(3) Performance verification of SRC Members

① The steel and reinforced concrete (SRC) members shall be designed against the flexural moment and shearing force, by taking full account of the structural characteristics due to differences in the structural type of the steel frame.

② SRC members can normally be classified as follows, depending on the structural type of steel frames :

(a) Full-web type

(b) Truss web type

③ For the flexural moment, the section stress can be calculated as a reinforced concrete member by converting steel frames to equivalent reinforcements. When the fixing of steel frame ends with concrete is insufficient in full-web type, it should be calculated as a composite of the independent steel frame member and the reinforced concrete member.

④ For shearing force, if the web is of truss type, the shear stress can be calculated as a reinforced concrete by converting steel frames to equivalent reinforcements. If it is of full-web type, steel frames themselves can resist against the shearing force, and they can be duly considered in design.

(4) Performance Verification of Partition Walls

Because partition walls function as a bearing side of the outer walls and bottom slab, in performance verification, stability of the cross section of the partition wall should be secured against the sectional forces calculated based on the actions on these bearing sides.

(5) Performance Verification of Corners and Joints

① Corners and joints shall be designed to smoothly and firmly transmit section forces, and to be easily fabricated and executed.

② To secure sufficient strength at corners and joints, it is desirable to firmly connect the steel materials on the tensile side to those of the compressive side. It is also desirable to provide shear reinforced steel materials (haunches) against concrete tensile stress of the inside of joints.

(6) Performance Verification for Fatigue Failure

① Hybrid caissons use a large number of welded joints for connecting steel plates, and attaching shear connectors and shear resistance steel. Therefore, where the members are frequently subject to repeated load, the fatigue strength in welded parts should be examined.

② In coastal revetments and quaywalls, the influence of repeated actions is small. However, in performance verifications of breakwaters, when the stress on members due to waves as a repeated action changes significantly, examination for fatigue failure of the caisson is needed.

1.6.5 Corrosion Control

(1) Corrosion control of hybrid caissons shall be set appropriately considering the performance requirements, level of maintenance control, construction conditions, and other relevant factors.

(2) The main cause of deterioration of hybrid members is corrosion of the steel materials. Because there are cases in which corrosion of the steel materials may result in developing cracks of the concrete, appropriate corrosion prevention measures should be taken for steel plates in order to improve the durability of the hybrid members. The deterioration characteristics of the concrete itself should be considered to be the same as that of conventional reinforced concrete.

(3) Steel materials used on the outside of hybrid caissons are generally covered with concrete or asphalt mats. The inside of a caisson is isolated from the external atmosphere by means of concrete lids. It is also in contact with filling sand in a static state and with residual seawater. Thus, when designing hybrid caissons, direct contact between the steel plates of members and the marine environment is generally avoided. For corrosion control, it is usual to set steel plate on the inside and concrete on the outside so as to avoid direct contact of steel plate with fresh seawater. If steel plates are in direct contact with seawater, corrosion control should be applied such as coating methods to splash zone or tidal zone and cathodic protection methods in seawater.

1.7 Armor Stones and Blocks

Public Notice

Performance Criteria of Armor Stones and Blocks

Article 28
The performance criteria of rubble stones and concrete blocks armoring a structure exposed to the actions of waves and water currents as well as armor stones and armor blocks of the foundation mound shall be such that the risk of exceeding the allowable degree of damage under the variable action situation, in which the dominant actions are variable waves and water currents, is equal to or less than the threshold level.

[Commentary]

(1) Performance Criteria of Armor Stones and Blocks
The settings of the performance criteria and design situations, excluding accidental situations, for armor stones and blocks shall be as shown in the **Attached Table 14**.

Attached Table 14 Settings for Performance Criteria and Design Situations (excluding accidental situations) for Armor Stones and Blocks

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Index of standard limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
7	1	–	28	1	–	Serviceability	Variable	Variable waves	Self weight, water pressure	Extent of damage	Limit value of damage rate, degree of damage, or deformation level

① Extent of damage
The indexes which express the extent of damage of armor stones and blocks for the variable situations in which the dominating actions are variable waves and water currents are the damage rate, the degree of damage, and the deformation level.
In the performance verification of armor stones and blocks, the indexes including the degree of damage and the limit value thereof shall be set appropriately considering the design working life of the objective facilities, the construction work conditions, the time and cost necessary for restoration, and the conditions of waves and water currents, etc.

[Technical Note]

1.7.1 Required Mass of Armor Stones and Blocks on Slope [24), 25)]

(1) General
The armor units for the slopes and a sloping breakwaters are placed to protect the rubble stones inside; it is necessary to ensure that an armor unit has a mass sufficient to be stable so that it does not scatter itself. This stable mass, required mass, can generally be obtained by hydraulic model tests or calculations using appropriate equations.

(2) Basic Equation for Calculation of Required Mass
When calculating the required mass of rubble stones and concrete blocks covering the slope of a sloping structure which is affected by wave forces, Hudson's formula with the stability number N_S, which is shown in the following equation, may be used.[26)] In this equation, the symbol γ is a partial factor for its subscript, and the subscripts k and d show the characteristic value and design value, respectively. For the partial safety factors γ_{NS} and γ_H in the equation, 1.0 may be used.

$$M_d = \frac{\rho_r H_d^{\,3}}{N_{Sd}^{\,3}(S_r - 1)^3} \tag{1.7.1}$$

where
M : required mass of rubble stones or concrete blocks (t)
ρ_r : density of rubble stones or concrete blocks (t/m3)
H : wave height used in stability calculation (m)
N_S : stability number determined primarily by the shape, slope, damage rate of the armor, etc.
S_r : specific gravity of rubble stones or concrete blocks relative to water

The design values in the equation may be calculated using the following equations.

$$H_d = \gamma_H H_k \quad , \quad N_{Sd} = \gamma_{Ns} N_{Sk}$$

(3) Design Wave Height H Used in the Performance Verification

Hudson's formula was proposed based on the results of experiments that used regular waves. When applying it to the action of actual waves which are random, there is thus a problem of which definition of wave heights shall be used. However, with structures that are made of rubble stones or concrete blocks, there is a tendency for damage to occur not when one single wave having the maximum height H among a random wave train attacks the armor units, but rather for damage to progress gradually under the continuous action of waves of various heights. Considering this fact and past experiences, it has been decided to make it standard to use the significant wave height of incident waves at the place where the slope is located as the wave height H in equation **(1.7.1)**, because the significant wave height is representative of the overall scale of a random wave train. Consequently, it is also standard to use the significant wave height when using the generalized Hudson's formula. Note however that for places where the water depth is less than one half of the equivalent deepwater wave height, the significant wave height at the water depth equal to one half of the equivalent deepwater wave height should be used.

(4) Parameters Affecting the Stability Number N_S

As shown in equation **(1.7.1)**, the required mass of armor stones or concrete blocks varies with the wave height and the density of the armor units, and also the stability number N_S. The N_S value is a coefficient that represents the effects of the characteristics of structure, those of armor units, wave characteristics and other factors on the stability. The main factors that influence the N_S value are as follows.

① Characteristics of the structure

(a) Type of structure; sloping breakwater, breakwater covered with wave-dissipating concrete blocks, and composite breakwater, etc.

(b) Gradient of the armored slope

(c) Position of armor units; breakwater head, breakwater trunk, position relative to still water level, front face and top of slope, back face, and berm, etc.

(d) Crown height and width, and shape of superstructure

(e) Inner layer; coefficient of permeability, thickness, and degree of surface roughness

② Characteristics of the armor units

(a) Shape of armor units (shape of armor stones or concrete blocks; for armor stones, their diameter distribution)

(b) Placement of armor units; number of layers, and regular laying or random placement, etc.

(c) Strength of armor material

③ Wave characteristics

(a) Number of waves acting on armor layers

(b) Wave steepness

(c) Form of seabed (seabed slope, where about of reef, etc.)

(d) Ratio of wave height to water depth as indices of non-breaking or breaking wave condition, breaker type, etc.

(e) Wave direction, wave spectrum, and wave group characteristics

④ Extent of damage (damage ratio, deformation level, relative damage level)

Consequently, the N_S value used in the performance verification must be determined appropriately based on hydraulic model experiments in line with the respective design conditions. By comparing the results of regular waves experiments with those of random wave experiments, [27] it was found that the ratio of the height of regular waves to the significant height of random waves that gave the same damage ratio, within the error of 10%, varied in the range of 1.0 to 2.0, depending on the conditions. In other words, there was a tendency for the random wave action to be more destructive than the action of regular waves. It is thus better to employ random waves in experiments.

(5) Stability Number N_S and K_DValue

In 1959, Hudson published the so-called Hudson's formula, [26] replacing the previous Iribarren-Hudson's formula. Hudson developed equation **(1.7.1)** by himself using $K_D \cot \alpha$ instead of N_S.

$$N_S^{\ 3} = K_D \cot \alpha \tag{1.7.3}$$

where

α : angle of the slope from the horizontal line(°)

K_D : constant determined primarily by the shape of the armor units and the damage ratio

The Hudson's formula was based on the results of a wide range of model experiments and has proved itself well in usage in-site. This formula using the K_D value has thus been used in the calculation of the required mass of armor units on a slope.

However, the Hudson's formula that uses the stability number in equation **(1.7.1)** has been used for quite a while for calculating the required mass of armor units on the foundation mound of a composite breakwater as discussed in **1.7.2 Required Mass of Armor Stones and Blocks in Composite Breakwater Foundation Mound against Waves**, and is also used for the armor units of other structures such as submerged breakwaters. It is thus now more commonly used than the old formula with the K_D value.

The stability number N_S can be derived from the K_D value and the angle α of the slope from the horizontal line by using equation **(1.7.3)** There is no problem with this process if the K_D value is an established one and the slope angle is within a range of normal design. However, most of the K_D values obtained up to the present time have not sufficiently incorporated various factors like the characteristics of the structure and the waves. Thus, this method of determining the stability number N_S from the K_D value cannot be guaranteed to obtain economical design always. In order to calculate more reasonable values for the required mass, it is thus preferable to use the results of experiments matched to the conditions in question, or else to use calculation formulas, calculation diagrams, that include the various relevant factors as described below.

(6) Van der Meer's Formula for Armor Stones

In 1987, van der Meer carried out systematic experiments concerning the armor stones on the slope of a sloping breakwater with a high crown. He proposed the following calculation formula for the stability number, which can consider not only the slope gradient, but also the wave steepness, the number of waves, and the damage level.[28] Note however that the following equations have been slightly altered in comparison with van der Meer's original one in order to make calculations easier. For example, the wave height $H_{2\%}$ for which the probability of exceedance is 2% has been replaced by $H_{1/20}$.

$$N_S = \max\left(N_{sp\ell}, N_{ssr}\right) \tag{1.7.4}$$

$$N_{sp\ell} = 6.2 C_H P^{0.18}\left(S^{0.2}/N^{0.1}\right) I_r^{-0.5} \tag{1.7.5}$$

$$N_{ssr} = C_H P^{-0.13}\left(S^{0.2}/N^{0.1}\right)\left(\cot\alpha\right)^{0.5} I_r^{\ p} \tag{1.7.6}$$

where

$N_{sp\ell}$: stability number for plunging breakers

N_{ssr} : stability number for surging breaker

I_r : iribarren number ($\tan\alpha/S_{om}^{0.5}$)), also called the surf similarity parameter

S_{om} : wave steepness($H_{1/3}/L_0$)

L_0 : deepwater wavelength ($L_0=gT_{1/3}^2/2\pi, g=9.81\text{m/s}^2$)

$T_{1/3}$: significant wave period

C_H : breaking effect coefficient {$=1.4/(H_{1/20}/H_{1/3})$}, (=1.0 in non-breaking zone)

$H_{1/3}$: significant wave height

$H_{1/20}$: highest one-twentieth wave height, see **Fig. 1.7.1**

α : angle of slope from the horizontal surface (°)

D_{n50} : nominal diameter of armor stone ($=(M_{50}/\rho_r)^{1/3}$)

M_{50} : 50% value of the mass distribution curve of an armor stone namely required mass of an armor stone

P : permeability index of the inner layer, see **Fig. 1.7.2**

S : deformation level ($S=A/D_{n50}^2$), see **Table 1.7.1**

A : erosion area of cross section, see **Fig. 1.7.3**

N : number of acting waves

The wave height $H_{1/20}$ in **Fig. 1.7.1** is for a point at a distance $5H_{1/3}$ from the breakwater, and H_0' is the equivalent deepwater wave height. The deformation level S is an index that represents the amount of deformation of the armor stones, and it is a kind of damage ratio. It is defined as the result of the area A eroded by waves, see **Fig. 1.7.3**, being divided by the square of the nominal diameter D_{n50} of the armor stones. As shown in **Table 1.7.1**, three stages are defined with regard to the deformation level of the armor stones : initial damage, intermediate damage, and failure. With the standard design, it is common to use the deformation level for initial damage for $N = 1000$ waves. However, in case where a certain amount of deformation is permitted, usage of the value for intermediate damage may also be envisaged.

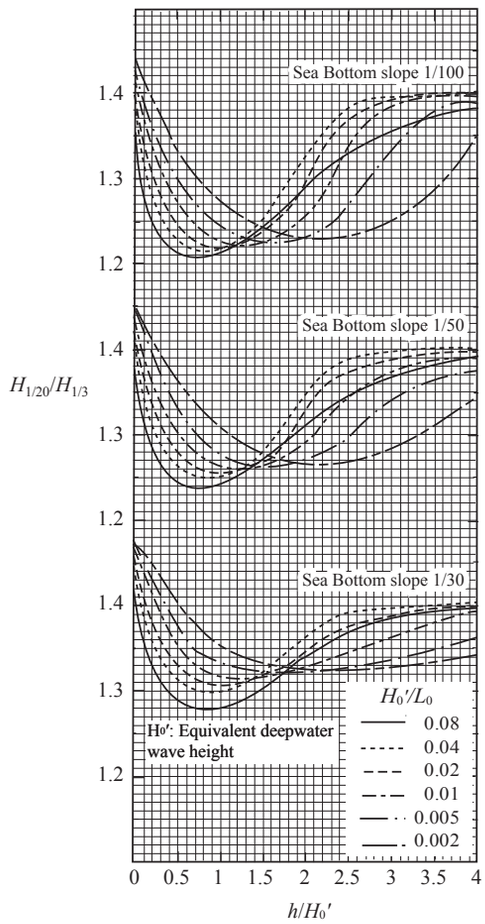

Fig. 1.7.1 Ratio of $H_{1/20}$ to $H_{1/3}$ ($H_{1/20}$ Values are at a Distance $5H_{1/3}$ from the Breakwater)

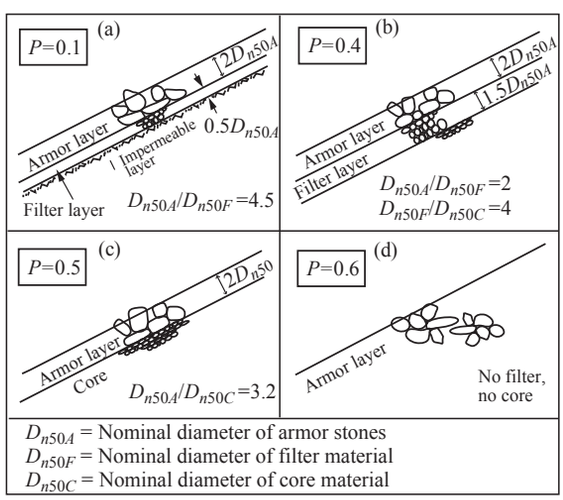

Fig. 1.7.2 Permeability Index P

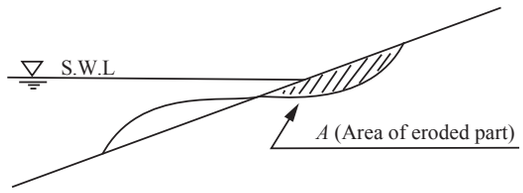

Fig. 1.7.3 Erosion Area *A*

Table 1.7.1 Deformation Level *S* for Each Failure Stage for a Two-layered Armor

Slope	Initial damage	Intermediate damage	Failure
1:1.5	2	3–5	8
1:2	2	4–6	8
1:3	2	6–9	12
1:4	3	8–12	17
1:6	3	8–12	17

(7) Formulation for Calculating Stability Number for Armor Blocks including Wave Characteristics

Van der Meer has carried out model experiments on several kinds of precast concrete blocks, and proposed the formulas for calculating the stability number N_S.[29] In addition, other people have also conducted research into establishing calculation formulas for precast concrete blocks. For example, Burcharth and Liu [30] have proposed a calculation formula. However, it should be noted that these are based on the results of experiments for a sloping breakwater with a high crown.

Takahashi et al.[31] showed a performance verification method of the stability against wave action for armor stones of a sloping breakwater using Van der Meer's formula as the verification formula, and proposed the performance matrix used for performance verification.

(8) Formulas for Calculating Stability Number for Concrete Blocks of Breakwater Covered with Wave-dissipating Blocks

The wave-dissipating concrete block parts of a breakwater covered with wave-dissipating blocks may have various cross-sections. In particular, when all the front face of an upright wall is covered by wave-dissipating concrete blocks, the stability is higher than that of armor concrete blocks of an ordinary sloping breakwater because the permeability is high. In Japan, much research has been carried out on the stability of breakwaters covered with wave-dissipating concrete blocks. For example, Tanimoto et al.[32], Kajima et al.[33], and Hanzawa et al.[34] have carried out systematic research on the stability of wave-dissipating concrete blocks. In addition, Takahashi et al.[35] have proposed the following equation for wave-dissipating concrete blocks that are randomly placed in all the front face of an upright wall.

$$N_S = C_H \left\{ a\left(N_0 / N^{0.5}\right)^{0.2} + b \right\} \tag{1.7.7}$$

where

N_0 : degree of damage, a kind of damage rate that represents the extent of damage: it is defined as the number of concrete blocks that have moved within a width D_n in the direction of the breakwater alignment, where D_n is the nominal diameter of the concrete blocks: $D_n=(M/\rho_r)^{1/3}$, where M is the mass of a concrete block

C_H : breaking effect coefficient; $C_H=1.4/(H_{1/20}/H_{1/3})$, in non-breaking zone $C_H = 1$.

a, b : coefficients that depend on the shape of the concrete blocks and the slope angle. With deformed shape blocks having a K_D value of 8.3, it may be assumed that $a = 2.32$ and $b = 1.33$, if $\cot\alpha=4/3$, and $a = 2.32$ and $b = 1.42$, if $\cot\alpha=1.5$.

Takahashi et al.[35] have further presented a method for calculating the cumulative degree of damage, the expected degree of damage, over the service lifetime. In the future, reliability design methods that consider the expected degree of damage is important as the more advanced design method. In the region where wave breaking does not occur, if the number of waves is 1000 and the degree of damage N_0 is 0.3, the design mass as calculated using the method of Takahashi et al. is more-or-less the same as that calculated using the existing K_D value. The value of $N_0 = 0.3$ corresponds to the conventionally used damage rate of 1%.

(9) Increase of Mass in Breakwater Head

Waves attack the head of a breakwater from various directions, and there is a greater risk of the armor units on the

top of the slope falling to the rear rather than the front. Therefore, rubble stones or concrete blocks which are to be used at the head of a breakwater should have a mass greater than the value given by equation **(1.7.1)**.

Hudson proposed increasing mass by about 10% in the case of rubble stones and about 30% in the case of concrete blocks. However, because this is thought to be insufficient, it is preferable to use rubble stones or concrete blocks with a mass at least 1.5 times the value given by equation **(1.7.1)**. Kimura et al.[36] have shown that, in a case where perpendicular incident waves act on the breakwater head, the stable mass can be obtained by increasing the required mass of the breakwater trunk by 1.5 times. In case of oblique incidence at 45º, in the breakwater head on the upper side relative to the direction of incidence of the waves, the necessary minimum mass is the same as for 0º incidence, whereas, on the lower side of the breakwater head, stability is secured with the same mass as the in the breakwater trunk.

(10) Submerged Armor Units

Since the action of waves on a sloping breakwater below water surface is weaker than above the water surface, the mass of stones or concrete blocks may be reduced at depths greater than $1.5H_{1/3}$ below the still water level.

