple shear test.

The shear strength s_u* used in the performance verification may be the average value of the shear strength s_{uc} obtained from a compression test and the shear strength s_{ue} obtained from an extension test as given by the following equation

$$s_u{}^* = \frac{s_{uc} + s_{ue}}{2} \tag{2.3.17}$$

or the direct shear strength s_{us} may be used as the representative value.

$$s_u{}^* = s_{us}$$

For most soils, the triaxial extension strength s_{ue} is about 70% of the triaxial compression strength s_{uc}.

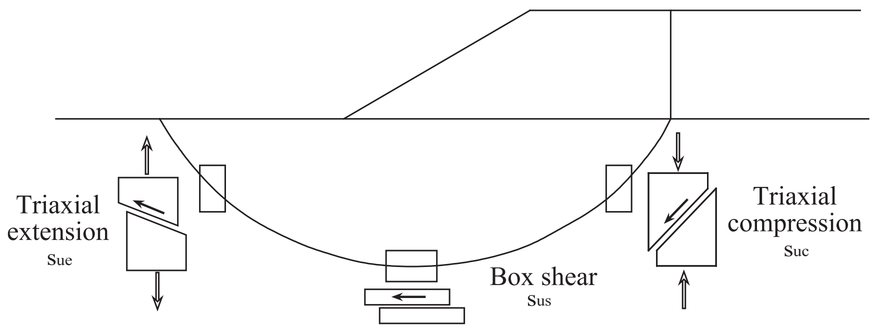

Fig. 2.3.7 Stability Problem and Strength Anisotropy for an Embankment Constructed on a Clayey Ground

Disturbance of a test specimen during sampling is inevitable to a certain extent, even if efforts are made to minimize it. Also, it has been said for a long time that the unconfined compression test is lacking in reliability, but the performance verification methods are frequently based on them, as in the present situation other methods cannot be adopted. As a method of determining the undrained shear strength, the method known as the "recompression method" [11] is said to be the most reliable among the test methods currently proposed. This method is based on the thinking that by reproducing the same stress state as the sampled test specimen in the original location, the effect of disturbance in the test specimen can be made smaller by consolidation.

Elements within a ground are subject to the vertical overburden effective stress σ'_{v0}, and the horizontal earth pressure at rest σ'_{h0} ($=K_0\sigma'_{v0}$). A sampled test specimen has zero stress under atmospheric pressure, and an isotropic residual effective stress due to suction remains to a certain extent. However, by consolidation to $\sigma'_1=\sigma'_{v0}$, $\sigma'_3=K_0\sigma'_{v0}$ in triaxial test apparatus, undrained shear tests can be carried out with the same effective stress state as the original position reproduced. The effective overburden pressure σ'_{v0} can be calculated from the unit weight of the sampled test specimens. However, a problem at this stage is how to obtain the coefficient of earth pressure K_0. Several methods for obtaining it from in-situ tests have been proposed, but it can also be obtained from a laboratory by a K_0 consolidation test using a triaxial cell.[12] Here, the K_0 consolidation test is a test in which the cell pressure σ_3 is controlled so that the cross-sectional area of the test specimen does not change when the axial pressure σ_1 or the axial strain ε_1 increases. However, K_0 obtained by this method is K_0 for the normally consolidated state, frequently expressed as K_{0NC}, so it is necessary to be aware that it is not the K_0 for soil with the aging effect as in a real ground. In Japanese clays, K_0 under normally consolidated conditions is mostly in the range 0.45 to 0.55.

The recompression method is also possible with the direct shear test. In this case, the change in the diameter of the test specimen is constrained by the shear ring, so by simply making the consolidation pressure equal to the effective overburden pressure σ'_{v0}, there is no particular need to be aware of K_0.

Although the undrained shear strength ($q_u/2$) obtained from a unconfined compression test has a large amount of variation, the average value is virtually the same as the average value value of s_{uc} and s_{ue} of the undrained shear strength obtained from triaxial compression and extension tests by the recompression method with consolidation of σ'_{v0} and $K_0\sigma'_{v0}$, which is capable of reproducing the same stress state as the test specimen in the original location. The reliability of the test results using triaxial compression and extension tests by the recompression method whose mechanical basis is clearer, is slightly higher than that of the unconfined compression tests. In section **2.1 Estimation of Ground Constants**, it is expected that triaxial tests, from which results with small variation can be obtained, are preferable for the performance verification.