(11) Correction for Wave Direction

In cases where waves act obliquely to the breakwater alignment, the extent to which the incident wave angle affects the stability of the armor stones has not been investigated sufficiently. However, according to the results of experiments carried out by Van de Kreeke, [37] in which the wave angles of 0º, i.e., direction of incidence is perpendicular to the breakwater alignment, 30º, 45º, 60º and 90º, i.e., direction of incidence is parallel to the normal line were adopted, the damage rate for a wave direction of 45º or smaller is more-or-less the same as that for a wave direction of 0º, and when the wave direction exceeds 60º, the damage rate decreases. Considering these results, when the incident wave angle is 45º or less, the required mass should not be corrected for wave direction. Moreover, Christensen et al.[38] have shown that stability increases when the directional spreading of waves is large.

(12) Strength of Concrete Blocks

In case of deformed shape concrete block, it is necessary not only to ensure that the block has a mass sufficient to be stable for the variable situation in respect of waves, but also to confirm that the block itself has sufficient structural strength.

(13) Stability of Armor Blocks in Reef Area

In general, a reef rises up at a steep slope from the relatively deep sea, and forms a relatively flat and shallow sea bottom. Consequently, when a large wave enters at such a reef, it breaks around the slope, and then the regenerated waves afterward propagate over the reef in the form of surge. The characteristics of waves over a reef are strongly dependent on not only the incident wave conditions but also the water depth over the reef and the distance from the shoulder of the reef. The stability of wave-dissipating concrete blocks situated on a reef also varies greatly due to the same reasons. Therefore the characteristics over a reef are more complicated than that in general cases. The stability of wave-dissipating concrete blocks situated on a reef must thus be examined based either on model experiments matching the conditions in question or on field experiences for sites having similar conditions.

(14) Stability of Wave-dissipating Blocks on Low Crest Sloping Breakwater

For a low crown sloping breakwater with wave-dissipating blocks and without supporting wall, it is necessary to note that the wave-dissipating blocks around its crown are easily damaged by waves.[39] For example, for detached breakwater composed of wave-dissipating blocks, unlike a caisson breakwater covered with wave-dissipating blocks, there is no supporting wall at the back and the crown is not high. This means that the concrete blocks near the crown in particular at the rear are easily damaged, and indeed such cases of block damage have been reported. In the case of a detached breakwater, it is pointed out that some kind of concrete blocks at the rear of the crown should have a larger size compared to the one at the front of the crown.

(15) Stability of Blocks on Steep Slope Seabed

In cases where the bottom slope is steep and waves break in a plunging wave form, a large wave force may act on the blocks, depending on their shapes. Therefore, appropriate examination should be carried out, considering this fact.[40]

(16) High-density Blocks

The required mass of blocks that are made of high-density aggregate may also be determined using the Hudson's formula with the stability number shown in equation **(1.7.1)**. As shown in the equation, high-density blocks have a high stability, so a stable armor layer can be made using relatively small blocks.[41]

(17) Effect of Structural Conditions

The stability of wave-dissipating blocks varies depending on structural conditions and on the method of placement, such as regular or random placement etc. According to the results of experiments under conditions of random placement over the entire cross section and regular two-layer placement on a stone core, the regular placement with good interlocking had remarkably higher stability in almost all cases.[32] Provided, however, that if the layer

thickness of the blocks is minimal and the permeability of the core material is low, conversely, the stability of the blocks decreases in some cases.[42)]

The stability of wave-dissipating blocks is also affected by the crown width and crown height of the blocks. For example, according to the results of a number of experiments, there is a tendency of having greater stability when the crown width and the crown height are greater.

(18) Standard Method of Hydraulic Model Tests

The stability of concrete blocks is influenced by a very large number of factors, and so it has still not been sufficiently elucidated. This means that when actually verifying the performance, it is necessary to carry out studies using model experiments, and it is needed to progressively accumulate the results of such tests. The following points should be noted when carrying out model experiments.

① It is standard to carry out experiments using random waves.

② For each particular set of conditions, the experiment should be repeated at least three times i.e., with three different wave trains. However, when tests are carried out by systematically varying the mass and other factors and a large amount of data can be acquired, one run for each test condition will be sufficient.

③ It is standard to study the action of 1000 waves in total of three runs for each wave height level. Even for the systematic experiments, it is desirable to apply more than 500 waves or so.

④ For the description of the extent of damage, in addition to the damage ratio which has been commonly used in the past, the deformation level or the relative damage level may also be used. The deformation level is suitable when it is difficult to count the number of armor stones or concrete blocks that have moved, while the degree of damage is suitable when one wishes to represent the damage to wave-dissipating blocks. The damage rate is the ratio of the number of damaged armor units in an inspection area to the total number of armor units in the same inspection area. The inspection area is taken from the elevation of wave runup to whichever is shallower, the depth of 1.5H below the still water level or to the bottom elevation of the armor layer, where the wave height H is inversely calculated from the Hudson's formula by inputting the mass of armor units. However, for the deformation level and the degree of damage, there is no need to define the inspection area. For evaluating the damage rate, an armor block is judged to be damaged if it has moved over a distance of more than about 1/2 to 1.0 times its height.

(19) K_D Value Proposed by C.E.R.C.

Table 1.7.2 shows the K_D value of armor stones proposed by the Coastal Engineering Research Center, C.E.R.C., of the United States Army Corp of Engineers. This value is proposed for the breakwater trunk, parts other than the breakwater head, in the 1984 Edition of the C.E.R.C.'s **Shore Protection Manual**.[43)] In the table, the values not in parenthesis are based on experiment results by regular waves, and it is considered that those corresponds to 5% or less of the damage rate due to action of random waves. The values in parentheses are estimated values. For example, the value (1.2) for rounded rubble stones which are randomly placed in two-layer under the breaking wave conditions is given as the value which is half of 2.4, because the K_D value of two-layer angular rubble stones under the breaking waves condition is 1/2 that under the non-breaking wave conditions.

However, in cases where the wave height of regular waves corresponds to the significant wave height, the wave which is close to the maximum wave height of random waves acts continuously under the breaking wave condition in the regular wave experiments. Therefore, the regular wave experiment under the breaking wave condition falls into an extremely severe state in comparison with that under the non-breaking wave conditions. In random waves experiments, as described previously, it is considered that so long as the significant wave height is a standard, as the breaking wave conditions gets severe, conversely, K_D has a tendency to increase. Thus, at least it is not necessary to reduce the value of K_D under the breaking wave conditions.

Table 1.7.2 K_D Value of Rubble Stones Proposed by C.E.R.C. (Breakwater Trunk)

Type of armor	Number of layers	Placement method	K_D		cotα
			Breaking waves	Non-breaking waves	
Rubble stones (rounded)	2 3 or more	Random 〃	(1.2) (1.6)	2.4 (3.2)	1.5–5.0 〃
Rubble stones (angular)	2 3 or more	〃 〃	2.0 (2.2)	4.0 (4.5)	〃 〃

( ) shows estimated values.

1.7.2 Required Mass of Armor Stones and Blocks in Composite Breakwater Foundation Mound against Waves

(1) General

The required mass of armor stones and blocks covering the foundation mound of a composite breakwater varies depending on the wave characteristics, the water depth where the facility is placed, the shape of the foundation mound such as thickness, front berm width and slope angle etc., and the type of armor unit, the placement method, and the position, breakwater head or breakwater trunk etc. In particular, the effects of the wave characteristics and the foundation mound shape are more pronounced than that on the armor stones and blocks on a sloping breakwater. Adequate consideration should also be given to the effects of wave irregularity. Accordingly, the required mass of armor stones and blocks on the foundation mound of composite breakwater shall be determined by performing hydraulic model experiments or proper calculations using an appropriate equation in reference with the results of past research and actual experiences in the field. Provided, however, that the stability of the armor units covering the foundation mound of a composite breakwater is not necessarily determined purely by their mass. Depending on the structure and the arrangement of the armor units it may be possible to achieve stability even when the armor units are relatively small.

(2) Basic Equation for Calculation of Required Mass

As the equation for calculation of the required mass of armor stones and blocks in the foundation mound of a composite breakwater, Hudson's formula with the stability number N_S, as shown in the following equation, can be used in the same manner as with armor stones and blocks on sloping breakwater. In this equation, the symbol γ is a partial safety factor for its subscript, and the subscripts k and d show the characteristic value and design value, respectively. For the partial safety factors γ_{NS} and γ_H in the equation, 1.0 may be used. This partial safety factor is the value in cases where the limit value of the damage rate is 1% or the limit value of the degree of damage is 0.3.

$$M_d = \frac{\rho_r H_d^{\,3}}{N_{sd}^{\,3}\left(S_r - 1\right)^3} \tag{1.7.1}$$

This equation was widely used as the basic equation for calculating the required mass of the foundation mounds of upright walls by Brebner and Donnelly.[44] In Japan, it is also called Brebner-Donnelly's formula. Because it has a certain degree of validity, even from a theoretical standpoint, it can also be used as the basic equation for calculating the required mass of armor unit on the foundation mound of a composite breakwater.[45] Provided, however, that the stability number N_S varies not only with the water depth, the wave characteristics, the shape of the foundation mound, and the characteristics of the armor units, but also with the position of placement, breakwater trunk, breakwater head etc. Therefore, it is necessary to assign the stability number N_S appropriately based on model experiments corresponding to the conditions. Moreover, the wave height used in the performance verification is normally the significant wave height, and the waves used in the model experiments should be random waves.

(3) Stability Number for Armor Stones

The stability number N_S may be obtained using the method proposed by Inagaki and Katayama,[46] which is based on the work of Brebner and Donnelly and past damage case of armor stones. However, the following formulas proposed by Tanimoto et al.[45] are based on the current velocity in the vicinity of the foundation mound and allow the incorporation of a variety of conditions. These formulas have been extended by Takahashi et al.[47] so as to include the effects of wave direction, and thus have high applicability.

(a) Extended Tanimoto's formulas

$$N_s = \max\left\{1.8,\ 1.3\frac{1-\kappa}{\kappa^{1/3}}\frac{h'}{H_{1/3}} + 1.8\exp\left[-1.5\frac{(1-\kappa)^2}{\kappa^{1/3}}\frac{h'}{H_{1/3}}\right]\right\} \ :\ B_M / L' < 0.25 \tag{1.7.8}$$

$$\kappa = \kappa_1 (\kappa_2)_B \tag{1.7.9}$$

$$\kappa_1 = \frac{4\pi h' / L'}{\sinh(4\pi h' / L')} \tag{1.7.10}$$

$$(\kappa_2)_B = \max\left\{\alpha_s \sin^2\beta \cos^2(2\pi\ell\cos\beta / L'),\ \cos^2\beta \sin^2(2\pi\ell\cos\beta / L')\right\} \tag{1.7.11}$$

where

h' : water depth at the crown of rubble mound foundation excluding the armor layer (m) (see **Fig. 1.7.4**)

ℓ : in the case of normal wave incidence, the berm width of foundation mound B_M (m)

in the case of oblique wave incidence, either B_M or B'_M , whichever gives the larger value of $(\kappa_2)_B$ (see **Fig. 1.7.4**)

L' : wavelength corresponding to the design significant wave period at the water depth h' (m)

α_s : correction factor for when the armor layer is horizontal (=0.45)

β : incident wave angle, angle between the line perpendicular to the breakwater face line and the wave direction, no angle correction of 15º is applied (see **Fig. 1.7.5**)

$H_{1/3}$: design significant wave height (m)

The validity of the above formulas have been verified for the breakwater trunk for oblique wave incidence with an angle of incidence of up to 60º.

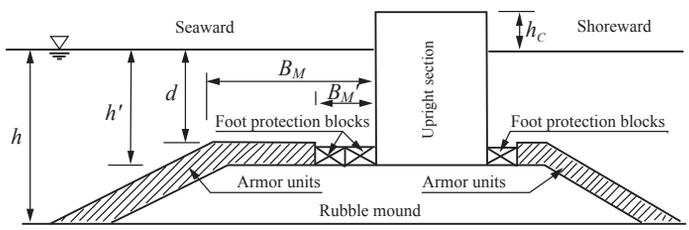

Fig. 1.7.4 Standard Cross Section of a Composite Breakwater and Notations

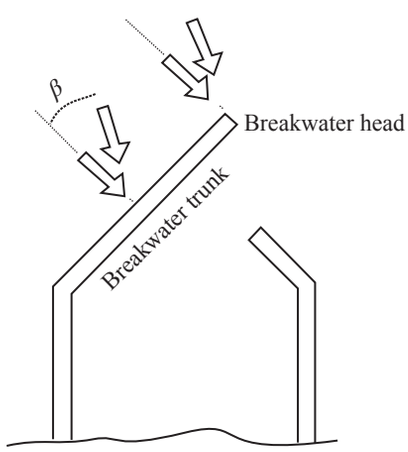

Fig. 1.7.5 Effects of Shape of Breakwater Alignment and Effects of Wave Direction

(b) Stability Number When a Certain Amount of Damage is Permitted

Sudo et al. have carried out stability experiments for the special case such that the mound is low and no wave breaking occurs. They investigated the relationship between the number of waves N and the damage rate, and proposed the following equation that gives the stability number N_S* for any given number of waves N and any given damage rate D_N (%).

$$N_S^* = N_S\left[D_N / \exp\{0.3(1-500/N)\}\right]^{0.25} \tag{1.7.12}$$

where

N_S is the stability number given by the Tanimoto's formula when $N = 500$ and the damage rate is 1%. In the performance verification, it is necessary to take $N = 1000$ considering the progress of damage, while the damage rate 3% to 5% can be allowed for a 2-layer armoring. If $N = 1000$ and $D_N = 5\%$, then $N_S^* = 1.44N_S$. This means that the required mass decreases to about 1/3 of that required for $N = 500$ and $D_N = 1\%$.

(4) Stability Number for Concrete Units

The stability number N_S for concrete blocks varies according to the shape of the block and the method of placement. It is thus desirable to evaluate the stability number by means of hydraulic model experiments.[49), 50)] When carrying out such experiments, it is best to employ random waves.

Based on the calculation method proposed by Tanimoto et al.,[45] Fujiike et al.[51] newly introduced reference stability number, which is a specific value for blocks, and separating the terms which is determined by the structural conditions of the composite breakwater etc., and then, presented the following equation regarding the stability number for armor blocks in cases where wave incidence is perpendicular.

$$N_S = N_{S0} \max\left\{ 1.0,\ A\frac{1-\kappa}{\kappa^{1/2}}\frac{h'}{H_{1/3}} + \exp\left[-0.9\frac{(1-\kappa)^2}{\kappa^{1/2}}\frac{h'}{H_{1/3}}\right] \right\} \tag{1.7.13}$$

$$\kappa = \kappa_1 (\kappa_2)_B \qquad \text{refer (1.7.9)}$$

$$\kappa_1 = \frac{4\pi h'/L'}{\sinh(4\pi h'/L')} \qquad \text{refer (1.7.10)}$$

$$(\kappa_2)_B = \begin{cases} \sin^2 \dfrac{2\pi B_M}{L'} & \left(\dfrac{B_M}{L'} \le 0.15\right) \\ 1.309 - \sin^2 \dfrac{2\pi B_M}{L'} & \left(0.15 < \dfrac{B_M}{L'} \le 0.25\right) \\ 0.309 & \left(0.25 < \dfrac{B_M}{L'}\right) \end{cases} \tag{1.7.14}$$

where

N_{S0} : reference stability number

A : constant determined based on wave force experiments (= 0.525)

(5) Conditions for Application of Stability Number to Foundation Mound Armor Units

In cases where the water depth above the armor units on the mound is shallow, wave breaking often causes the armor units to become unstable. Therefore, the stability number for foundation mound armor units shall be applied only when $h'/H_{1/3}>1$, and it is appropriate to use the stability number for armor units on a slope of a slope structure when $h'/H_{1/3} \le 1$. The stability number for armor stones in the Tanimoto's formulas have not been verified experimentally in cases where $h'/H_{1/3}$ is small. Accordingly, when $h'/H_{1/3}$ is approximately 1, it is preferable to confirm the stability number by hydraulic model experiments.

On the other hand, Matsuda et al.[52] carried out model experiments in connection with armor blocks, including the case in which $h'/H_{1/3}$ is small and impulsive waves act on the blocks, and proposed a method that provides a lower limit of the value of κ corresponding to the value of α_I in the case where the impulsive breaking wave force coefficient α_I is large.

(6) Armor Units Thickness

Two-layers are generally used for armor stones. It may be acceptable to use only one layer provided that consideration is given to examples of armor units construction and experiences of damaged armor units. It also may be possible to use one layer by setting the severe damage rate of 1% for N=1000 acting waves in equation **(1.7.12)**. One layer is generally used for armor blocks. However, two layers may also be used in cases where the shape of the blocks is favorable for two-layer placement or sea conditions are severe.

(7) Armor Units for Breakwater Head

At the head of a breakwater, strong currents occur locally near the corners at the edge of the upright section, meaning that the armor units become liable to move. It is thus necessary to verify the extent to which the mass of armor units should be increased at the breakwater head by carrying out hydraulic model experiments. If hydraulic model experiments are not carried out, it should increase the mass to at least 1.5 times that at the breakwater trunk. As the extent of the breakwater head in the case of caisson type breakwater, the length of one caisson may be usually adopted. The mass of the armor stones at the breakwater head may also be calculated using the extended Tanimoto's formula. Specifically, for the breakwater head, the velocity parameter κ in equation **(1.7.9)** should be rewritten as follows:

$$\kappa = \kappa_1 (\kappa_2)_T \tag{1.7.15}$$

$$(\kappa_2)_T = 0.22 \tag{1.7.16}$$

Note however that if the calculated mass turns out to be less than 1.5 times that for the breakwater trunk, it is preferable to set it to 1.5 times that for the breakwater trunk.

(8) Armor Units at Habor Side

It is preferable to decide the necessity and required mass of armor units at the harbor side, not only referring to past examples, but also performing hydraulic model experiments if necessary and considering the waves at the harbor side, the wave conditions during construction work and wave overtopping etc.

(9) Reduction of Mass of Armor

The equations for calculation of the required mass of armor units are normally applicable to the horizontal parts and the top of slope. In cases where the mound thickness is minimal, armor units of the entire slope have the same mass in many cases. However, in cases where the mound is thick, the mass of armor units places on the slope in deep water may be reduced.

(10) Foundation Mound Armor Units in Breakwaters Covered with Wave-dissipating Blocks

In the case of breakwaters covered with wave-dissipating blocks, the uplift pressure acting on the armor and the current velocities in the vicinity of the mound are smaller than those of conventional composite breakwaters. Fujiike et al.[51] carried out model experiments in connection with the stabilities of both the armor units of the conventional composite breakwaters and the breakwaters covered with wave-dissipating blocks, and proposed a method of multiplicating equation **(1.7.9)** by the compensation rate. Namely,

$$\kappa = C_R \kappa_1 (\kappa_2)_B \tag{1.7.17}$$

where

C_R : breakwater shape influence factor, it may be used 1.0 for conventional composite breakwaters approximately 0.4 for breakwaters covered with wave-dissipating blocks.

(11) Flexible Armor Units

Use of bag-type foot protection units which consist of synthetic fiber net filled with stones as the armor units on the foundation mound has various advantages: large stones are not required, and mound leveling is not virtually needed because they have high flexibility and can adhere to the irregular sea bed. Shimosako et al.[53] proposed a method of calculating the required mass of armor units on the foundation mound using bag-type foot protection units, and also examined their durability.

1.7.3 Required Mass of Armor Stones and Blocks against Currents

(1) General

The required mass of rubble stones and other armor materials for foundation mounds to be stable against water currents may be generally be determined by appropriate hydraulic model experiments or calculated using the following equation. In this equation, the symbol γ is a partial safety factor for its subscript, and the subscripts k and d show the characteristic value and the design value, respectively.

$$M_d = \frac{\pi \rho_r U_d^{\,6}}{48 g^3 (y_d)^6 (S_r - 1)^3 (\cos\theta - \sin\theta)^3} \tag{1.7.18}$$

where

M : stable mass of rubble stones or other armor material (t)
ρ_r : density of rubble stones or other armor material (t/m^3)
U : current velocity of water above rubble stones or other armor material (m/s)
g : gravitational acceleration (m/s^2)
y : Isbash's constant, for embedded stones, 1.20; for exposed stones, 0.86
S_r : specific gravity of rubble stones or other armor material relative to water
θ : slope angle in axial direction of water channel bed (^{o})

The design values in the equation may be calculated by using the following equations. For the partial safety factors γ_U and γ_y , 1.0 may be used.

$$U_d = \gamma_U U_k, \;\; y_d = \gamma_y y_k$$

This equation was proposed by the C.E.R.C. for calculation of the mass of rubble stones required to prevent scouring by tidal currents and is called Isbash's formula.[43] As also shown in the equation, attention should be given to the fact that the required mass of armor units against currents increases rapidly as the current velocity increases. The required mass also varies depending on the shape and density of the armor units etc.

(2) Isbash's Constant

Equation **(1.7.18)** was derived considering the balance of the drag force of the flow acting on a spherical object on a slope and the friction resistance force. The constant y is Isbash's constant. The values of 1.20 and 0.86 for embedded stones and exposed stones, respectively, are given by Isbash, and are also cited in Reference 54). It should be noted that, because equation **(1.7.18)** was obtained considering the balance of forces in a steady flow, it is necessary to use rubble stones with a larger mass in the place where strong vortices will be generated.

(3) Armor Units on Foundation Mound at Openings of Tsunami Protection Breakwaters

Iwasaki et al.[55] conducted experiments on 2-dimensional steady flows for the case in which deformed concrete blocks are used as the armor units on a foundation mound in the opening of the submerged breakwaters of tsunami protection breakwater, and obtained a value of 1.08 for Isbash's constant in equation **(1.7.18)**. Tanimoto et al.[56] carried out a 3-dimensional plane experiment for the opening of breakwaters, clarifying the 3-dimensional flow structure near the opening, and also revealed the relationship between Isbash's constant and the damage rate for the cases where stone materials and deformed concrete blocks are used as the armor units.

1.8 Scouring and Washing-out

Public Notice

Performance Criteria Common to Structural Members

Article 22

3 In cases where the effects of scouring of the seabed and sand outflow on the integrity of structural members may impair the stability of the facilities, appropriate countermeasures shall be taken.

[Commentary]

(1) Scouring and Washout (serviceability)
In cases where scouring of the foundation of facilities concerned and ground and outflow of sand from the ground behind structures might impair the stability of the facilities, appropriate countermeasures against scouring and countermeasures against washout must be taken, considering the structural type of the objective facilities.

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2 Foundations

2.1 General Comments

(1) The foundation structures of the port facilities shall be selected appropriately, giving due consideration to the importance of the facilities and soil conditions of the foundation ground.

(2) When the stability of the foundation structures seems to be an obstacle, countermeasures such as pile foundation or soil improvement, etc. shall be applied as necessary.

(3) When the foundation ground is soft, excessive settlement or deformation may arise owing to the lack of the bearing capacity. When the foundation ground consists of loose sandy soil, liquefaction due to action of ground motion causes the structure failure or significantly damage its functions. In such cases, the stress in subsoil by the weight of structures needs to be reduced or the foundation ground should be improved.

(4) For the stability of foundations, **2.2 Shallow Spread Foundations**, and **2.3 Deep Foundations**, or **3 Stability of Slopes** can be used as reference. For settlement of foundations, **2.5 Settlement of Foundations**, and for liquefaction due to action of ground motion, **Part II**, **Chapter 6 Ground Liquefaction** can be used as reference. For the performance verification of pile foundations, **2.4 Pile Foundations** can be used as reference. In cases where it is necessary to conduct the performance verification for ground motion, the verification shall be performed corresponding to the characteristics of the respective foundations.

(5) Methods of Reducing Ground Stress
The following are methods of reducing ground stress due to the weight of structures.

① Reduction of the weight of the structure itself

② Expansion of the area of the bottom of the structure

③ Use of a pile foundation
Shear stress due to the facilities may be reduced by the counterweight method.

(6) Method of Soil Improvement
For method of soil improvement, **4 Soil Improvement Methods** can be used as reference.

2.2 Shallow Spread Foundations

2.2.1 General

(1) When the embedment depth of the foundation is less than the minimum width of the foundation, the foundation may generally be examined as a shallow spread foundation.

(2) In general, the bearing capacity of a foundation is the sum of the bottom bearing capacity and the side resistance of the foundation. Bottom bearing capacity is determined by the value of the pressure applied to the foundation bottom considered necessary to cause plastic flow in the ground. The side resistance of a foundation is the frictional resistance or the cohesion resistance acting between the sides of the foundation and the surrounding soil. Although considerable research has been done on the bottom bearing capacity of foundations, relatively little research has been done on side resistance. If the embedment depth of the foundation is less than the minimum width of the foundation, in the case of so-called shallow spread foundations, the magnitude of the side resistance will be small in comparison with that of the bottom bearing capacity. Therefore, it is not necessary to consider the side resistance in such cases.

(3) When an eccentric and inclined action acts on the foundation, **2.2.5 Bearing Capacity for Eccentric and Inclined Actions** can be used as reference.

2.2.2 Bearing Capacity of Foundations on Sandy Ground

(1) The following equation can be used to calculate the design value of the bearing capacity of the foundations on sandy ground. In this case, appropriate values corresponding to the characteristics of the facilities can be adopted as the partial factors. In general, 0.4 or less can be considered an appropriate partial factor γ_R.

$$q_d = \gamma_R\left(\beta\rho_{1d} g \frac{B}{2} N_{\gamma d} + \rho_{2d} gD\left(N_{q d} - 1\right)\right) + \rho_{2d} gD \tag{2.2.1}$$

where

q_d : design value of foundation bearing capacity considering buoyancy of submerged part (kN/m^2)
γ_R : partial factor for bearing capacity of sandy ground

β : shape factor of foundation, see **Table 2.2.1**

$\rho_{1d}g$: design value of unit weight of soil of ground below foundation bottom or unit weight in water, if submerged (kN/m³)

B : minimum width of foundation (m)

N_{rd}, N_{qd} : design values obtained by multiplying partial factors γ_{Nq} and $\gamma_{N\gamma}$ by the characteristic values of the bearing capacity factor N_{qk} and $N_{\gamma k}$ (see **Fig. 2.2.1**), [1] respectively. The characteristic values of the bearing capacity factor are expressed by the following equations.

$$N_{q_k} = \frac{1+\sin\phi_k}{1-\sin\phi_k}\exp(\pi\tan\phi_k) \quad \text{(Prandtl's solution)}$$

$$N_{\gamma_k} = (N_{q_k} - 1)\tan(1.4\phi_k) \quad \text{(Meyerhof's solution)}$$

$\rho_{2d}g$: design value of unit weight of soil of ground above foundation bottom, or unit weight in water, if submerged (kN/m³)

D : embedment depth of foundation in ground (m)

(2) When the actions on the foundation increase, first, settlement of the foundation occurs in proportion to the actions. However, when the actions reach a certain value, settlement increases suddenly and shear failure of the ground occurs. The intensity of the load required to cause this shear failure which is obtained by dividing the load by the contact area is called the ultimate bearing capacity of the foundation. The bearing capacity of the foundation can be calculated by multiplying the ultimate bearing capacity obtained from the bearing capacity formula by the partial factor γ_R.

Table 2.2.1 Shape Factors

Shape of foundation	Continuous	Square	Round	Rectangular
β	1	0.8	0.6	1–0.2(B/L)

B: length of short side of rectangle, L: length of long side of rectangle

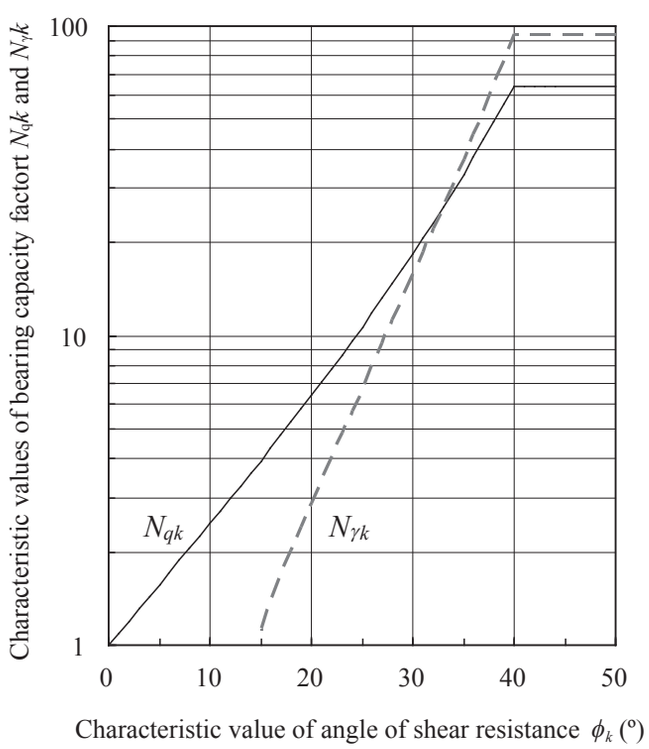


Fig. 2.2.1 Relationship between Bearing Capacity Factors N_{rk} and N_{qk} and Angle of Shear Resistance ϕ_k

2.2.3 Bearing Capacity of Foundations on Cohesive Soil Ground

(1) In calculations of the design values for foundations of cohesive soil ground in cases where the undrained shear strength increases linearly with depth, the following equation can be used. In this case, an appropriate value corresponding to the characteristics of the facilities shall be selected for the partial factor γ_R.

$$q_d = \gamma_R N_{c0_d}\left(1 + n\frac{B}{L}\right)c_{0_d} + \rho_{2_d} gD \tag{2.2.2}$$

where

q_d : design value of foundation bearing capacity considering buoyancy of submerged part (kN/m²)
γ_R : partial factor for bearing capacity of cohesive soil ground
N_{c0d} : design value of bearing capacity factor for continuous foundation
n : shape factor of foundation, see **Fig. 2.2.2**
B : minimum width of foundation (m)
L : length of foundation
c_{0d} : design value of undrained shear strength of cohesive soil at bottom of foundation (kN/m²)
$\rho_{2d}g$: design value of unit weight of soil of ground above foundation bottom, or unit weight in water, if submerged (kN/m³)
D : embedment depth of foundation in ground (m)

(2) As the undrained shear strength of cohesive soil ground in port areas usually increases linearly with depth, the bearing capacity of foundation should be calculated by the equation that takes account of the effect of shear strength increase.

(3) Equation for Calculating Design Value of Bearing Capacity of Cohesive Soil Ground Considering Strength Increase in Depth Direction

The design value N_{c0d} of the bearing capacity factor in equation **(2.2.2)** can be calculated using **Fig. 2.2.2**. Here, k is the strength increase rate in the depth direction. If the surface strength is assumed to be c_0, the strength at depth z is expressed by $c_0 + kz$. As the partial factor for the bearing capacity γ_R, an appropriate value of 0.66 or less can be used generally, but in cases where there is a possibility that slight settlement or deformation of the ground may remarkably impair the functions of superstructure, as in the case of crane foundations, a value of no more than 0.4 shall be used.

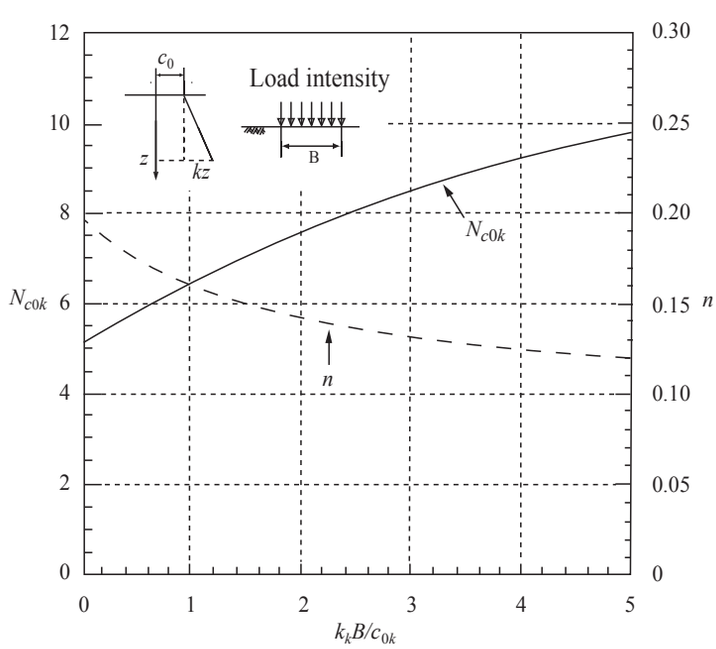


Fig. 2.2.2 Relationship of Bearing Capacity factor N_{cok} of Cohesive Soil Ground in which Strength Increases in Depth Direction and Shape Factor n

(4) Practical Equation for Calculating Design Value of Bearing Capacity
Based on the bearing capacity factors shown in **Fig. 2.2.2**, the design value of the bearing capacity of foundations in case of continuous foundations can be calculated using the practical equation shown in equation **(2.2.3)** in the range where $k_{kB}/c_{0k} \leq 4$. The symbols used are the same as in equation **(2.2.2)**.

$$q_d = \gamma_R \gamma_{N_{c0}} \left(1.018 k_k B + 5.14 c_{0_k}\right) + \rho_{2_k} g D \quad \text{(provided, however, that kkB/c0k≤4)} \qquad (2.2.3)$$

2.2.4 Bearing Capacity of Multi-layered Ground

(1) Examination of stability for the bearing capacity when the foundation ground has a multi-layered structure can be performed by circular slip failure analysis. Assuming the overburden pressure above the level of the foundation bottom as the surcharge, circular slip failure analysis is performed by the modified Fellenius method for an arc passing through the edge of the foundation, as shown in **Fig. 2.2.3**. As the partial factor γ_R for the analysis method, 0.66 or less can be used generally, but in cases where settlement will have a large effect on the functions of the facilities like crane, it is preferable to use a value of no more than 0.4.

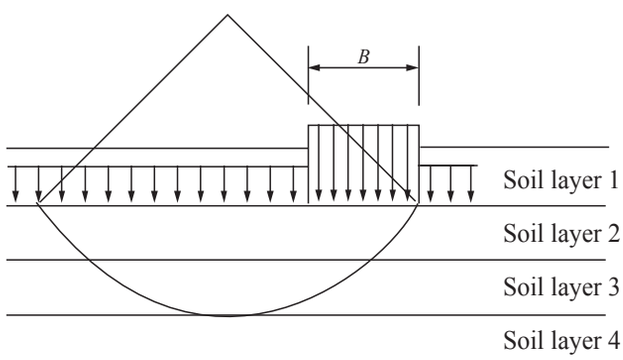


Fig. 2.2.3 Calculation of Bearing Capacity of Multi-layered Ground by Circular Slip Failure Analysis

(2) If the cohesive soil layer thickness H is significantly less than the smallest width of the foundation B (i.e., $H < 0.5B$), a punching shear failure, in which the cohesive soil layer is squeezed out between the surcharge plane and the bottom of cohesive soil layer, is liable to occur. The bearing capacity against this kind of squeezed-out failure can be calculated by the following equation [4)]

$$q_d \geq \gamma_R (4.0 + 0.5 B/H) c_{u_d} + \rho_{2_d} g D \qquad (2.2.4)$$

where

- q_d : design value of bearing capacity of foundation considering the buoyancy of the submerged part (kN/m2)
- B : smallest width of foundation (m)
- H : thickness of cohesive soil layer (m)
- c_{ud} : design value of mean undrained shear strength in layer of thickness H (kN/m2)
- $\rho_{2d}g$: design value of unit weight of soil above the level of foundation bottom or unit weight in water, if submerged (kN/m3)
- γ_R : partial factor for bearing capacity
- D : embedded depth of foundation (m)

2.2.5 Bearing Capacity for Eccentric and Inclined Actions

(1) Examination of the bearing capacity for eccentric and inclined actions acting on the foundation ground of gravity-type structures can be performed by circular slip failure analysis with the simplified Bishop method using the following equation. In this equation, the symbol γ is the partial factor for its subscript, and the subscripts k and d indicate the characteristic value and design value, respectively. In this case, the partial factor shall be an appropriate value corresponding to the characteristics of the facilities. It is necessary to set the strength constant of the ground, the forms of the actions, and other factors appropriately considering the structural characteristics of the facilities.

$$\gamma_{F_f} F_f = \frac{\sum \left\{ \left[c_d S + (W'_d + q_d) \tan \phi_d \right] \frac{\sec \theta}{1 + \tan \theta \tan \phi_d / \gamma_{F_f} F_f} \right\}}{\sum \left\{ (W_d + q_d) \sin \theta + a P_{H_d} / R \right\}} \tag{2.2.5}$$

where

R : radius of in circular slip failure (m)

c_d : in case of cohesive soil ground, design value of undrained shear strength, and in case of sandy ground, design value of apparent cohesion in drained condition (kN/m^2)

W'_d : design value of effective weight to discrete segment per unit of length, submerges unit weight if submerged (kN/m)

q_d : design value of vertical action from top of discrete segment (kN/m)

θ : angle of bottom of discrete segment to horizontal (º)

ϕ_d : in case of cohesive soil ground, the value shall be 0, and in case of sandy ground, design value of angle of shear resistance in drained condition (º)

W_d : design value of total weight of discrete segment per unit of length, namely total weight of soil and water (kN/m)

P_{Hd} : design value of horizontal action on lumps of earth in circular slip failure (kN/m)

a : arm length from the center of circular slip failure at position of action of an external action H

S : width of discrete segment (m)

γ_{Ff} : partial factor for analysis method

Based on equation **(2.2.5)**, γ_{Ff} is calculated, and stability is verified by the verification parameter $F_f \geq 1$. The design values in the equation can be calculated by the following equations. Provided, however, that in cases where partial factors are given by structural type, the partial factor for the part concerned shall be used. In other cases where partial factors are not particularly designated, the value of the partial factor γ can be set at 1.00.

$$c_d = \gamma_c c_k,\ W'_d = \gamma_{W'} W'_k, q_d = \gamma_q q_k,\ \phi_d = \tan^{-1}(\gamma_{\tan\phi} \tan\phi_k),\ P_{Hd} = \gamma_{P_H} P_{Hk} \tag{2.2.6}$$

(2) In gravity-type quaywalls and gravity-type breakwaters, actions due to self weight, earth pressure, wave force, and ground motion shall be considered. However, the resultant of these actions is normally both eccentric and inclined. Therefore, examination for eccentric and inclined actions is necessary in examination of the bearing capacity of foundations. Here, eccentric and inclined action means an action with an inclination ratio equal to or greater than 0.1.

(3) Because normal gravity-type structures are two-layered structures with a rubble mound layer on foundation ground, an examination method which adequately reflects this feature is necessary. The fact that circular slip failure calculations by the Bishop method, simplified Bishop method, accurately express stability for bearing capacity has been confirmed in a series of research results, including laboratory model experiments, in-*situ* loading experiments, and analysis of the existing breakwaters and quaywalls, and this method is therefore used as a general method.[5)]

(4) Analysis of Bearing Capacity by Circular Slip Failure Analysis based on the Bishop Method
Analysis through circular slip failure analysis based on the Bishop method is more precise than the analysis based on the modified Fellenius method, except when a vertical action exerts on horizontally layered sandy ground. Therefore, the circular slip failure analysis by the Bishop method is applied under the condition that eccentric and inclined actions exert act. As shown in **Fig. 2.2.4 (a)**, the start point of the slip surface is set symmetrical about the acting point of resultant load to one of the foundation edges that is closer to the load acting point. In this case, the vertical action exerting on the rubble mound is converted into uniformly distributed load acting on the width between fore toe of the bottom and the start point of the slip surface as indicated in **Fig. 2.2.4 (b)** and **(c)**. The horizontal force is assumed to act at the bottom of structure. When calculating the bearing capacity during an earthquake, seismic force is assumed not to act on the rubble mound and the ground.

When subgrade reaction has a trapezoidal distribution; $q=\frac{(p_1+p_2)}{4\,b'}B$

When subgrade reaction has a triangular distribution; $q=\frac{p_1 b}{4\,b'}$

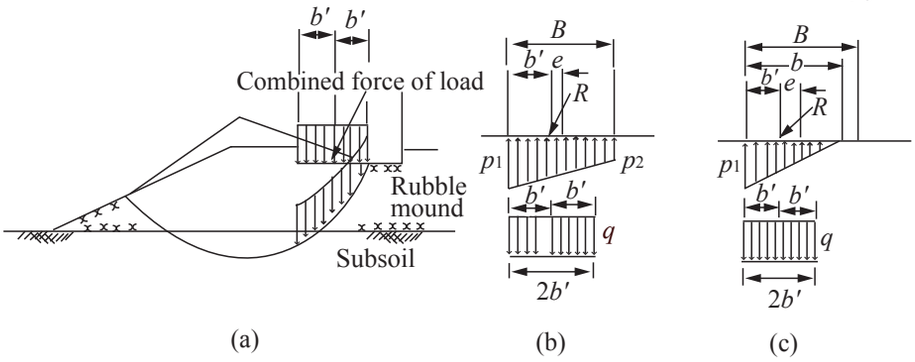


Fig. 2.2.4 Analysis of Bearing Capacity for Eccentric and Inclined Actions

(5) Verification Parameter and Partial Factors

① The verification parameter is expressed by the ratio of the sliding moment due to actions and the weight of earth and the resistant moment due to shear resistance (see **3.2.1 Stability Analysis by Circular Slip Failure Surface**). As general values of the partial factors for the analysis method, the values shown in **Table 2.2.2** can be used. Provided, however, that in cases where partial factors are indicated by structural type, the partial factor for the part concerned shall be used.