③ Method using strength from a direct shear test:
This method uses the strength τ_{DS} determined by a direct shear test after undisturbed sample is consolidated one-dimensionally under in-situ effective overburden pressure. The direct shear test can be conducted according to the **JGS 0560 Method for Consolidation parameters Pressure Direct Direct shear test on Soil** of the Geotechnical Society of Japan. The undrained shear strength c_u used for the performance verification is given by the following equation:

$$c_u = 0.85\tau_{DS} \tag{2.3.18}$$

In this equation, 0.85 is a correction factor related to shear rate effect. The measured values have therefore undergone primary processing to arrive at the derivative values.

④ Methods combining unconfined compressive strength and strength from triaxial compression tests:
One problem with the q_u method is that the test's reliability is low in soil with no past records, because the test is subject to the influence of disturbance during sampling. To resolve this problem, a combination method can be used to determine the strength by comparing the q_u of undisturbed samples with the strength from a triaxial CU test and evaluating the quality of the sample. In this method, the sample is isotropically consolidated by in- situ mean effective stress of $2\sigma'_{v0}/3$ when K_0=0.5, after which triaxial CU test is performed in undrained compression condition. The undrained shear strength thus obtained must be empirically corrected by multiplying 0.75. In other words, as is the case with the direct shear test, for this triaxial test, measured values must undergo primary processing to arrive at the derivative values. This method is used for natural soil ground and cannot be applied to unconsolidated reclaimed ground. For more details see the references 13) and 14).

⑤ Method for determining undrained shear strength from an in-situ vane shear test:
A vane shear test is conducted as described in **1.3 Selection of Investigation Methods**. The average value of the obtained shear strength $c_{u(v)}$ can be used in the performance verification as the undrained shear strength c_u [15]. An in-situ vane shear test can be carried out rather easily with mobility at a field site. The test is able to determine the shear strength for very soft clay for which an unconfined compression test cannot be performed due to the difficulty in making a specimen freestanding. It can thus be applied, for example, to the construction management where soil is being improved using vertical drains. Although the test method and principle are simple, attention must be given to the effect of friction on the rod. Ways of reducing the friction and calibrating its effect need to be devised.

Each method has its own characteristics, which must be duly considered in order to select the most appropriate one.

The undrained shear strength c_u of cohesive soils increases as consolidation progresses, and the higher the consolidation load the larger the c_u after consolidation. Therefore, the consolidation pressure increases with depth as the overburden pressure increases, so normally the c_u of a clay ground increases with depth, and the distribution of undrained shear strength used in the performance verification is frequently expressed by the following equation.

$$c_u = c_{u0} + kz \tag{2.3.19}$$

where

c_u : undrained shear strength at depth z from the surface of the clay layer
c_{u0} : undrained shear strength at surface of the clay layer
k : rate of increase of c_u with depth z
z : depth from the surface of the clay layer

(6) Increase in Cohesive Soil Strength due to Consolidation
The undrained strength of cohesive soil will increase with the progress of consolidation. For soil improvement methods such as the vertical drain method, the ratio of strength increase c_u/p by consolidation is an important parameter because the strength is increased by the drainage of pore water by consolidation. Naturally sedimented cohesive soil ground can be somewhat overconsolidated, or even if it is normally consolidated in terms of stress history, it can appear to be overconsolidated with large consolidation yield stress p_c due to aging effect. For this reason, the ratio of strength increase becomes the cohesive soil's specific parameter in the case of slight overconsolidation through normalizing, not by the effective overburden pressure σ'_{v0} equivalent to the consolidation pressure, but by the consolidation yield stress p_c ($m=c_u/p_c$). The larger the value of c_u/p_c, which is a soil property parameter used in the vertical drain method for increasing strength, the larger the increase ratio of the strength and the more effective soil improvement are expected. From the past experiences in the field and research results for marine clay in Japan, the value of c_u/p_c lies in a range shown by the following equation, regardless of plasticity.

$$c_u / p_c = 0.2 \sim 0.25 \tag{2.3.20}$$

In view of the fact that the overconsolidation ratio OCR of naturally sedimented cohesive soil is normally in the range from 1.0 to 1.5, and $\sigma'_{v0}=p_c$/OCR, therefore, the data in **Fig. 2.3.8** [15] provides substantiation for equation **(2.3.20)**.