② Regarding actions on breakwaters due to ground motion, few examples of damage are available, and the degree of damage is also small. As the reasons for this, in many cases actions due to ground motion are basically equal in the harbor direction and the outer sea direction, and large displacement does not occur due to the short duration of the action. Accordingly, examination of the bearing capacity due to actions of ground motion may be omitted in the case of ordinary breakwaters. Provided, however, that detailed examination by dynamic analysis is desirable for breakwaters where stability due to actions of ground motion may be a serious problem.

Table 2.2.2 Standard Values of Partial Factor γ_{Ff} in Analysis Method for Bearing Capacity for Eccentric and Inclined Actions (Bishop Method)

	Quaywalls	Breakwaters
Permanent situation	≤0.83	–
Variable situation for Level 1 earthquake ground motion	≤1.00	–
Variable situation for waves	–	≤1.00

Note) In case partial factors are indicated by structural type, the partial factor for the part concerned shall be used.

(6) Strength Parameters for Mound Materials and Foundation Ground

① Mound materials

Model and field experiments on bearing capacity subject to eccentric and inclined actions have verified that high precision results can be obtained by conducting circular slip failure analyses based on the simplified Bishop method, applying the strength parameters obtained by triaxial compression tests [5]. Large-scale triaxial compression test results of crushed stone have confirmed that the strength parameters of large diameter particles are approximately equal to those obtained from similar grained materials with the same uniformity coefficient [6]. Therefore, triaxial compression tests using samples with similar grained materials are preferably conducted in order to estimate the strength parameters of rubbles accurately. If the strength tests are not conducted, the values of cohesion c_D = 20 kN/m^2 and shearing resistance angle ϕ_D = 35° are applied as the standard strength parameters for rubbles generally used in port construction works.

The above standard values have been determined as safe side values based on the results of large-scale triaxial compression tests of crushed stones. The values have been proven appropriate from the analysis results of the bearing capacity of the existing breakwaters and quaywalls. It should be noted that cohesion c_D = 20 kN/m^2 as a strength parameter is the apparent cohesion, taking account of variations of the shear resistance angle

ϕ_D of crushed stones under variable confining pressures. **Fig. 2.2.5** shows the results of triaxial compression tests on various types of crushed stones and rubbles [5]. It shows that as the confining pressure increases, ϕ_D decreases due to particle crushing. The solid line in the figure represents the value under the assumption that the apparent cohesion is c_D = 20 kN/m² and the shear friction angle is ϕ_D= 35º. Here, the dependency of ϕ_D on the confining pressure is well described by taking the apparent cohesion into account. These standard values can be applied only to the stone material with an unconfined compressive strength in the mother rock of 30 MN/m² or more. If weak stones with the compressive strength of the mother rock of less than 30 MN/m² are used as a part of the mound, the strength parameters will be around c_D= 20 kN/m² and ϕ_D = 30º [7].

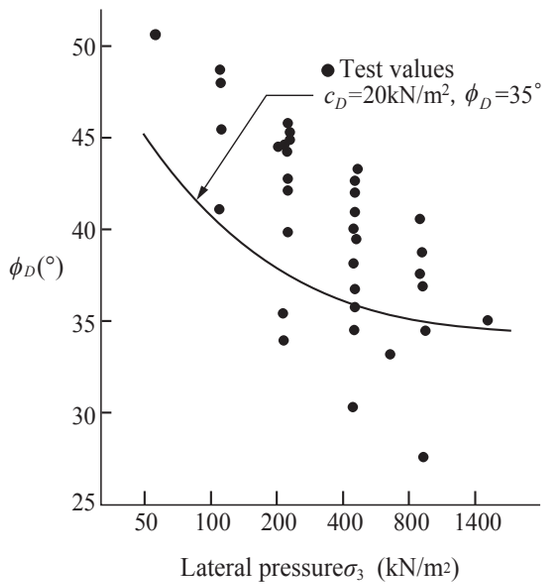


Fig. 2.2.5 Relationship between ϕ_D and Lateral Confining Pressure σ_3 and Apparent Cohesion

② Foundation ground

Foundations subject to eccentric and inclined actions often cause shallow surface slip failure. In these cases, it is important to evaluate the strength near the surface of foundation ground. If the foundation ground is sandy, the strength coefficient ϕ_D is usually estimated from *N*-value. The estimation formulas employed up to now have tended to underestimate ϕ_D in case of shallow sandy grounds. This is because no correction has been made regarding the effective surcharge pressure in-situ.

Fig. 2.2.6 collates the results of triaxial compression tests on undisturbed sand in Japan and presents a comparative study of the formulas proposed in the past. Even when the *N*-values are less than 10, shearing resistance angles of around 40º have been obtained. In many cases, the bearing capacity for eccentric and inclined actions is important on the performance verification which is not under the permanent situation but under dynamic external forces such as wave and seismic forces. In addition to the above and based on the results of bearing capacity analysis of the structures damaged in the past, the values given below are applied as the standard values of ϕ_D in foundation ground.

Sandy ground with *N*-value of less than 10 : ϕ_D= 40º
Sandy ground with *N*-value of 10 or more : ϕ_D= 45º

If the ground consists of cohesive soil, the strength may be determined by the method indicated in **Part II, Chapter 3, 2.3.3 Shear Characteristics**.

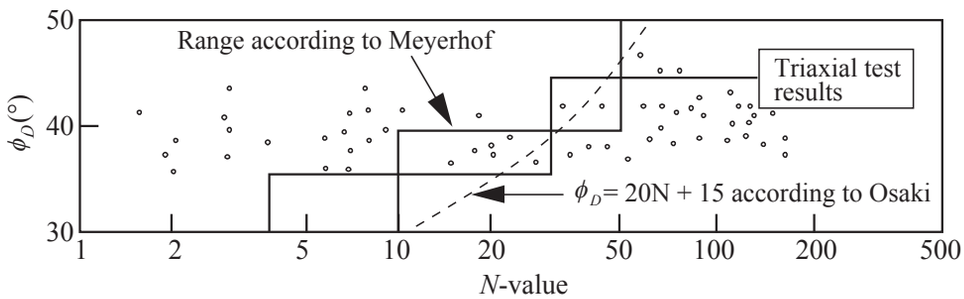


Fig. 2.2.6 Relationship Between *N*-value and ϕ_D Obtained by Triaxial Tests of Undisturbed Sand Samples

2.3 Deep Foundations

2.3.1 General

(1) When the penetration depth of a foundation is greater than the minimum width of the foundation, it shall be examined as a deep foundation. Means of distinguishing the deep foundations described here from pile foundations include the method of judging whether βL (L: embedment length of pile) $\leqq 1$ or not, based on calculations by the method proposed by Y. L. Chan, see **2.4.5 Static Maximum Lateral Resistance of Piles**.

(2) Foundations of the type described in (1) generally include the well, pneumatic caisson and continuous underground wall. For pile foundations, see **2.4 Pile Foundations**.

(3) Deep foundations support the superstructure stably by transmitting the action due to the heavy superstructure through the weak upper strata to the strong lower strata. Accordingly, it can normally be considered that vertical force is supported by the frictional resistance at the side surfaces of the foundation and the vertical bearing capacity at the bottom, and the horizontal force is supported by the passive resistance of the ground.

2.3.2 Characteristic Value of Vertical Bearing Capacity

(1) The characteristic value of the vertical bearing capacity of a deep foundation shall be set taking into account the soil conditions, the structural type, and the method of construction.

(2) Generally, the vertical bearing capacity of a deep foundation can be determined from the bearing capacity of the foundation bottom and the resistance of the foundation sides, as shown in equation **(2.3.1)**. However, in cases where the amount of displacement and/or deformation of the facilities may be a problem, the deformation of deep foundations should be estimated by assuming the ground behaves as a spring.

$$q_{u_k} = q_{u_{1_k}} + q_{u_{2_k}} \tag{2.3.1}$$

where

q_{uk} : characteristic value of vertical bearing capacity of deep foundation (kN/m^2)

q_{u1k} : characteristic value of bearing capacity of foundation bottom (kN/m^2)
see **2.2.2 Bearing Capacity of Foundations on Sandy Ground**, **2.2.3 Bearing Capacity of Foundations on Cohesive Soil Ground**

q_{u2k} : characteristic value of bearing capacity due to resistance of foundation sides (kN/m^2)

(3) The design value of the vertical bearing capacity of deep foundations shall consider a safety margin in the characteristic value of the vertical bearing capacity, as in equation **(2.3.2)**. The characteristic value of the foundation bottom bearing capacity determined as described in **2.2.2 Bearing Capacity of Foundations on Sandy Ground** and **2.2.3 Bearing Capacity of Foundations on Cohesive Soil Ground**, and the partial factor γ_a, which is used in cases where the characteristic value of the vertical bearing capacity is determined using equation **(2.3.3)** and equation **(2.3.5)**, as shown in the following, can generally be set at 0.4 or less for important facilities and 0.66 or less for other facilities.

$$q_{u_d} = \gamma_a q_{u_k} \tag{2.3.2}$$

where

q_{ud} : design value of vertical bearing capacity of deep foundation (kN/m^2)

q_{uk} : characteristic value of vertical bearing capacity of deep foundation (kN/m^2)

(4) Caution is required concerning the resistance of the sides of deep foundations, as there are cases in which the surrounding ground may be disturbed by construction and, as a result, adequate bearing capacity by side resistance cannot be expected, depending on the structural type and method of construction.

① The characteristic value of the bearing capacity due to the frictional resistance of the foundation sides in sandy ground can be calculated by equation **(2.3.3)**.

$$q_{u_{2_k}} = \left(1 + \frac{B}{L}\right)\frac{D^2}{B} K_{a_k} \gamma_{2_k} \mu_k \tag{2.3.3}$$

where

K_{ak} : characteristic value of coefficient of active earth pressure (δ = 0°), see **Part II, Chapter 5, 1 Earth Pressure**

γ_{2k} : characteristic value of unit weight of soil above level of foundation bottom, or submerged unit weight if submerged (kN/m^3)

D : penetration depth of foundation (m)
μ_k : characteristic value of coefficient of friction between foundation sides and sandy soil, $\mu_k = \frac{2}{3}\tan\phi_k$
ϕ_k : characteristic value of shear resistance angle (º)
B : width of foundation (m)
L : length of foundation (m)

q_{u2k} in equation (**2.3.3**), is obtained by dividing the total friction resistance by the bottom area of foundation. The total friction resistance is calculated as the product of the mean side friction strength $\overline{f}$ multiplying with the penetration depth D and the total contact surface area between the sandy soil and foundation sides. Equation (**2.3.4**) is generally used to calculate the mean side friction strength $\overline{f}$ corresponding to the penetration depth D.

$$\overline{f} = \frac{1}{D}\int_0^D \gamma z K_a \mu dz = \frac{1}{2}\gamma D K_a \mu \tag{2.3.4}$$

The friction angle between the foundation sides and sandy soil should not be greater than the shear resistance angle of soil ϕ, and it may be taken as (2/3) ϕ for the case between concrete and sandy soil.

② The characteristic value of bearing capacity due to the cohesive resistance of the foundation sides in cohesive soil ground can be calculated by equation **(2.3.5)**.

$$q_{u_{2_k}} = 2\left(1+\frac{B}{L}\right)\frac{D_c}{B}\overline{c_{ak}} \tag{2.3.5}$$

where
$\overline{c_{ak}}$: characteristic value of mean adhesion (mean value in embedded part) (kN/m2)
Dc : penetration depth of foundation below groundwater level (m)
B : width of foundation (m)
L : length of foundation (m)

In case of deep foundations in cohesive soil ground, there is generally a possibility of drying shrinkage during summer in the soil above the groundwater level; therefore, this soil is not considered to be an effective contact surface. Accordingly, the mean adhesion $\overline{c_a}$ in equation **(2.3.5)** should be the mean adhesion in the effective contact part.

As practical values of mean adhesion in cohesive soil, the values in **Table 2.3.1** can be used as reference.

Table 2.3.1 Relationship between Unconfined Compression Strength and Mean Adhesion of Cohesive Soil

(kN/m2)

Class of ground at foundation side	q_u	$\overline{c_a}$
Soft cohesive soil	20–50	–*)
Medium cohesive soil	50–100	6–12
Hard cohesive soil	100–200	12–25
Extremely hard cohesive soil	200–400	25–30
Consolidated cohesive soil	>400	>30

*Note) with soft cohesive soil, side resistance should not be considered.

(5) Consideration of Negative Skin Friction
In cases where the deep foundation penetrates through the consolidable ground and reaches the bearing layer, it is necessary to examine negative skin friction acting on the body. As the method of examination in this case, **2.4.3 [9] Examination of Negative Skin Friction** can be used as reference.

2.3.3 Horizontal Resistance Force of Deep Foundations

(1) The characteristic value of the lateral bearing capacity of a deep foundation shall be determined as appropriate taking into account soil conditions, structural characteristics, and the method of construction.

(2) The lateral bearing capacity of a deep foundation is governed by the horizontal subgrade reaction of the foundation sides and the vertical subgrade reaction at the bottom of foundation.

(3) The characteristic value of the horizontal resistance force of deep foundations can be determined from the passive earth pressure and ultimate bearing capacity.

(4) The design value of the horizontal resistance force of deep foundations should include a safety margin in the characteristic value, as in the following equation. When the characteristic value of the horizontal resistance force of a deep foundation is obtained by the method presented below, the partial factors shown in **Table 2.3.2** can generally be used.

$$F_{u_d} = \gamma_a F_{u_k} \tag{2.3.6}$$

where

F_{ud} : design value of horizontal resistance force of deep foundation (kN/m2)
F_{uk} : characteristic value of horizontal resistance force of deep foundation (kN/m2)
γ_a : partial factor

Table 2.3.2 Partial Factor γ_a

	Resistance force by passive earth pressure	Resistance force by vertical bearing capacity
Important facilities	0.66	0.40
Other facilities	0.90	0.66

(5) Calculation Method for Performance Verification

① When a resultant force at a bottom of foundation acts inside the core, namely the eccentricity of total resultant force acting at the bottom of foundation is within one-sixth of the foundation width from the central axis of the foundation, the maximum horizontal subgrade reaction p_1 and maximum vertical subgrade reaction q_1 can be estimated by assuming the distributions of horizontal and vertical subgrade reaction are assumed as in **Fig. 2.3.1.**

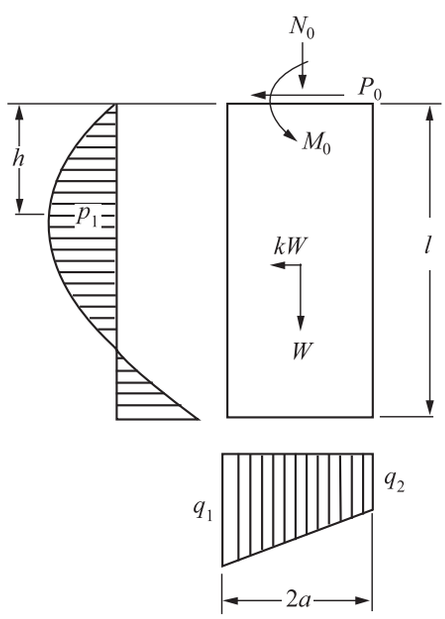


Fig. 2.3.1 When Resultant Force is inside the Core

② Assumption on the Distribution of Subgrade Reaction
The distribution of horizontal subgrade reaction shown in **Fig. 2.3.1** may be assumed as being a quadratic parabola with the subgrade reaction of 0 at the ground surface. This assumption is equivalent to the relationship between the displacement y and the subgrade reaction p of equation (**2.3.7**) when the foundation rotates as a rigid body.

$$p = kxy \tag{2.3.7}$$

where

p : subgrade reaction (kN/m2)
k : rate of increase in coefficient of horizontal subgrade reaction with depth (kN/m4)
x : depth (m)
y : horizontal displacement at depth x (m)

When a linear distribution is assumed for vertical subgrade reaction and a resultant force acting at the bottom of foundation is inside the core, the distribution of the vertical subgrade reaction becomes trapezoidal as shown in **Fig. 2.3.1**.

③ Conditions when vertical resultant is in the core and characteristic value for horizontal resistance force in such cases

The conditions for the case in which the vertical resultant at the bottom is in the core are expressed as in equation **(2.3.8)**.

$$\frac{N_0 + w_1\ell}{A} \geq \frac{3aK'\left(kw_1\ell^2 + 4P_0\ell + 6M_0\right)}{b\left(\ell^3 + 24\alpha K'\alpha^3\right)} \tag{2.3.8}$$

The maximum horizontal subgrade reaction p_1 (kN/m2) and the maximum vertical subgrade reaction q_1 (kN/m2) in this case are obtained by equations **(2.3.9)** and **(2.3.10)**, respectively.

$$p_1 = \frac{3\left\{kw_1\ell^4 + 3P_0\ell^3 + 4M_0\ell^2 + 8\alpha K'\alpha^3\left(kw_1\ell + P_0\right)\right\}^2}{4b\ell^3\left(\ell^3 + 24\alpha K'\alpha^3\right)\left(kw_1\ell^2 + 4P_0\ell + 6M_0\right)} \tag{2.3.9}$$

$$q_1 = \frac{N_0 + w_1\ell}{A} + \frac{3aK'\left(kw_1\ell^2 + 4P_0\ell + 6M_0\right)}{b\left(\ell^3 + 24\alpha K'\alpha^3\right)} \tag{2.3.10}$$

When determining the horizontal resistance force of deep foundations, the values of p_1 and q_1 obtained by equations **(2.3.9)** and **(2.3.10)** must satisfy equations **(2.3.11)** and **(2.3.12)**, respectively.

$$p_1 \leq \gamma_a p_{p_k} \tag{2.3.11}$$

$$q_1 \leq q_{u_d} \tag{2.3.12}$$

where

l : penetration depth (m)
$2b$: maximum width perpendicular to horizontal force (m)
$2a$: maximum length (m)
A : bottom area (m2)
P_0 : horizontal force acting on structure above ground surface (kN)
M_0 : moment due to P_0 at ground surface (kN・m)
N_0 : vertical force acting at ground level (kN)
k : horizontal seismic coefficient
K' : $K'=K_2/K_1$
K_1 : rate of increase in coefficient of vertical subgrade reaction (kN/m4)
K_2 : rate of increase in coefficient of horizontal subgrade reaction (kN/m4), see equation **(2.3.7)**
w_1 : self weight of deep foundation per unit of depth (kN/m)
α : constant determined by bottom shape (α= 1.0 for rectangular shape and α= 0.588 for round shape)
p_{pk} : characteristic value of passive earth pressure at depth h (m) (kN/m2), see **Part II, Chapter 5, 1 Earth Pressure**.

Provided, however that h is given by equation **(2.3.19)**.

$$h = \frac{kw_1\ell^4 + 3P_0\ell^3 + 4M_0\ell^2 + 8\alpha K'\alpha^3\left(kw_1\ell + P_0\right)}{2\ell\left(kw_1\ell^2 + 4P_0\ell + 6M_0\right)} \tag{2.3.13}$$

q_{ud} : design value of vertical bearing capacity at bottom level (kN/m2), see equation **(2.3.2)**
γ_a : partial factor for horizontal resistance force

④ When Vertical Resultant Force at the Bottom is outside the Core [12)]

When the vertical resultant force acting at the base of foundation is not inside the core, a triangular distribution of vertical subgrade reaction is assumed as shown in **Fig. 2.3.2** [12)]. When the vertical subgrade reaction is expressed as q_d (kN/m2), the maximum subgrade reaction p_1(kN/m2) in the front ground is obtained from equation **(2.3.14)**.

$$p_1 = \frac{3\left(kW\ell + 4M_0 - 4N_0e - 4We + 3P_0\ell\right)^2}{4b\ell^2\left(kW\ell + 6M_0 - 6N_0e - 6We + 4P_0\ell\right)} \tag{2.3.14}$$

The value of p_1 calculated by equation (**2.3.14**) should satisfy equation (**2.3.11**). In this case, h is obtained by equation (**2.3.12**).

$$h = \frac{\ell\left(kW\ell + 4M_0 - 4N_0e - 4We + 3P_0\ell\right)}{2\left(kW\ell + 6M_0 - 6N_0e - 6We + 4P_0\ell\right)} \tag{2.3.15}$$

where

h : depth at which horizontal subgrade reaction becomes maximum (m), see **Fig. 2.3.2**
W : self weight of foundation (kN)
e : eccentric distance (m)

The distance e is defined as shown in **Fig. 2.3.2**. When the foundation bottom is rectangular with the length of $2a$ (m) and the width of $2b$ (m), the value of e is calculated by equation (**2.3.16**).

$$e = \alpha - \frac{W + N_0}{4bq_a} \tag{2.3.16}$$

In the case of a round foundation bottom, the calculation may be made by replacing it with a rectangular foundation bottom having length $2a$ and width $2b$ defined by equation (**2.3.17**).

$$\left.\begin{aligned} 2a &= \frac{\pi}{3}D \\ 2b &= \frac{3}{4}D \end{aligned}\right\} \tag{2.3.17}$$

where

D : diameter of circle (m)

In this way, the horizontal bearing capacity can be estimated at a safer side by approximately 10%. However, this substitution should be applied on the basis of the appropriate judgement, by referring to reference [12].

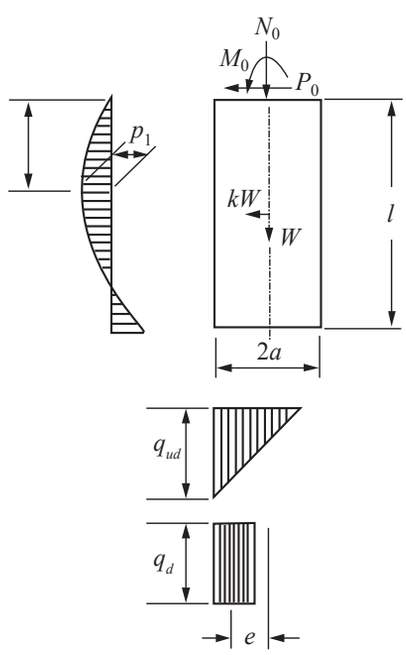


Fig. 2.3.2 When Resultant Force is Not Inside the Core

2.4 Pile Foundations

2.4.1 General

(1) Definition of Pile Foundation
Pile foundation means a foundation which supports superstructures by means of a single pile or multiple piles, or a foundation which transfers actions on the facilities or the foundation to the ground by means of single piles or multiple piles, even when no facilities exist above the piles.

(2) Definition of Pile
Pile means a columnar structural element which is provided underground in order to transfer actions on the facilities or the foundation to the ground.

2.4.2 Fundamentals of Performance Verification of Piles

(1) The loads received by piles as a result of actions are complex. However, in general, the components of the loads acting on a pile consist of the axial load component and the lateral load component, and verification can be performed based on the pile resistance performance with respect to the loads in these respective directions.

(2) Depending on the types of superstructures supported by the pile foundation and the types of loads acting on the piles, there are cases in which is necessary to perform analysis by the component coupling method, treating the superstructure and pile foundation as components.

2.4.3 Static Maximum Axial Pushing Resistance of Pile Foundations

[1] General

(1) The design value of the axial bearing resistance of pile foundations comprising vertical piles is generally determined based on the maximum axial bearing resistance due to the resistance of the ground to vertical single piles as a standard value in taking consideration of the following items.

① Safety margin for displacement in the axial direction based on ground failure and deformation of the ground
② Compressive stress of pile material
③ Joints
④ Slenderness ratio of piles
⑤ Action as pile group
⑥ Negative skin friction
⑦ Settlement of pile head

(2) The above (1) describes the general principle for determining the axial bearing resistance of pile foundations comprising vertical piles. In order to determine the axial bearing resistance of a pile foundation, first, the static maximum axial bearing resistance due to the resistance of the ground is determined, and a safety margin is considered on this. Then, the above items (a) to (g) are examined, and the maximum axial bearing resistance is reduced as necessary. The result obtained in this manner is the design value of the axial bearing resistance of the piles which should be used in performance verification of the pile foundation.

(3) When considering the axial bearing resistance characteristics of a single pile based on the resistance of the ground, the axial compressive load P_0 acting on the pile head of the single pile is supported by the end resistance R_p and the shaft resistance R_f of the pile, and can be expressed as in equation **(2.4.1)**.

$$P_0 = R_p + R_f = R_t \tag{2.4.1}$$

where

R_t : axial bearing resistance of single pile

(4) Characteristic Value of Axial Bearing Resistance of Single Pile Due to Resistance of Ground

① Typical characteristic values for the axial bearing resistance of single piles include the following.

(a) Second limit resistance: Resistance equivalent to the load at the maximum bearing resistance in a static loading test. Provided, however, that the displacement of the end of the pile shall be within a range of no more than 10% of the end diameter. The static maximum axial bearing resistance given by appropriate calculations shall be equivalent to this.

(b) First limit resistance: Resistance equivalent to the load at a clear break point appearing in the log$P$–log$S$ curve in the static compressive loading test. P represents load at the head and S means settlement value at the head of a pile.

(c) Vertical spring constant of pile head: Slope of the secant of the pile head load displacement curve in the static compressive loading test.

(5) Setting of Design Value of Axial Bearing Resistance of a Single Pile Based on Resistance of Ground

① A safety margin shall be provided in the second limit resistance. The following equations are used in this safety margin. Provided, however, that γ in the equation is the partial factor for its subscript, and the subscripts *k* and *d* indicate the characteristic value and the design value, respectively.

$$R_{p_d} = \gamma_{Rp} R_{p_k} \tag{2.4.2}$$
$$R_{f_d} = \gamma_{Rf} R_{f_k} \tag{2.4.3}$$

where

R_p : bearing resistance of the end of pile
R_f : shaft resistance of pile during compressive loading

In cases where only the bearing resistance of the pile head can be obtained in the loading test, and a safety margin can be determined from the bearing resistance of the pile head, the following equation can be used.

$$R_{t_d} = \gamma_{R_t} R_{t_k} \tag{2.4.4}$$

where

R_t : axial bearing resistance of single pile

The standard values of the partial factors γ_{Ri} for the pile end resistance, the shaft resistance, and the axial bearing resistance of piles shall be as shown in **Table 2.4.1–Table 2.4.3**. Provided, that in cases where partial factors are determined separately by code calibrations, etc., in the design system. The subscript *i* represents *p*, *f*, or *t*.

Table 2.4.1 Standard Values of Partial Factors for Shaft Resistance

Design situation	γ_{R_i}: Partial factor
Variable situation for load acting due to ship berthing	0.40
Variable situation for load acting due to ship traction	0.40
Variable situation for Level 1 earthquake ground motion	0.66
Variable situation for load during crane operation	0.40
Variable situation for load acting due to waves	0.66

Table 2.4.2 Standard Values of Partial factors for Pile End Resistance

Design situation	γ_{R_i}: Partial factor
Variable situation for load acting due to ship berthing	0.40
Variable situation for load acting due to ship traction	0.40
Variable situation for Level 1 earthquake ground motion	0.66 (0.50)
Variable situation for load during crane operation	0.40
Variable situation for load acting due to waves	0.66 (0.50)

In case the end of the pile remains in an incomplete bearing stratum which appears to be unsafe, the figures in parentheses shall be used.

Table 2.4.3 Standard Values of Partial Factors for Total Resistance

Design situation	γ_{R_i}: Partial factor	
	End Bearing pile*	Friction pile*
Variable situation for load acting due to ship berthing	0.40	0.40
Variable situation for load acting due to ship traction	0.40	0.40
Variable situation for Level 1 earthquake ground motion	0.66	0.50
Variable situation for load during crane operation	0.40	0.40
Variable situation for load acting due to waves	0.66	0.50

*) End bearing piles and friction piles shall be as classification provided in (10).

(6) Based on information for the performance verifications of normal port facilities , the use of the partial factors listed above may give conservative results.

(7) Because the axial bearing resistance of piles is strongly affected by the construction method, it is necessary to carry out construction in advance with test piles and collect information for the verification by various types of examination. Depending on the results obtained with the test piles, it may be necessary to change the dimensions of the piles or the construction method.

(8) Among the axial resistance factors of a certain pile, when the end resistance of the pile R_p is governing, the pile is called the end bearing pile, and when the shaft resistance R_f is governing, it is called the friction pile. According to this definition, a pile becomes a bearing pile or a friction pile depending on load conditions such as the magnitude of the load, loading velocity, loading duration, etc. Therefore, the distinction between end bearing piles and friction piles cannot be considered absolute. Although the following definitions lack strictness, here, a pile which passes through soft ground and whose end reaches bedrock or some other bearing stratum is called the end bearing pile, and a pile whose end stops in a comparatively soft layer, and not a hard layer that could particularly be considered a bearing stratum, is called the friction pile.

(9) In general, when a pile penetrates to a so-called bearing stratum such as bedrock, or dense sandy ground, axial resistance is larger and settlement is smaller than when a pile only penetrates to an intermediate layer. When a pile penetrates to a so-called bearing stratum, the pile itself rarely settles, even when the soft layers surrounding the pile undergo consolidation settlement. Therefore, negative skin friction acts on the pile, applying a downward load, and the amount of settlement differs in the head of the pile and the surrounding ground. As these phenomena cause a variety of problems, caution should be necessary. Although these defects are slight in piles which only penetrate to intermediate layers, settlement due to consolidation of the ground under the pile continues, and as a result, there is a danger of uneven settlement.

(10) The partial factor for the serviceability limit is applied to ultimate failure phenomena of the ground. When the designer desires to avoid yielding of the ground, the use of the first limit resistance is conceivable. The Partial factor in this case can be set at a value on the order of 0.5.

(11) In case permanent deformation of the ground is expected to remain after an earthquake, separate examination is necessary. Furthermore, because there are cases in which the shear strength of the soil is remarkably reduced by the action of ground motion, caution is necessary. For example, when sensitive cohesive soil is affected by violent motion, loss of strength is conceivable, and from past examples of earthquake damage, it has been pointed out that liquefaction may occur in loose sandy layers as a result of the action of ground motion, causing a large decrease in the resistance of piles. Accordingly, with friction piles, which are easily affected by phenomena of this type, due caution is necessary in setting the partial factors.

(12) Pile group means a group of piles in which the piles are mutually affected by pile axial resistance and displacement.

[2] Static Maximum Axial Resistance of Single Piles due to Resistance of Ground

(1) The static maximum axial resistance of single piles can be obtained by vertical loading tests or calculation by static bearing capacity formulas after an appropriate soil investigation.

(2) As methods of estimating the static maximum axial resistance of single piles from the resistance of the ground, the following are conceivable:

① Estimation by loading tests
② Estimation by static bearing capacity formulas
③ Estimation from the existing data

(3) It is preferable to estimate the static maximum axial resistance of single piles from the resistance of the ground by conducting axial loading tests. Determining the characteristic value of the static maximum axial resistance by this method and then conducting the performance verification is the most rational method. In this case, the soil conditions may differ at the location where the loading test is conducted and at the site where the actual piles are to be driven. Therefore, it is necessary to evaluate the results of loading tests with caution with regard to their relationship to soil conditions, based on a sound understanding of the soil conditions at the location where the loading test is conducted.

(4) It may be difficult to conduct loading tests prior to the performance verification due to circumstances related to the construction period or cost. In such cases, estimation of the static maximum axial resistance depending on the failure of the ground by static bearing capacity formulas taking account of the results of soil investigation is permissible. Even when estimating the static maximum axial resistance by methods other than the above-mentioned (2)(a), and conducting the performance verification by setting the axial resistance of piles based

thereon, the appropriateness of the pile axial resistance used in the performance verification should be confirmed by conducting loading tests at the initial stage of construction.

[3] Estimation of Static Maximum Axial Resistance from Loading Tests

(1) When the second limit resistance can be confirmed from the load-settlement curve, the characteristic value for static maximum axial resistance can be set based on that value. When it is not possible to confirm the second limit resistance from the load-settlement curve, it is permissible to confirm the first limit resistance and use that value as the characteristic value, or to estimate the second limit resistance from the first limit resistance. It is also permissible to obtain the vertical spring coefficient of the pile head based on the load-settlement curve at the pile head.

(2) Effect of Negative Skin Friction
When a pile passes through soft ground, there is a danger that the direction of skin friction may be reversed due to consolidation of the soft ground, this phenomenon is called negative skin friction. In such cases, it is necessary to conduct tests to appropriately evaluate the pile end resistance.

(3) Load-total Settlement Curve Obtained by Static Loading Test
A load-total settlement curve obtained by a static loading test is shown schematically in **Fig. 2.4.1**. The curve, which is initially gentle, shows pronounced break points, and the settlement of the pile head becomes remarkable, even though there is no increase in the load.

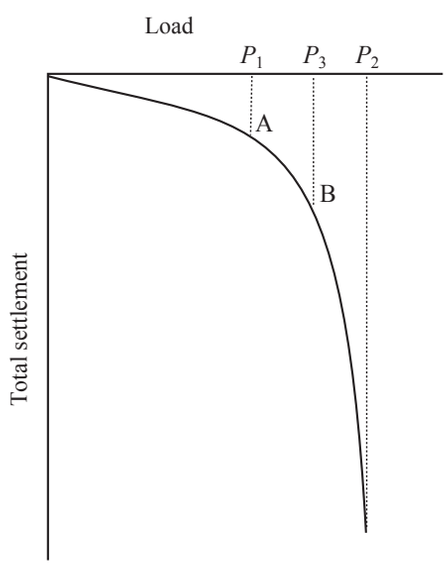


Fig. 2.4.1 Yield Load and Ultimate Load

(4) Case in which the Second Limit Resistance is not Obtained Directly by Loading Test
Although there is no problem if the second limit resistance can be obtained by a loading test, in many cases, it is not possible to apply a sufficiently large load to confirm the second limit resistance due to constraints related to the test equipment. In such cases, the second limit resistance can be assumed by multiplying the first limit resistance obtained by a loading test by 1.2. This judgment is based on the results of research by Yamakata and Nagai [14] on steel pipe piles and statistical studies by Kitajima et al.[15] When the first limit resistance also cannot be obtained in loading tests, the second limit resistance should be assumed to be 1.2 times the maximum load in the test, or a method of setting the design value of the pile axial resistance which does not depend on the second limit resistance should be examined. In either case, a condition which assumes that the pile axial resistance estimated in this way will be larger than the pile axial resistance that can actually be expected is required.

(5) Alternative Loading Test Methods for Static Loading Test

① The rapid load test [17] is a loading test which shall be performed in less than 1 second. Test equipment capable of applying a large instantaneous load is necessary; however, because various innovations have eliminated the need for reaction piles, the test can be performed more easily than the static loading test.

② The end loading test is a method in which a jack is installed near the bottom end of the pile, and the pile body is pushed up while pushing the bottom end of the pile. This method enables separate measurement of the pile end resistance and pile shaft resistance.

③ The dynamic loading test [18] is a type of loading test which employs an ordinary pile driver. As a feature of this test method, changes over time in the elastic strain and displacement of the pile head are measured. In this test, there are limits to the resistance which can be obtained, depending on the magnitude of the pile-driving energy. Therefore, when the axial resistance which is to be estimated is large, as in long or large-diameter piles, in many cases it is not applied as a method for direct estimation of the second limit resistance. It can be used to estimate the relationship between static resistance and driving stop control during construction.

[4] Estimation of Static Maximum Axial Resistance by Static Resistance Formulas

(1) When estimating static maximum axial resistance using static resistance formulas, attention must be paid to the soil conditions, pile conditions, construction methods, and limits of applicability of the static resistance formulas.

(2) The static maximum axial resistance obtained by static resistance formulas may be considered to be equivalent to the second limit resistance.

(3) When using static resistance formulas, it is necessary to consider differences in construction methods.

① Piles driven by hammer driving method [a)]

(a) When employing static resistance formulas using the results of standard penetration test results and undrained shear strength of ground

i) End resistance of a pile

a) Equation **(2.4.5)** can be used in estimating end resistance of a pile when the bearing stratum is sandy ground.

$$R_{p_k} = 300NA_p \tag{2.4.5}$$

where

R_{Pk} : characteristic value of end resistance of a pile by static resistance formula (kN)
A_p : effective area of end of pile (m^2). In determining the effective area of an open-ended pile, it is necessary to consider the degree of closure of the end of the pile.
N : N value of ground around pile end

Provided, however, N is calculated by equation **(2.4.6).**

$$N = \frac{N_1 + \overline{N_2}}{2} \tag{2.4.6}$$

where

N_1 : N-value at end of pile ($N_1 \leq 50$)
$\overline{N_2}$: mean N-value in range above the end of pile to distance of $4B$ ($\overline{N_2} \leq 50$)
B : diameter or width of pile (m)

In equation **(2.4.5)**, the coefficient of the equation proposed by Meyerhof based on the correlation between the static penetration test and the standard penetration test in sandy ground was modified to conform to real conditions.

In estimating the ultimate pile end resistance of piles supported by ground with an N-value of 50 or more, caution is necessary, as N-values itself is not reliable when it is measured larger than 50, and furthermore, the applicability of equation **(2.4.5)** in its current form to hard ground of this kind has not been adequately confirmed.

b) In estimation of the point resistance of piles when the point of the pile penetrates clayey ground, equation **(2.4.7)** can be used.

$$R_{p_k} = 6c_pA_p \tag{2.4.7}$$

where

c_p : undrained shear strength at position of the end of a pile (kN/m^2)

The bearing capacity coefficient of the end resistance of a pile in cohesive soil ground shown in equation **(2.4.7)** was obtained by the same method as the bearing capacity of foundations on cohesive soil ground in **2.2 Shallow Spread Foundations**. Because the cross-sectional shape of ordinary piles has point symmetry, B/L = 1.0, and $Bk/c_p < 0.1$. Based on these facts, the bearing capacity coefficient N_c of foundations is obtained from **Fig. 2.2.2,** see **2.2.3 Bearing Capacity of Foundations on Cohesive Soil Ground**. Therefore, the bearing capacity coefficient of the end of the pile is 6. Accordingly, the end resistance R_p of the pile can be shown as $6c_pA_p$.

As the undrained shear strength used here, the undrained shear strength c_u obtained in the unconfined compression test was commonly used up to the present.

ii) Pile shaft resistance

Pile shaft resistance may be obtained as the sum of the products obtained by multiplying the average strength of skin friction per unit of area in each layer with which the pile is in contact. Namely, equation **(2.4.8)** can be used.

$$R_{f_k} = \sum \bar{r}_{f_{k_i}} A_{s_i} \tag{2.4.8}$$

where

R_{fk} : characteristic value of pile shaft resistance (kN)

$\bar{r}_{f_{ki}}$: average strength of skin friction per unit of area in i-th layer (kN/m^2)

A_{si} : circumferential area of pile in contact with ground in i-th layer (= length of outer circumference Us x thickness of layer l) (m^2)

For sandy ground, equation **(2.4.9)** can be used.

$$\bar{r}_{f_{k_i}} = 2\bar{N} \tag{2.4.9}$$

where

$\bar{N}$: mean N-value of i-th layer

For cohesive soil ground, equation **(2.4.10)** can be used.

$$\bar{r}_{f_{k_i}} = \bar{c}_a \tag{2.4.10}$$

where

$\bar{c}_a$: mean adhesion of pile in i-th layer (kN/m^2)

Here, the value of the adhesion of the pile may be obtained as follows.

in case $c \leq 100$kN/m^2; $c_a = c$

in case $c > 100$kN/m^2; $c_a = 100$kN/m^2 (2.4.11)

However, because theoretical problems [24)] arise in obtaining the adhesion of piles from the undrained shear strength c of the ground, the value of adhesion should be examined, paying due attention to the characteristics of the ground and conditions of the piles.

(b) Method of estimating the end resistance of piles which remain in sandy ground from bearing capacity theory

i) Expansion of bearing capacity theory of shallow spread foundations

If the shear resistance angle of the bearing stratum is known, the end resistance of the pile can be estimated as an expansion of the bearing capacity theory for shallow spread foundations. Here, the following method is introduced as an example. The end resistance of the pile is obtained using equation **(2.4.12)**.

$$R_{p_k} = N_q \sigma'_{v0} A_p \tag{2.4.12}$$

where

N_q : bearing capacity coefficient proposed by Berezantzev, see **Fig. 2.4.2**

σ'_{v0} : effective overburden pressure at the end of pile (kN/m^2)

When N_q is to be obtained from **Fig. 2.4.2**, it is necessary to obtain the shear resistance angle. When obtaining the shear resistance angle, equation **(2.3.21)** in **Part II, Chapter 3, 2.3.4 Interpretation Methods for *N* Values** can be used. When the shear resistance angle is to be obtained by a triaxial compression test, it is necessary to consider the fact that the shear resistance angle is reduced as a result of confining pressure.