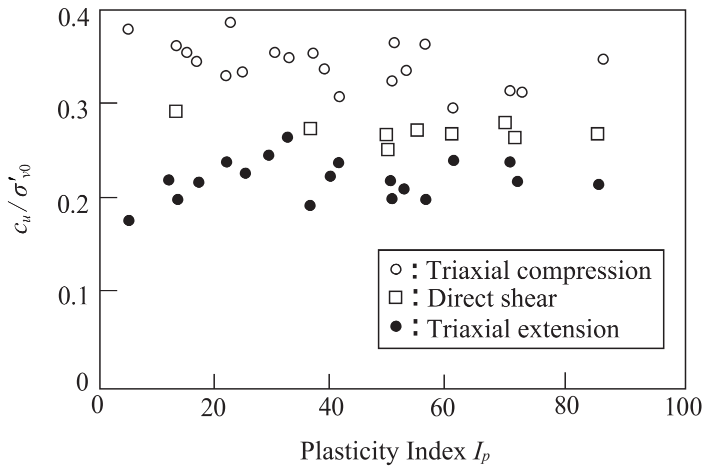

Fig. 2.3.8 Relationship between Plasticity Index and c_u/σ'_{v0}

(7) Decrease of Strength of Cohesive Soils due to Swelling
If part of the load is removed after consolidation, cohesive soils absorb water and swell with time, which causes the c_u to reduce. In addition, the time required for swelling is considerably shorter than the time required for consolidation. The drainage conditions of this case correspond to the left of ③ as shown in **Fig. 2.3.6**, so it is necessary to evaluate the likely strength decrease after swelling.[16] Specifically, the removal of load at the end of consolidation in soil improvement works such as the vertical drain method or the preload method, excavation for earth retaining structures, [17] and dredging to deeper the sea bottom, etc., correspond to this situation.

(8) Strength of Intermediate Soil
Soil with a sand content in the range of 50% - 80% is intermediate soil between sandy soil and cohesive soil.[10] For this type of soil, the hydraulic conductivity and design conditions are taken into consideration to determine whether the soil is sandy soil or cohesive soil. Then the shear strength is determined accordingly. For intermediate soil with a large sand fraction or with coral gravel, the hydraulic conductivity determined from an incremental loading oedometer test generally gives an underestimated value, because of the limitations of test conditions. It is preferable not only to improve the test procedure, but also to conduct an in-situ permeability test or an electrical cone test to determine the hydraulic conductivity.[19]

When the hydraulic conductivity determined by this kind of procedure is greater than 10^{-4} cm/s, the ground is regarded permeable. Hence, the value of ϕ_D determined from an electrical cone penetration resistance or a triaxial CD test can be used as design parameters regarding $c_D = 0$. According to experience in investigating the properties of intermediate soils in Japan, the value of ϕ_D is greater than 30º in many cases.[20, 21, 22]

When the hydraulic conductivity is less than 10^{-4} cm/s, the performance verification of the intermediate soil should be conducted as a cohesive soil. Because the influence of stress release during sampling in intermediate soil is much greater than that in cohesive soil, the shear strength determined by q_u method is underestimated. A correction method is used for the strength of such intermediate soil with a large sand fraction by means of clay fraction and plasticity index.[23] However, it is preferable that the combined method with unconfined compression test and triaxial compression test or the direct shear test be used as the method for evaluating the strength of intermediate soil.[24]

2.3.4 Interpretation Method for N Values

(1) The angle of shear resistance for sandy soils is calculated using the following equation from a standard penetration test value.

$$\phi = 25 + 3.2\sqrt{\frac{100N}{70 + \sigma'_{v0}}} \tag{2.3.21}$$

where

ϕ : angle of shear resistance of sand (º)