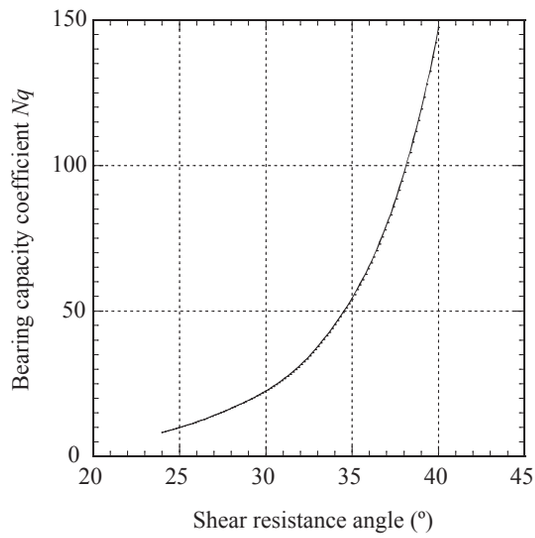


Fig. 2.4.2 Bearing Capacity Coefficient proposed by Berezantzev

ii) Void expantion theory

The failure mode when the area around the end of the pile fails due to compressive force is considered to be one in which a plastic region forms at the outside of a spherical rigid region around the end of the pile and is in balance with an elastic region at its outer side.[25] This theory is called the void expantion theory.

End resistance of a pile according to the void expantion theory can be shown by the following equations.[26), 27)]

$$
\begin{aligned}
q_p &= \frac{3(1+\sin\phi'_{cv})}{(1-\sin\phi'_{cv})(3-\sin\phi'_{cv})}\left[I_{rr}\right]^{(4\sin\phi'_{cv}/(3(1+\sin\phi'_{cv})))}\left(\frac{3-2\sin\phi'_{cv}}{3}\right)\sigma'_{v0} \\
I_{rr} &= \frac{I_r}{1+I_r\Delta_{av}} \\
I_r &= \frac{3G}{(3-\sin\phi'_{cv})\sigma'_{v0}\tan\phi'_{cv}}
\end{aligned}
\tag{2.4.13}
$$

where

q_p : end resistance of a pile per unit area (kN/m^2)

I_{rr} : corrected rigidity index

I_r : rigidity index

ϕ_{cv}' : shear resistance angle in limit condition; assumes $\phi_{cv}' = 30+\Delta\phi_1+\Delta\phi_2$. the values of $\Delta\phi_1$ and $\Delta\phi_2$ shall be as shown in **Table 2.4.4**.

Δ_{av} : coefficient defining compressibility of ground. $\Delta av = 50\ (I_r)^{-1.8}$

G : shear rigidity. May be obtained as $G = 7000N^{0.72}$ (kN/m^2). N is the N-value around the end of the pile.

Table 2.4.4 $\Delta\phi_1$; $\Delta\phi_2$ of Sand and Gravel

(Depends on particle shape)	$\Delta\phi_1$(°)	(Uniformity coefficient)	$\Delta\phi_2$(°)
Round	0	Uniform (U_c<2)	0
Somewhat angular	2	Moderate particle size distribution (2<U_c<6)	2
Angular	4	Good particle size distribution (6<U_c)	4

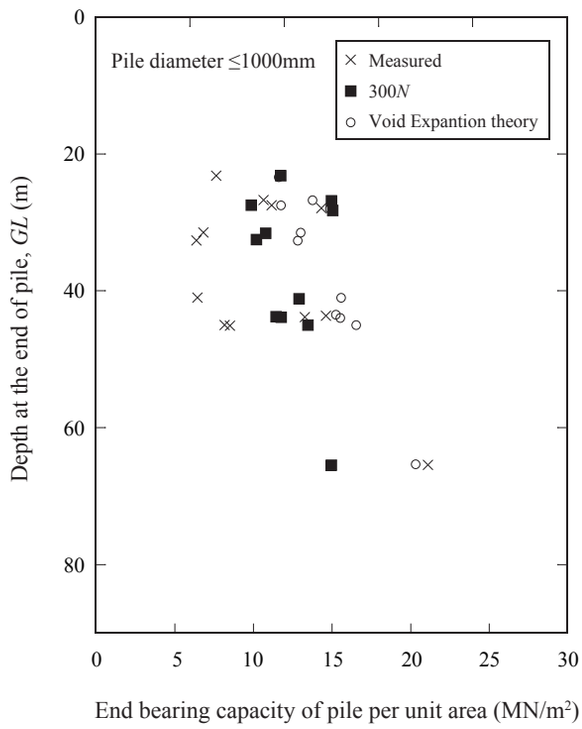


Fig. 2.4.3 Comparison of Measured End Bearing Capacity of Pile and Results of Calculation by Void Expantion Theory

Fig. 2.4.3 shows the results of a comparison of the measured end bearing capacity of pile and the results of an estimation of end bearing capacity by the expanded void theory assuming $\phi_{cv}' = 34$.

② The vibratory pile driving method, vibro-hammer method, is increasingly being used for driving piles because of the capacity increase of pile-driving machinery in recent years. As the principles of this method differ from those of pile driving by hammer, the bearing capacity should be carefully estimated. When using this method, the ground should be compacted by the method of hammer pile driving instead of vibratory pile driving in the course of final driving, or vertical loading tests should be conducted to confirm the characteristics of bearing capacity of the ground in question.

③ In recent years, the use of pile installation method by inner excavation instead of pile driving by hammer has been increasing in port and harbor construction works. In such cases, the characteristics of the bearing capacity of piles in question should be confirmed by vertical loading tests.

(4) Effective Areas of Pile End

① Even if there is no shoe on the pile end, the end bearing area of steel piles can be considered closed, as shown by the shaded areas in **Fig. 2.4.4**. In this case, the outer edge of the closed area is taken as the perimeter. This is based on the following principle. Soil enters the interior of steel pipes or the space between the flanges of H-shaped steel during the pile driving until the internal friction between the soil and the surface of steel pile becomes equal to the end resistance of pile. This balance prevents soil from entering to the piles and has the same effect as the case when the open end section is closed. But complete closure cannot be expected in the case of large-diameter piles. In such cases the plugging ratio should be examined.

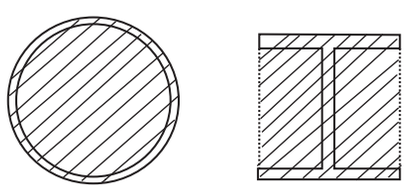
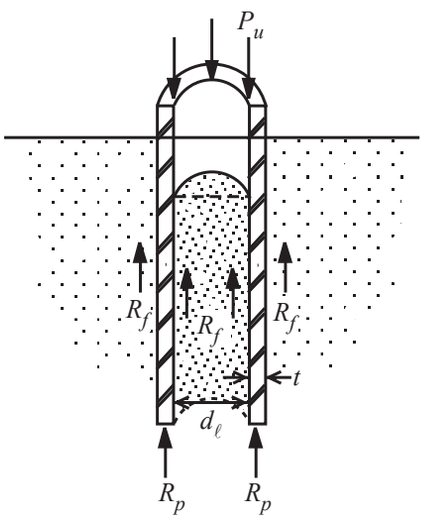


Fig. 2.4.4 End Bearing Area of Steel Piles

② Plugging ratio

The mechanism of the end resistance of open ended piles is composed of the sum of the end resistance of the substantial part of the end of the pile and the skin friction of the inner surface of the pile as shown in **Fig. 2.4.5**.

The resistance from the inner surface of the pile is considered to be determined from the direct stress action on the circumference and the inner circular area of the pile. Because the pile cross-sectional area is proportional to the square of its diameter and its circumference is proportional to its diameter, as the diameter of a pile becomes larger, the concept that the total cross-sectional area of the pile is effective for resistance loses validity. In piles of this type, among the resistances which are conceivable due to closure of the pile end, only some fraction can be expected to function as the end resistance. That fraction is called the plugging effect ratio. The size of the plugging effect ratio is affected by the diameter or width of the pile, the penetration depth of the pile, the properties of the ground, the construction method, and cannot be determined simply by the diameter or width of the pile alone.



P_u : actions
R_f : outer skin friction of pile
R_p : resistance attributable to wall thickness of pile end in open-ended pile
R_f : resistance due to plugged soil
d_ℓ : ile diameter

Fig. 2.4.5 Schematic Diagram of Plugging Effect Ratio

③ Different from plugging effect ratio, the plugging ratio refers to the ratio of the end resistance that can actually be expected to the end resistance obtained by static resistance formulas. From past data, the plugging ratio can be considered to be 100% when the diameter of steel pipe piles is less than 60 cm or H-shaped steel piles which short side width is less than 40 cm. Numerous theoretical calculation methods [30), 31), 32), 33), 34), 35)] and results of laboratory experiments [36), 37)] have been presented as methods of estimating the plugging effect ratio which consider the various factors mentioned above for piles with larger diameters or widths. There are also examples of study by actually conducting pile loading tests. However, in addition to the fact that the plugging effect ratio varies greatly depending on the properties of the ground, the construction method, and other factors, the state of plugging of actual piles differs depending on the penetration depth, including the stress in the ground, making it difficult to obtain the ratio by theoretical calculation.

④ The Japan Association of Steel Pipe Piles collected examples of measurements of the plugging ratio.[38)] **Fig. 2.4.6** shows data based thereon together with additional new data. The new data added here are for piles with diameters of 1100mm to 2000mm. According to these data, the plugging ratio for the case where equation **(2.4.5)** is considered to express the end resistance for complete plugging is in the range of 30-140%. In any case, it appears that there is virtually no correlation between the embedded length ratio in the bearing stratum and the plugging ratio. Provided, however, that there is clearly a difference in the plugging ratio in steel pipe piles with diameters of less than 1000mm and those with diameters greater than 1000mm. Caution is particularly necessary when using large diameter steel pipe piles with diameters larger than 1000mm. **Fig. 2.4.7** shows the results when the *x*-axis indicates the pile diameter. In spite of some dispersion in the data, the pile diameter has a large effect on the plugging ratio, as can be understood by comparison with **Fig. 2.4.6**.

The plugging ratio is affected by construction methods and soil condition, therefore it is necessary to grasp the plugging ratio in actual construction works and by carrying out the loading tests.

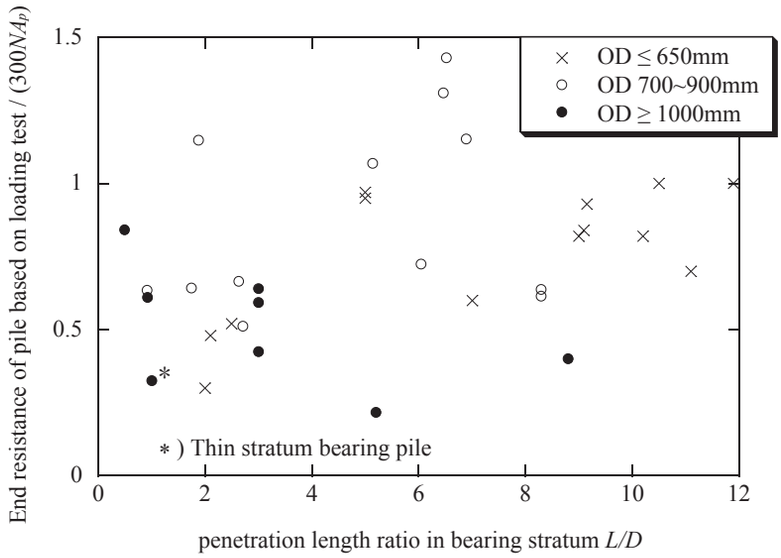


Fig. 2.4.6 Plugging Effect of Open Ended Piles (effect of embedded length ratio in bearing stratum)

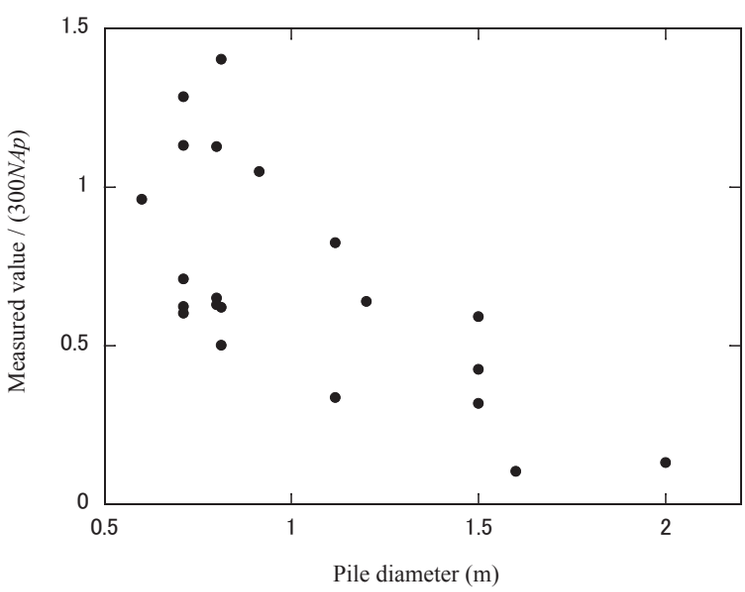


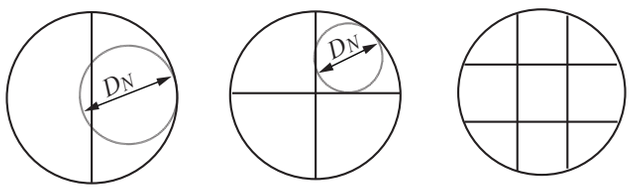
Fig. 2.4.7 Plugging Effect of Open Ended Piles (effect of pile diameter)

(5) Bearing Capacity of Soft Rock

When piles are supported on soft rock or hard clay, the bearing capacity may be calculated by equation (**2.4.5**). If unconfined compressive strength q_u (kN/m2) has been measured by undisturbed soil samples, equation (**2.4.14**) may alternatively be used.

$$R_{p_k} = 5q_u A_p \tag{2.4.14}$$

Further, the value of q_u should be reduced to 1/2 or 1/3 of the measurement values depending on the conditions of cracking in the ground. In any event, however, the value of q_u should not exceed 2×10^4kN/m^2.



N=2 N=4 N=9

N ; Division Number

[5] Examination of Compressive Stress of Pile Material

When determining the axial resistance of piles, it is necessary to consider safety with respect to failure of the pile material.

[6] Decrease of Bearing Capacity due to Joints

(1) If it is necessary to splice piles, the splicing work shall be executed under appropriate supervision and reliability of joints of spliced pile shall be confirmed by appropriate inspection.

(2) If joints are sufficiently reliable, it may not be necessary to decrease the axial bearing capacity due to joints.

(3) When spliced piles are used, the joints sometimes become the weak points in the pile. Therefore, it is necessary to adequately examine the structural reliability of the joints. If the structural reliability of the joints is inadequate, it is necessary to reduce the axial resistance, in consideration of the effect of the joint on the bearing capacity of the pile foundation as a whole.

(4) In-site circular welding by semi-automatic methods is generally employed for the splicing of steel pipe piles used in the field of port and harbor construction works. When such highly reliable jointing methods are applied under appropriate supervision and the reliability of the joints has been confirmed by inspection, it is not necessary to decrease the axial bearing capacity.

(5) For other matters related to the structures of joints, **2.4.6[4] Joints of piles** of piles can be used as reference.

[7] Decrease of Bearing Capacity due to Slenderness Ratio

(1) For piles with a very large ratio of length to diameter, the axial bearing capacity of piles needs to be decreased in consideration of the accuracy of installation, unless the safety of bearing capacity is confirmed by loading tests.

(2) This provision takes account of the fact that the inclination of piles during installation reduces their bearing capacity. If loading tests are conducted on foundation piles, the ultimate bearing capacity can be determined, accounting for the decrease of bearing capacity due to installation accuracy. Therefore, in this case the decrease due to the slenderness ratio may not necessarily be taken into account.

(3) When decreasing the bearing capacity due to the slenderness of piles, the following values may be used as references:

① Except for steel pipe piles

$$\alpha = \begin{cases} 0 & \left(\dfrac{\ell}{d} \le 60\right) \\ \dfrac{\ell}{d} - 60 & \left(\dfrac{\ell}{d} > 60\right) \end{cases} \tag{2.4.15}$$

② For steel piles

$$\alpha = \begin{cases} 0 & \left(\dfrac{\ell}{d} \le 120\right) \\ \dfrac{\ell}{2d} - 60 & \left(\dfrac{\ell}{d} > 120\right) \end{cases} \tag{2.4.16}$$

where

α : rate of reduction (%)
ℓ : pile length (m)
d : pile diameter (m)

[8] Bearing Capacity of Pile Groups

(1) When a group of piles are examined as a pile group, the bearing capacity of pile group may be studied as a single and deep foundation formed with the envelope surface surrounding the outermost piles in the group of piles.

(2) Terzaghi and Peck state that a failure of a pile group foundation does not mean the failure of the individual piles but failure as a single block, [45), 46)] based on the principle that the soil and piles inside the hatched area in **Fig. 2.4.8** work as a single unit when the intervals between the piles are small. The axial resistance of a pile group when considered in this manner is expressed by equation **(2.4.17)**.

$$R_{gu_d} = \gamma_q q_{dk} A_g + \gamma_s \overline{s_k} UL \tag{2.4.17}$$

where

R_{gud} : design value of axial resistance of pile group as single block (kN)
q_{dk} : static maximum axial resistance (characteristic value) when bottom of block is assumed to be foundation load plane according to Terzghi's equation (kN/m^2)
γ_q : partial factor for bottom bearing capacity (bearing capacity of foundation on sandy ground and bearing capacity of foundation on cohesive soil ground in **2.2 Shallow Spread Foundations**)
A_g : bottom area of pile group (m^2)
U : perimeter length of pile group (m)
L : penetration length of piles (m)
$\overline{s_k}$: mean shear strength of soil in contact with piles (characteristic value) (kN/m^2)
γ_s : partial factor for skin friction (see **2.4.3[1] General**)

The axial resistance per pile is shown by equation **(2.4.18)**.

$$R_{ad} = (R_{gu_d} - \gamma'_2 A_g L)\frac{1}{n} \tag{2.4.18}$$

where

R_{ad} : design value of axial resistance per pile against failure as a block (kN)
γ'_2 : mean unit weight of whole block including piles and soil (kN/m^3); below groundwater level, the mean unit weight is calculated considering buoyancy, and above ground water level, using the wet unit weight.
n : number of piles in pile group

In the case of cohesive soil, equation **(2.4.18)** is replaced by equation **(2.4.19)**, where c is undrained shear strength and $\gamma'_2 \fallingdotseq \gamma_2$ (γ_2: mean unit weight of soil above the end of the pile).

$$R_{ad} = \frac{1}{n}\gamma_a \left\{ 5.7cA_g \left(1 + 0.3\frac{B}{B_1} \right) + cUL - \gamma_2 A_g L \right\} \tag{2.4.19}$$

where

B : short side width of pile group (block) (m)
B_1 : long side width of pile group (block) (m)
γ_a : partial factor (see **2.2.3 Bearing Capacity of Foundations on Cohesive Soil Ground**)

As the axial resistance of each pile when used as a pile group, it is necessary to use the smaller of the axial resistance of the single piles or the resistance against block failure given by equation **(2.4.18)** or **(2.4.19)**, respectively.

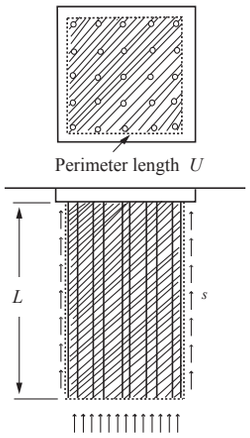


Fig. 2.4.8 Pile Group Foundation

[9] Examination of Negative Skin Friction

(1) If bearing piles penetrate through a soil layer that is susceptible to consolidation, it is necessary to consider negative skin friction when calculating the allowable axial bearing capacity of piles.