N : standard penetration test value

σ'_{v0} : effective overburden pressure at the depth where the standard penetration test is performed (kN/m²)

(2) Relationships between the N-value and many soil parameters have been established by the data at various sites. When using these relationships, however, it is necessary to consider the background of their derivation and the ground conditions of the data and to confirm the range of their applicability. As can be seen in Dunham's equation, which has commonly been used for many years, the value of ϕ was determined directly from the N-values without considering the effective overburden pressure σ'_{v0}. However, because the relative density D_r varies with σ'_{v0} as seen in **Fig. 2.3.9**, σ'_{v0} must be taken into consideration to determine D_r from an N-value. This concept was incorporated in the judgment of liquefaction. In this judgment, liquefaction resistance is examined from N_{65}, the equivalent N-value converted into N-value when effective overburden pressure σ'_{v0}=65kN/m². Similarly, it is known that even in grounds with the same ϕ , the N-value increases with the increase in effective overburden pressure. Therefore, the influence of σ'_{v0} must be taken into account when determining ϕ from the N-values.

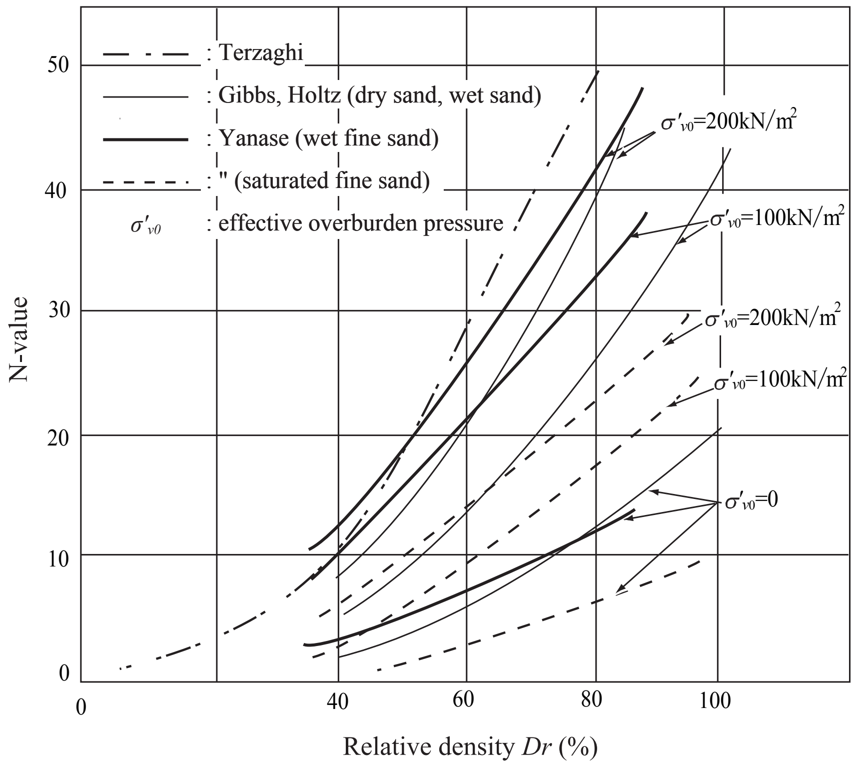

Fig. 2.3.9 Influence of Effective Overburden Pressure and Relative Density on N-Values

(3) Factors Affecting the N-values

The factors that affect N-values mutually overlap, and methods for quantitatively correcting for these factors have not yet been established. However, for understanding of N-values, the extent of the effect of the important influencing factors is as follows:

① Density

As the density, relative density, of the subsoil increases, in particular for sandy soils, the N-value increases.

② Water content

Apart from well compacted fine sand and silty soils, the N-value increases in the order of saturated sand, dry sand, and wet sand.

③ Effective overburden pressure

The N-value increases as the effective overburden pressure increases.

④ Effect of groundwater level

As the groundwater level fluctuates, the effective overburden pressure and the degree of saturation of the soil varies, so the N-values vary accordingly.

⑤ Other influencing factors

The N- value varies in accordance with the soil particle shape, the grain size distribution, and the mineral composition of the soil.