(2) When a pile penetrates through a cohesive soft layer to reach a bearing stratum, the friction force from the soft layer acts upwards and bears a part of the load acting on the pile head. When the cohesive soft layer is consolidated, the pile itself is supported by the bearing stratum and hardly settles, the direction of the friction force is reversed, as shown in **Fig. 2.4.9**. The friction force on the pile circumference now ceases to resist the load acting on the pile head, but instead turns into a load downwards and places a large burden on the end of the pile. This friction force acting downwards on the pile circumference is called the negative skin friction or negative friction.

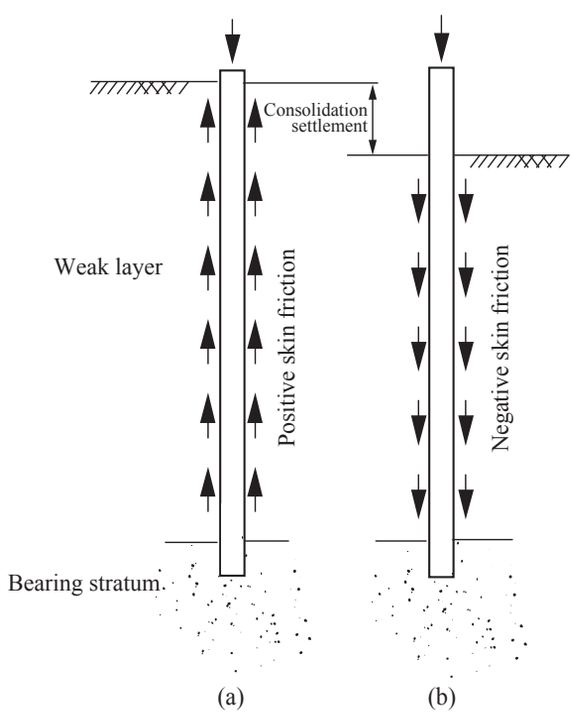


Fig. 2.4.9 Negative Skin Friction

(3) Although the actual value of negative skin friction is not well known yet, the maximum value may be obtained from equation (**2.4.20**).

$$R_{nf,\max_k} = \varphi L_2 \overline{f_s} \tag{2.4.20}$$

where

$R_{nf,\max_k}$: characteristic value of negative skin friction for single pile (maximum value) (kN)

φ : circumference of piles (perimeter of closed area in the case of H-shaped steel piles) (m)
L_2 : length of piles in the consolidating layer (m)
$\overline{f_s}$: mean skin friction intensity in the consolidating layer (kN/m2)

(4) In the above, $\overline{f_s}$ in cohesive soil ground is sometimes taken at $q_u/2$. If a sand layer is located between consolidating layers, or if a sand layer lies on top of consolidating layer, the thickness of the sand layer should be included in L_2. The skin friction in the sand layer is sometimes taken into account for $\overline{f_s}$. The characteristic value of negative skin friction in such cases is shown by equation (**2.4.21**).

$$R_{nf,\max_k} = \left(2\overline{N_{s2}}L_{s2} + \frac{\overline{q_u}L_c}{2}\right)\varphi \tag{2.4.21}$$

where
L_{s2} : thickness of sand layer included in L_2 (m)
L_c : thickness of cohesive soil layer included in L_2 (m)

$L_{s2}+L_c=L_2$

$\overline{N_{s2}}$: mean *SPT-N*-value of the sand layer of thickness L_{s2}
$\overline{q_u}$: mean unconfined compressive strength of cohesive soil layer of thickness L_c (kN/m2)

(5) In pile groups, the characteristic value of negative skin friction may be calculated by obtaining the negative skin friction assuming all of the piles form a single and deep foundation, and dividing the result by the number of piles to obtain the negative skin friction per pile. (see **Fig. 2.4.10**).

$$R_{nf,\max_k} = \frac{\overline{s}UH + A_g\gamma L_2}{n} \tag{2.4.22}$$

where
$R_{nf,\max k}$: characteristic value of negative skin friction for pile group (kN)
U : perimeter length of group of piles acting as pile group (m)
H : depth from ground level to bottom of consolidation layer (m)
$\overline{s}$: mean shear strength of soil in range of H in **Fig. 2.4.10** (kN/m2)
A_g : bottom area of group of piles acting as pile group (m2)
γ : mean unit weight of soil in range of L_2 in **Fig. 2.4.10** (kN/m3)
n : number of piles in group of piles acting as pile group

Equations **(2.4.20)** to **(2.4.22)** give the conceivable maximum value for negative skin friction. The actual value of negative skin friction is considered to be governed by the amount of consolidation settlement and the speed of consolidation, the creep characteristics of the soft layers and the deformation characteristics of the bearing stratum.

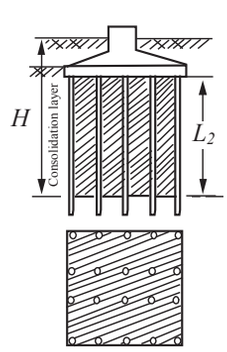


Fig. 2.4.10 Skin Friction of Pile Group

(6) The design value of negative skin friction can be calculated by the following equation, using the characteristic value of negative skin friction.

$$R_{nf,\max_d} = \gamma_{nf} R_{nf,\max_k} \tag{2.4.23}$$

where
γ_{nf} : partial factor for negative skin friction (normally, 1.0 can be used)

(7) Verification

When calculating the axial bearing capacity of piles, many uncertainties exist as to how the influence of negative skin friction should be considered. However, at the present stage, when negative skin friction is adequately considered, one method assumes safety when it is confirmed that the force transmitted to the end of the pile possesses adequate safety against failure of the ground at the pile end and compressive failure of the pile material cross section. That is, when the design value of the axial bearing capacity in the serviceability limit state is R_{ad}, in addition to securing the required safety against ordinary loads, R_{ad} satisfies equations **(2.4.24)** and **(2.4.25).**

$$R_{a_d} \leq \gamma_{R_p} R_{p_k} - R_{nf,\max d} \tag{2.4.24}$$

$$R_{a_d} \leq \gamma_{\sigma_f} \sigma_{f_k} A_e - R_{nf,\max d} \tag{2.4.25}$$

where

R_{ad} : design value of axial bearing capacity (serviceability limit state) (kN)
R_{pk} : characteristic value of end resistance of pile (second limit resistance) (kN)
$R_{nf,\max d}$: design value of maximum negative skin friction (kN)
(smaller of values for single pile or pile group)
σ_{fk} : characteristic value of compressive yield stress of pile (kN/m^2)
A_e : effective cross-sectional area of pile (m^2)
γ_{Rp} : partial factor for end resistance of pile (generally, 0.8 can be used)
$\gamma_{\sigma f}$: partial factor for compressive yield stress of pile (generally, 1.0 can be used)

The characteristic value for end resistance of pile R_{pk} can be calculated using equation **(2.4.5)**. When the pile penetrates into the bearing stratum, the circumference resistance of that section shall be included in the pile end bearing capacity. In this case, the characteristic value of end resistance can be calculated using the following equation (see **Fig. 2.4.11**).

$$R_{p_k} = 300NA_p + 2\overline{N_{s1}}L_{s1}\varphi \tag{2.4.26}$$

where

R_{pk} : characteristic value of end bearing capacity of pile (ultimate value) (kN)
N : N-value of ground at the end of pile
A_p : area of the end of pile (m^2)
$L_{s1} = L_1$: length of pile penetrates into bearing stratum (sandy ground) (m)
N_{s1} : mean N-value for zone L_{s1}
φ : circumference of pile (m)

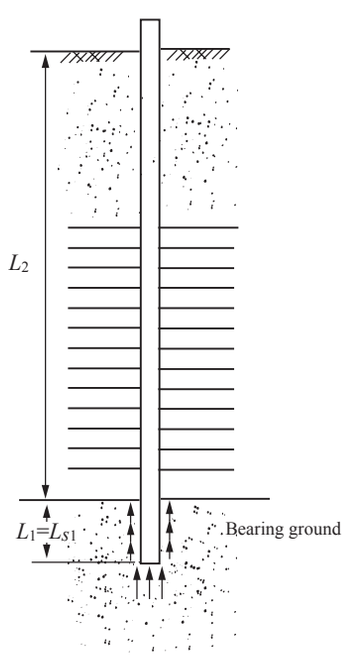


Fig. 2.4.11 End Bearing Capacity

[10] Examination of Pile Settlement

The axial bearing capacity of pile shall be determined in such a way that an estimated settlement of pile head does not exceed the allowable settlement determined for superstructures.

2.4.4 Static Maximum Pulling Resistance of Pile Foundations

[1] General

(1) The design value of the pulling resistance of foundation piles must be determined considering the following items, using the static maximum pulling resistance of a single pile due to failure of the ground as a standard.

① Tensile stress of pile material
② Effect of pile joints
③ Load on pile group due to actions
④ Upward displacement of piles by pulling

(2) The design value of the pulling resistance of piles can be obtained as follows. First, the characteristic value of the static maximum pulling resistance of a single pile is obtained based on failure of the ground and adding safety margin. The design value of the pulling resistance of the pile is then determined considering the stress of the pile material, actions of joints, the pile group and displacement. .

(3) The characteristic values of the pulling resistance of piles are as follows;

① The first limit resistance
The first limit resistance is the load when the shearing stress generated in the pile circumference or the soil surrounding the pile by pulling of the pile affects substantially the entire length of the pile and yielding begins. When a loading test is performed and the log$P$–log$S$ curve is drawn, the clear break point which appears on the curve shall be considered as the first limit resistance.

② The second limit resistance
The second limit resistance is the resistance when the pulling resistance of the pile circumference shows its maximum value. If the maximum resistance is unclear, the second limit resistance shall be the load when the displacement of the end of the pile reaches 10% of the diameter or width of the pile end. The resistance obtained using static bearing capacity formulas may be considered equivalent to this resistance.

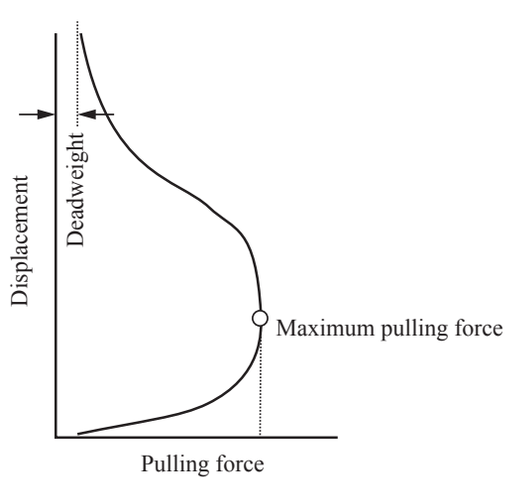


Fig. 2.4.12 Pulling Resistance of Piles

(4) Setting of Design Value of Pulling Resistance of Single Pile

(a) A safety margin shall be taken in the second limit resistance. As the method, the following equation can be used.

$$R_{t_d} = \gamma_R R_{t_k} \tag{2.4.27}$$

where

γ_R : partial factor

The standard value of partial factors can be as shown in **Table 2.4.5**.

Table 2.4.5 Standard Values of Partial Factors for Total Resistance

Design situation	γ_R: Partial factor
Variable situation for load acting due to ship berthing	0.33
Variable situation for load acting due to ship traction	0.33
Variable situation for Level 1 earthquake ground motion	0.40
Variable situation for load during crane operation	0.33
Variable situation for load acting due to waves	0.40

(5) In cases where there appears to be a possibility of liquefaction of sandy layers during an earthquake, it is necessary to determine pulling resistance giving due consideration to this fact.

(6) Because the self weight of the pile can be expected to act reliably as pulling resistance together with the weight of the soil in the pile, a partial factor of 1.0 may be used for this. Accordingly, it is rational to calculate the design value of the pulling resistance due to failure of the ground from the characteristic value of pulling resistance due to failure of the ground as follows. Provided, however, that when the self weight of the pile is comparatively small, this process is normally omitted. When the diameter of the pile is excessively large, it is considered that the soil filled in the pile is not necessarily lifted with the pile, but separates and falls down.

① when maximum pulling resistance is obtained by pulling test

$$R_{at_d} = \gamma_{W_p} W_{p_k} + (R_{ut1_k} - \gamma_{W_p} W_{p_k})\gamma_R \tag{2.4.28}$$

② when maximum pulling resistance is obtained by static bearing capacity formula

$$R_{at_d} = \gamma_{W_p} W_{p_k} + \gamma_R R_{ut2_k} \tag{2.4.29}$$

where

R_{ad} : design value of allowable pulling resistance of pile (kN)
W_{pk} : characteristic value of self weight of pile with buoyancy subtracted (kN)
R_{ut1k} : characteristic value of maximum pulling resistance of pile by pulling test (kN)
R_{ut2k} : characteristic value of maximum pulling resistance of pile by static bearing capacity formula (kN)
γ : Partial factor corresponding to subscript

[2] Static Maximum Pulling Resistance of Single Pile

(1) It is preferable to obtain the maximum pulling resistance of a single pile on the basis of the results of pulling tests.

(2) Unlike axial bearing capacity, there are few comparative data for pulling resistance, and indirect estimations may involve some risk. Thus conduct of pulling tests is preferable to determine the maximum pulling resistance of a single pile. However, in the case of relatively soft cohesive soil, skin friction during driving of a pile is considered to be virtually the same as that during pulling of piles. Therefore, the maximum pulling resistance may be estimated from the results of loading tests (pushing direction) and static bearing capacity equations.

(3) Estimation of the maximum pulling resistance by static bearing capacity formulas may follow the explanation given in **2.4.3[4]. Estimation of Static Maximum Axial Resistance by Static Resistance Formulas**. However, the end bearing capacity shall be ignored. Thus, for piles driven by hammer, the following equations may be used.

① Sandy ground

$$R_{ut_k} = 2\overline{N}A_s \tag{2.4.30}$$

② Cohesive soil ground

$$R_{ut_k} = \overline{c_a}A_s \tag{2.4.31}$$

where

R_{ut_k} : characteristic value of the maximum pulling resistance of pile (kN)
$\overline{N}$: mean N-value for total penetration length of pile
A_s : total circumference area of pile (m^2)
$\overline{c_a}$: mean adhesion for total penetration length of pile (kN/m^2)

(4) In cases where the static maximum pulling resistance of a pile is to be estimated using a static bearing capacity formula, examination is sometimes performed using Terzaghi's equation, which is shown in equation **(2.4.32)**.

In this case, an appropriate value shall be adopted, based on comparison of the values calculated using equation **(2.4.30)** and equation **(2.4.31)** and the value calculated using Terzaghi's equation.

$$R_{ut_k} = R_{f_k} = \varphi L \overline{f_s} \tag{2.4.32}$$

$$\overline{f_{s_k}} = \frac{\sum \left(c_{ai_k} + K_{s_k} q_{i_k} \mu_k \right) \ell_i}{L} \tag{2.4.33}$$

where

R_{utk} : characteristic value of the static maximum pulling resistance of pile (kN)
Rf_k : characteristic value of skin friction of pile (kN)
φ : circumference of pile (m)
L : penetration depth of pile (m)
$\overline{f_{s_k}}$: characteristic value of average strength of skin friction (kN/m2)
$\bar{c}_{aik}$: characteristic value of adhesion between soil and pile in *i*-th layer (kN/m2)
K_{sk} : characteristic value of coefficient of horizontal earth pressure acting on pile
q_{ik} : characteristic value of mean effective overburden pressure in *i*-th layer (kN/m2)
μ_k : characteristic value of coefficient of friction between pile and soil
l_i : thickness of *i*-th layer (m)

For c_a and μ, see **2.4.3[4] Estimation of Static Maximum Axial Resistance by Static Resistance Formulas**.

The value of the coefficient of horizontal earth pressure K_s is considered to be smaller than in the case of pushing. In general, a value between 0.3 and 0.7, which is close to the coefficient of earth pressure at rest, is frequently used.

[3] Items to be Considered when Calculating Design Value of Pulling Resistance of Piles

(1) When determining the pulling resistance of piles, it is necessary to consider the following items.

① The resistance used in verification of the pulling resistance of piles should be no more than the product of the resistance of the pile material and the effective cross-sectional area of the pile.

② In spliced piles, the pulling resistance of the pile below the joint is generally ignored. Provided, however, that when high-quality joints can be used in steel piles, the pulling resistance of the lower pile can be considered within the range of the tensile strength of the joint after confirming the reliability of the joint.

③ In case of a pile group, it is necessary to examine the pulling resistance as a single block surrounded with the envelope surface of the outermost piles in the group of piles that act as a pile group.

④ When determining the pulling resistance of piles, it is necessary to consider the limit value of the upward displacement of pile heads by pulling determined by the superstructure.

(2) Tensile Strength of Pile Materials
The design value of the pulling resistance of piles is limited to the tensile strength of the pile materials. The method of examination can conform to **2.4.3[5] Examination of Compressive Stress of Pile Materials**.

2.4.5 Static Maximum Lateral Resistance of Piles

[1] General

(1) The static maximum lateral resistance of a single pile shall be determined as appropriate on the basis of the behavior of the pile when it is subject to lateral forces.

(2) The characteristic value of the static maximum lateral resistance of a pile must be determined so as to satisfy the following two conditions:

① The pile material shall not fail due to stress generated in the pile body. Especially the pile material shall not fail due to bending stress generated in the pile body.

② The displacement in lateral direction and inclination of the pile head shall not exceed the limit value of the displacement determined by the superstructure.

(3) Penetration Length of Piles
The length of penetrated part of pile that yields effective resistance against external forces is called the effective length. Piles are called long piles when the penetrated length is longer than their effective length. Piles are called

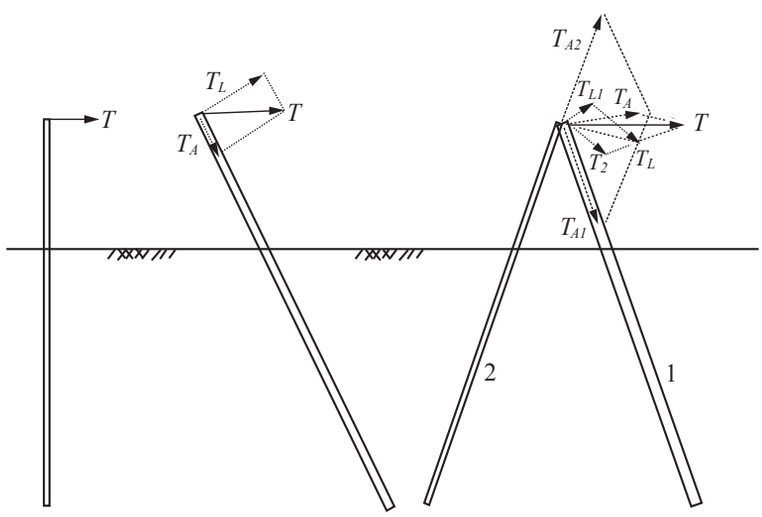
short piles when the penetrated length is shorter than their effective length.

(4) Piles Subject to Lateral Actions
The resistance which a pile performs when subjected to actions in the lateral direction (actions in the horizontal or near-horizontal direction) is called the lateral resistance of the pile, and may be categorized in the three basic forms shown in **Fig. 2.4.13**.[63]

(a) The resistance of the pile is limited to the lateral direction, and resistance in the vertical direction does not appear. This is the simplest form of lateral resistance and is frequently called the lateral resistance of a pile in the narrow sense.

(b) Some part of the resistance of the pile is composed of axial resistance. However, because the shares of the load borne by lateral resistance and axial resistance are determined almost entirely by the inclination angle of the piles, resistance may be divided into lateral resistance and axial resistance and examined separately.

(c) Coupled piles are those in which two or more piles with differing axial directions are combined. The simplest form of coupled piles is shown in **Fig. 2.4.13**. In coupled piles, most of the action is supported by the axial resistance of the respective piles. Therefore, when the free length of the piles is long, the lateral resistance is normally ignored and only the axial resistance is considered in estimating resistance. With coupled piles, it is quite difficult to calculate the pile head displacement. So far, a number of methods have been proposed, [64), 65)] but none can yet be called adequate (see **2.4.5[6] Lateral Bearing Capacity of Coupled Piles**). However, because the displacement of coupled piles is far smaller than that of single piles, displacement rarely becomes a problem.



(a) When one vertical pile is subject to lateral force
(b) When one batter pile is subject to lateral force
(c) When coupled piles are subject to lateral force

Fig. 2.4.13 Piles Subject to Lateral Force

[2] Estimation of Behavior of Piles

(1) The behavior of a single pile which is subject to lateral force can be estimated by either of the following methods or by a combination thereof.

① Methods using loading tests

② Analytical methods

[3] Estimation of Behavior of a Single Pile by Loading Tests

(1) When loading tests are planned to estimate behavior of a single pile subject to lateral force, it is necessary to consider sufficiently the differences in the pile and load conditions between those of actual structures and loading tests.