(4) Angle of Shear Resistance of Sandy Ground
The angle of shear resistance ϕ is an important constant for the performance verification of grounds, similar to the undrained shear strength of clay soils. However, ϕ is a complex value that is governed by many factors, and even the same soil will not have a constant value. Therefore, it is necessary to investigate sufficiently the background to establishing the performance verification methods, such as what conditions are assumed in the performance verification methods using ϕ .

(5) *N*- value of Cohesive Soil Ground
Compared with sand ground, the *N*- value of a cohesive soil is small, and its reliability is low. According to past experience and test results, if the q_u is 100kN/m^2 or lower the measurement of *N*- value is difficult. In cohesive soils with q_u of this value, when soft cohesive soils are found by taking test specimens with a standard penetration test type sampler during a preliminary investigation, or when tests are carried out to know the physical properties, this has significance, but the strength and other mechanical subsoil constants cannot be determined from the *N*- values only. In the case of a Pleistocene clay soil with high strength, the past deposition environment and stress history has changed several times, so even within the same stratum the properties of the soil are not uniform, and an overconsolidated state that is unrelated to the present effective overburden pressure is frequently found. Therefore, the *N*- values and soil properties change greatly with only small changes in position or depth. Also, techniques for sampling stiff soils are difficult, and cracks can easily be firmed in the test specimens. In Japan the strength of stiff cohesive soil is frequently evaluated using the q_u value, but the q_u value is very easily affected by the quality of the test specimens.

2.4 Dynamic Analysis

2.4.1 Dynamic Modulus of Deformation

(1) For seismic response analysis, an appropriate dynamic modulus of deformation of soils shall be determined to prescribe the relationship between the shear stress and shear strain of soil.

(2) The performance verification of seismic-resistant can be broadly classified into the static performance verification methods and the dynamic performance verification methods. One example of static performance verification methods is the seismic coefficient method of which the seismic force is assumed to act on the ground or structures in the form of a static inertia force, and stability is examined from the equilibrium of forces. In the dynamic performance verification methods, on the other hand, dynamic magnification factors or amplification values of acceleration, speed, and deformation of subsoils shallower than bedrock and foundation ground for structures are calculated to examine the stability of ground or structures. As for the seismic response analysis method, both the time domain analysis and the frequency domain analysis are used. For either method, the relationship between the shear stress and shear strain of the soils is required.

Normally the relationship between the shear stress and shear strain in ground subjected to dynamic actings is described by a skeleton curve and a hysteresis curve, as shown in **Fig. 2.4.1 (a)**. A skeleton curve will display remarkable nonlinearity as the shear strain amplitude becomes larger. Since the dynamic modulus of deformation prescribes this relationship between the shear stress and shear strain, it must be appropriately applied when conducting a seismic response analysis.

(3) Relationship between Dynamic Shear Stress and Shear Strain of Soil
There are many models to apply the shear stress and shear strain curves of soil into analysis, such as the hyperbolic model called Hardin-Dornevich model, and the Ramberg-Osgood model.[29)]

(4) Expression Method of Deformation Properties in the Equivalent Linear Model
To estimate the behavior of ground during an earthquake, the nonlinearity of the relationship between the dynamic stress and strain of soil for a wide range of the shear strain amplitude must be appropriately assessed and modeled. The relationship of the dynamic stress and strain of soil is expressed with two parameters: the shear modulus and the damping factor in the equivalent linear model. The shear modulus *G* and the damping factor *h* are defined with the shear strain amplitude by equation **(2.4.1)** and equation **(2.4.2)** as shown in **Fig. 2.4.1 (b)**.

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

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

where

G : shear modulus (kN/m^2)
τ : shear stress amplitude (kN/m^2)
γ : shear strain amplitude

h : damping factor
W : strain energy (kN/m2)
ΔW : damping energy (kN/m2)

Since the values of shear modulus G and damping factor h vary nonlinearly depending on the value of γ, a $G/G_0 \sim \gamma$ curve and a $h \sim \gamma$ curve are normally drawn as shown in **Fig 2.4.2**, where G_0 is the shear modulus at $\gamma \fallingdotseq 10^{-6}$.

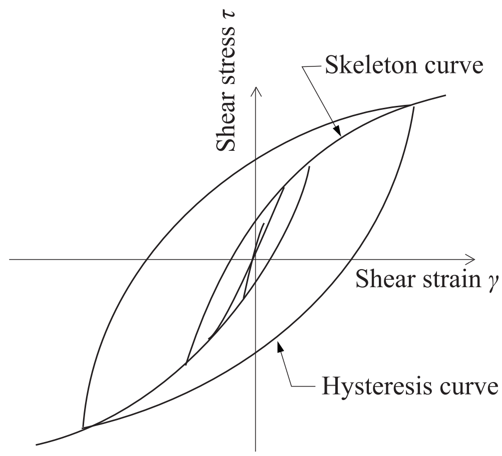

Fig. 2.4.1 (a) Stress Strain Curve

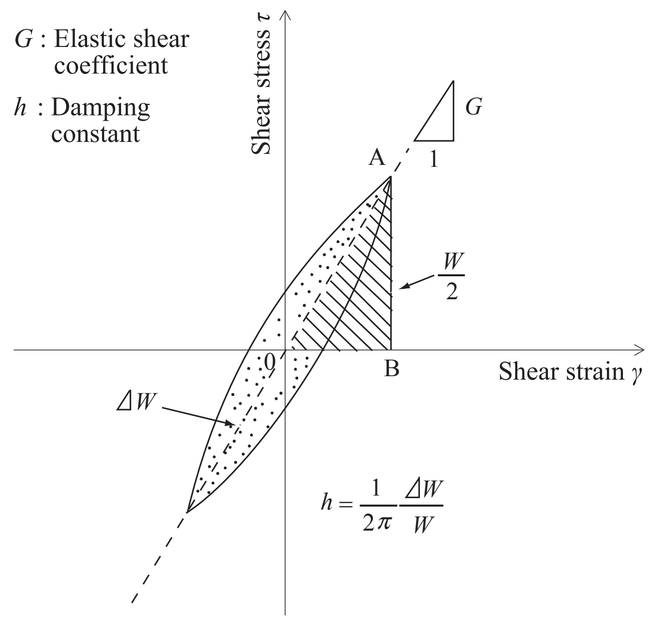

Fig. 2.4.1 (b) Shear Modulus and Damping factor

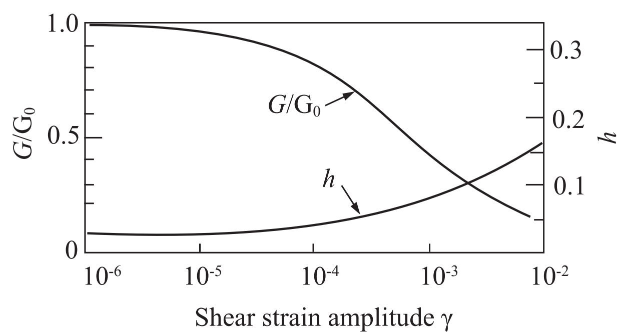

Fig. 2.4.2 Shear Modulus, Damping Factor and Shear Strain Amplitude

(5) Measurement of the Shear Modulus and the Damping Factor

The shear modulus and the damping factor must be determined by laboratory tests such as the resonance test or cyclic triaxial test, or by the in-situ tests using elastic waves such as the PS logging method or the cross hole velocity measurement method. The laboratory tests can be used to measure the shear modulus and damping factor for a wide range of shear strain amplitudes from the shear strain of 10^{-6} to the failure although undisturbed samples from the field must be obtained. The tests can also be used to evaluate the change in the modulus of dynamic deformation due to construction of structures. With the cyclic triaxial test, the shear modulus is determined from equation **(2.4.3)** together with Poisson's ratio ν.

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

where

σ_a : axial stress amplitude (kN/m²)

ε_a : axial strain amplitude

For ν, the value of 0.33 is normally used for a drainage condition and 0.45 is used for an undrained condition.

The damping factor is calculated from equation **(2.4.2)** with W and ΔW obtained from the stress-strain curve similar to that shown in **Fig. 2.4.1 (b)**.

In-situ tests are limited to measurements of the shear modulus that only corresponds to 10^{-6} level of shear strain amplitude. Such tests have not been put to practical application to measure the shear modulus and damping factor for the large shear strain amplitude. But the tests possess the advantage of being able to measure the values in- situ directly. They are also used to correct the shear modulus obtained from laboratory tests. The elastic constant of subsoil is obtained by equations **(2.4.4)** to (**2.4.6**) from the data of elastic wave velocity measurements by a seismic exploration using bore holes.

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

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

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

where

V_p : longitudinal wave velocity (m/s)

V_s : transverse wave velocity (m/s)

G_0 : shear modulus (kN/m²)

E_0 : Young's modulus (kN/m²)

ν : Poisson's ratio

ρ : density (t/m³)

γ_t : wet unit weight (kN/m³)

g : gravitational acceleration (m/s²)

There are various items requiring attention relating to the taking of measurements when carrying out elastic wave exploration on soft seabed ground. These include vibration induction and reception methods for elastic waves such as longitudinal and transverse waves, accuracy of wave profile readings and methods for protecting bore holes.

(6) Simple Estimation of Shear Modulus and Damping Factors

In cases where it is difficult to directly measure the shear modulus and the damping factors of soils from laboratory tests or in-situ tests, there are methods for estimating from the plasticity index, the void ratio, the unconfined compressive strength, and the N-value.[30] However, it is necessary to be aware that in the method of estimating from the N-value, the variation in the estimated values is large, and the coefficient of variation is about 0.2. For example, on the basis of the variation of N-value and S wave velocities by Imai, [31] for each ground type, accuracy examination of estimation error of S wave velocity is shown for Holocene sandy and clayey soil in **Fig.2.4.3.** The horizontal axis shows the ratio of estimated values of S wave velocity converted from the N- values and the actual values. For Holocene sandy soils the average value of the ratio is 1.12 with a standard deviation of 0.29, an extremely large variation. For Holocene clay soils the average value of the ratio is 0.95 with a standard deviation of 0.32. In both cases the statistical distribution may be regarded as a log-normal distribution.[32)]

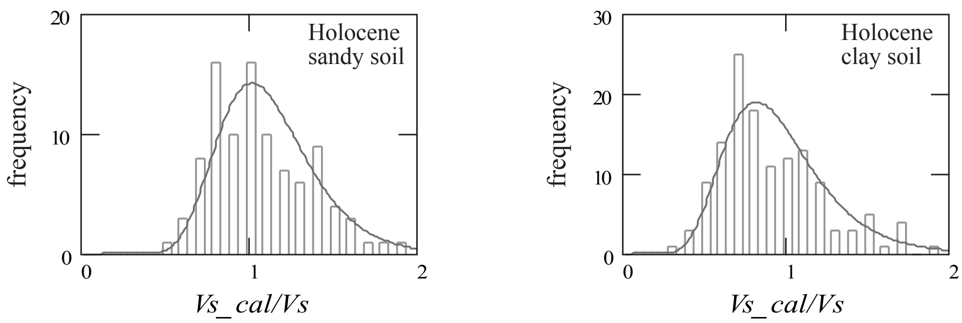

Fig. 2.4.3 Estimation Accuracy for S Wave Velocity

2.4.2 Dynamic Strength Properties

(1) Soil strength against dynamic external actions is normally determined through laboratory tests. In this case, the properties of the external forces and the subsoil conditions need to be appropriately determined.

(2) The typical dynamic external actions encountered in ports and harbors are seismic movement and wave force. Seismic movements are characterized by a short period and few cyclic repetitions, while wave forces are characterized by a long period and many cyclic repetitions. At present these dynamic external actions are normally converted into static actions like in the seismic coefficient method. There are the cases, however, in which it is necessary to treat them as dynamic loads like in liquefaction analysis or in strength decrease analysis of cohesive soil of foundation ground beneath structures exposed to waves. In such cases the dynamic strength of soils are normally obtained by cyclic triaxial tests. When conducting cyclic triaxial tests, the cyclic undrained triaxial test method explained in the **Soil Testing Methods and Commentary** of the Geotechnical Society of Japan can be used.[33)]

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