(2) Loading test results and characteristic value and design value of lateral resistance
When loading tests are conducted under the same conditions as those in actual facilities, the characteristic value

of the static maximum lateral resistance may be obtained from the loading test results by the following method.

The load-pile head displacement curve in lateral loading tests generally shows a curved form from the beginning of the loading. Therefore, with the exception of short piles, a clear yield load or ultimate load normally cannot be obtained. As explained previously in **[1] General**, this is because only gradual small-scale failure occurs in the ground with long penetration lengths, and overall failure of the ground does not occur. Therefore, the load-pile head displacement curve is not used to obtain the yield load or the ultimate load, but to confirm the pile head displacement itself. In other words, the fundamental concept of the performance verification of piles subject to lateral force is determination of the limit value of the displacement of the pile head and design so as not to exceed that limit value.

Furthermore, the bending stress corresponding to the resistance obtained in this manner must also be considered. Hence, it is necessary to ensure that failure associated with the bending stress of the pile material (see **Part II, Chapter 11, 2.2 Characteristic Values of Steel**) does not occur when the expected load acts. To calculate the allowable lateral bearing capacity of short piles, overturning of piles must be considered, in addition to the pile head displacement and bending stress mentioned already. When the overturning load cannot be ascertained, the maximum test load may be used instead of the overturning load.

[4] Estimation of Pile Behavior using Analytical Methods

(1) When estimating behavior of a single pile subject to lateral force by using analytical methods, it is preferable to analyze the pile as a beam is placed on an elastic foundation.

(2) Methods of analytically estimating the behavior of a single pile subject to lateral force as a beam is placed on an elastic foundation include the relatively simple Chang's methods well as the PHRI (Port and Harbor Research Institute, name is changed to PARI) method.[68)]

(3) Basic Equation for Beam on Elastic Foundation
Equation (**2.4.34**) is the basic equation for analytically estimating behavior of a pile as a beam placed on an elastic foundation.

$$EI\frac{d^4y}{dx^4} = -P = -pB \tag{2.4.34}$$

where

EI : flexural rigidity of pile (kN•m^2)
x : depth from ground level (m)
y : displacement of pile at depth x (m)
P : subgrade reaction per unit length of pile at depth x (kN/m)
p : subgrade reaction per unit area of pile at depth x (kN/m^2) $p = \frac{P}{B}$
B : pile width (m)

Analytical methods differ depending on how the subgrade reaction P is considered in equation **(2.4.34)**. If the ground is considered simply as a linear elastic body, P or p is a linear function of displacement of pile y.

$$P = E_s y \tag{2.4.35}$$

or

$$p = \frac{E_s}{B}y = k_{CH}y \tag{2.4.36}$$

where

E_s : modulus of elasticity of ground (kN/m^2)
k_{CH} : coefficient of lateral subgrade reaction (kN/m^3)

There is much discussion concerning the characteristics of the modulus of elasticity E_s, but the simplest concept is that $E_s = k_{CH}B$ = constant, as proposed by Chang.[69)]
Shinohara, Kubo, and Hayashi proposed the PHRI method as an analytical method considering the nonlinear elastic behavior of the ground.[70), 71)] This method can describe the behavior of actual piles more accurately than other methods. The PHRI method uses equation (**2.4.41**) to describe the relationship between the subgrade reaction and the pile displacement.

$$p = kx^m y^{0.5} \tag{2.4.37}$$

where

k : constant of lateral resistance of ground (kN/m$^{3.5}$ or kN/m$^{2.5}$)
m : index 1 or 0

(4) PHRI Method

① Characteristics of the PHRI method

In the PHRI method, the ground is classified into the S type and the C type. The relationship between the subgrade reaction and the pile displacement for each ground is assumed by equation (**2.4.38**) and (**2.4.39**), respectively.

(a) S-type ground

$$p = k_s x y^{0.5} \tag{2.4.38}$$

(b) C-type ground

$$p = k_c y^{0.5} \tag{2.4.39}$$

where

k_s : constant of lateral resistance in S-type ground (kN/m$^{3.5}$)

k_c : constant of lateral resistance in C-type ground (kN/m$^{2.5}$)

The identification of S-type or C-type ground and the estimation of ks and kc are based on the results of loading tests and soil investigation.

In the PHRI method, the nonlinear relationships between p and y are introduced as given by equations (**2.4.38**) and (**2.4.39**) to reflect the actual state of subgrade reaction. Therefore, the solutions under individual conditions would remain unattainable without help of numerical calculation, and the principle of superposition could not be applied. The results of many full-scale tests have confirmed that this method reflects the behavior of piles more accurately than the conventional methods. It is commented here that for piles to behave as long piles, they must be at least as long as 1.5 ℓ_{m1} (ℓ_{m1}: depth of the first zero point of flexural moment in the PHRI method).[64)]

② Constants of lateral resistance of the ground

The two ground types in the PHRI method are defined as follows;

(a) S-type ground

1) Relationship between p-y is expressed as $p = k_s x_y^{0.5}$ refer **(2.4.38)**

2) N-value by the standard penetration test increases in proportion to the depth.

3) Actual examples: sandy ground with uniform density, and normally consolidated cohesive soil ground.

(b) C-type ground

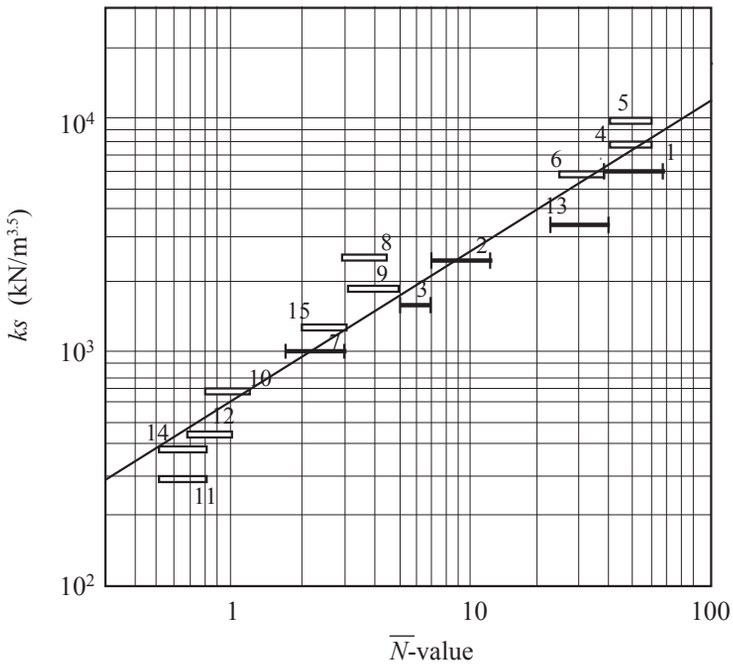
1) *R*elationship between p-y is expressed as $p = k_c y^{0.5}$ refer **(2.4.39)**

2) N-value by the standard penetration test is constant regardless of depth.

3) Actual examples: sandy ground with compacted surface, and heavily-overconsolidated cohesive soil ground.

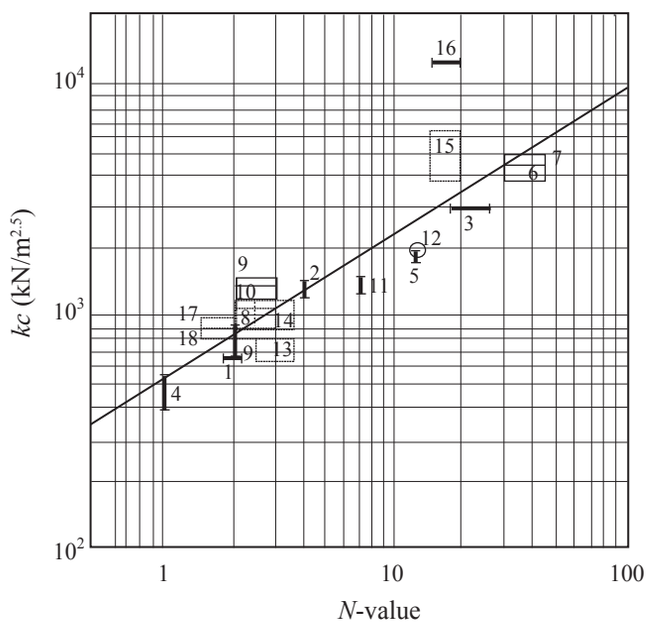
A relationship shown in **Fig. 2.4.14** exists between the rate of increase in the N-value per meter of depth in S-type ground $\overline{N}$ and the lateral resistance of piles k_s.[72)] In cases where the distribution of the N-value in the depth direction does not become 0 at the ground surface, $\overline{N}$ can be determined from the average inclination of the N-value plotting through the zero point at the surface. In C-type ground, a relationship of the type shown in **Fig. 2.4.15** exists between the N-value itself and k_c.[68), 73)] Thus, a rough estimate of k_s or k_c can be made from the distribution of the N-value

.



1. ALTON.ILLINOIS (FEAGIN)
2. WINFIELD.MONTANA (GLESER)
3. PORT HUENEME (MASON)
4.5. Hakkenbori No.1, No.2
6. Ibaragigawa (GOTO)
7. Osaka National Railways (BEPPU)
8.9. Tobata No.6, No.9
10. Tobata K-I (PHRI)
11. Tobata K-II (PHRI)
12. Tobata L-II (PHRI)
13. Kurihama model experiment
14. Shin-Kasai Bridge (TATEISHI)
15. Yamanoshita (IGUCHI)

Fig. 2.4.14 Relationship between $\overline{N}$-value and k_s



1. Tobata K-I (TTRI)
2. Tobata K-III (TTRI)
3. Tobata K-IV (TTRI)
4. Tobata L-II (TTRI)
5. Tobata L-IV (TTRI)
6. Hakkenbori No.1
7. Hakkenbori No.2
8. Osaka National Railways
9. Yahata Seitetsu No.6
10. Yahata Seitetsu No.9
11. Tobata preliminary test-1 (TTRI)-1
12. Tobata preliminary test-2 (TTRI)-2
13. Wagner (Callif.) No.15
14. Wagner (Callif.) No.25
15. Wagner-1 (Alaska)-1
16. Wagner-1 (Alaska)-2
17. Tokyo National Railways b
18. Tokyo National Railways A4
19. Tokyo National Railways B

Fig. 2.4.15 Relationship between N-value and k_c

③ Estimation of lateral resistance constants by loading tests

Estimations of the lateral resistance constants by using the N-value can only provide approximate values. It is preferable to conduct loading tests to obtain more accurate values. The constants k_s and k_c are determined from the ground conditions alone, and are unaffected by other conditions unlike E_s in Chang's equation. Therefore, if k_s or k_c can be obtained by a loading test, those values can be applied to other conditions as well.

④ Effective length

For a certain pile to function as a long pile, its penetration length must be greater than its effective length. Based on the results of model tests with short piles, Shinohara and Kubo found that the lower part of a pile is considered to be fixed completely in the ground when the penetration length exceeds $1.5\ell_{m1}$, and therefore proposed using $1.5\ell_{m1}$ as effective length.[77] Actually, if the penetration length exceeds $1.5\ell_{m1}$, the behavior of the pile will not differ substantially from that of a long pile. However, as the minimum penetration length of long

piles, $1.5\ell_{m1}$ should be used, considering the effects of soil fatigue or creep.

It should also be noted that the value of ℓ_{m1} increases as the stiffness of the pile increases and decreases as the lateral resistance of the ground increases. However, the value of ℓ_{m1} is virtually unaffected by the loading height and pile head fixing conditions. Furthermore, ℓ_{m1} also has the character of increasing gradually as loading increases.

⑤ Effect of pile width

There are two ways in considering the effect of pile width. The first is to consider that the pile width B has no effect on the relationship between the subgrade reaction p per unit area and the displacement y. The second, as proposed by Terzaghi, is to assume that the value of p corresponding to a given y value is inversely proportional to B. Shinohara, Kubo [78] and Sawaguchi [79] conducted model experiments on the relationship between the k_s value in sandy ground and B. The results are shown in **Fig. 2.4.16**. It seems to show a combination of the two theories mentioned above and indicates that the first theory is effective if the pile width B is sufficiently large. On the basis of these results, it was decided not to consider the effect of pile width in the PHRI method.

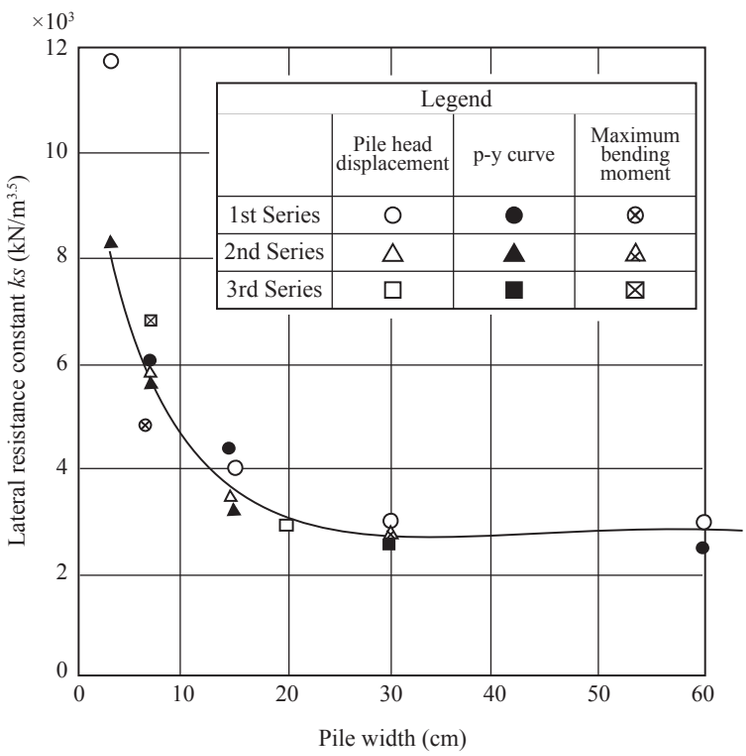


Fig. 2.4.16 Relationship between k_s and Pile Width

⑥ Effect of pile inclination

For batter piles, a relationship shown in **Fig 2.4.17** exists between the inclination angle of the piles and the ratio of the lateral resistance constant of batter piles to that of vertical piles [80] This tigure shows the in-*situ* tests examples which examined driving of batter piles in horizontal ground and the laboratory tests examples obtained by preparing the ground after driving of the batter pile and then sufficiently compacting the ground around the pile. In the in-*situ* tests, when filling was performed after the batter piles were driven, results were obtained in which the coefficient of the subgrade reaction did not increase even when the angle of inclination of the pile is minus. In this case, however, an increase in the coefficient of the subgrade reaction due to subsequent compaction of the surrounding ground can be expected.[81), 82)]

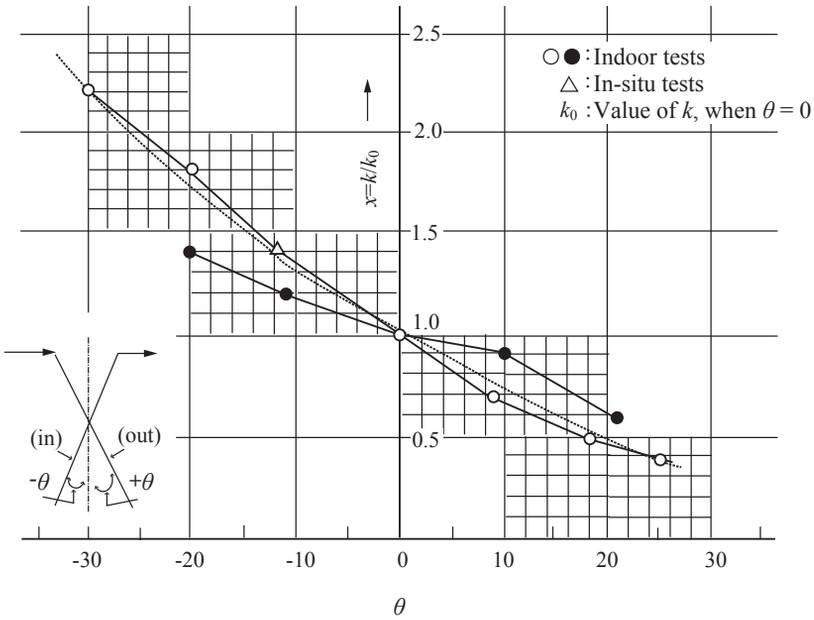


Fig. 2.4.17 Relationship between Pile Inclination Angle and Lateral Resistance Constants

(5) Chang's Method

① Calculation Equation

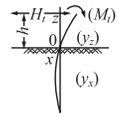
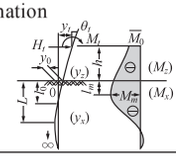
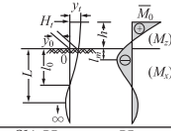
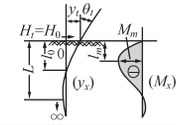
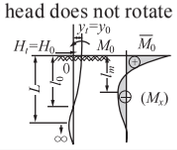
Using the elasticity modulus of the ground $E_s = B\,k_{CH}$, the elasticity equation of piles is expressed as follows;

$$\left.\begin{array}{lll} \text{Exposed section} & EI\dfrac{d^4 y_1}{dx^4} = 0 & (0 \geq x \geq -h) \\ \text{Embedded section} & EI\dfrac{d^4 y_2}{dx^4} + Bk_{CH}\,y_2 = 0 & (x \geq 0) \end{array}\right\} \quad (2.4.40)$$

By calculating these general solutions with $B\,k_{CH}$ as a constant and inputting the boundary conditions, the solution for piles of semi-infinite length can be obtained (see **Table 2.4.6**).[83)]

According to Yokoyama, piles of finite length may be equivalent to the piles of infinite length if $\beta L \geq \pi$. When a pile is shorter than this, a pile must be treated as a finite length pile. Diagrams are available to simplify this process.[85)]

Table 2.4.6 Calculations for Piles of Semi-Infinite Length if k_{ch} is Constant

Differential equations of deflection curve and explanation of symbols	 Exposed sections: $EI\dfrac{d^4y_z}{dz^4}=0$ Embedded sections: $EI\dfrac{d^4y_x}{dx^4}+Bk_{CH}y_x=0$ [Symbols] Ht :Lateral force on pile head(kN) Mt :External force moment on pile head(kN•m) B :Pile diameter(m) EI :Flexural rigidity(kN•m²) k_{CH} :Coefficient of horizontal subgrade reaction(kN/m³) h :Height of pile head above ground(m) β : $\sqrt[4]{Bk_{CH}/4EI}$ (m−1) $h_0=h+\dfrac{M_t}{H_t}$			
Situation of pile	Protruding above ground(h≠0)		Embedded underground(h=0)	
Deflection curve diagram Flexural moment diagram	①Basic formation 	② If pile head does not rotate 	③ Basic system (but Mt = 0) 	④ If pile head does not rotate 
Deflection curve y	$y_z=y_t-\theta_t z+\dfrac{M_t}{2EI}z^2+\dfrac{H_t}{6EI}z^3$ $y_x=\dfrac{H_t}{2EI\beta^3}e^{-\beta x}\left[(1+\beta h_0)\cos\beta x-\beta h_0\sin\beta x\right]$	$y_z=y_t-\dfrac{(1+\beta h)H_t}{4EI\beta}z^2+\dfrac{H_t}{6EI}z^3$ $y_x=\dfrac{H_t}{4EI\beta^3}e^{-\beta x}\left[(1+\beta h)\cos\beta x+(1-\beta h)\sin\beta x\right]$	$y_x=\dfrac{H_t}{2EI\beta^3}e^{-\beta x}\cos\beta x$ (If $Mt\neq0$, use equations in ① putting $h0=Mt/Ht$: the same applies below)	$y_x=\dfrac{H_t}{4EI\beta^3}e^{-\beta x}(\cos\beta x+\sin\beta x)$
Pile head displacement y_t	$y_t=\dfrac{(1+\beta h)^3+1/2}{3EI\beta^3}H_t+\dfrac{(1+\beta h)^2}{2EI\beta^2}M_t$	$y_t=\dfrac{(1+\beta h)^3+2}{12EI\beta^3}H_t=\dfrac{H_t}{K_1}$	$y_t=\dfrac{H_t}{2EI\beta^3}=\dfrac{2\beta H_t}{Bk_{CH}}$	$y_t=\dfrac{H_t}{4EI\beta^3}=\dfrac{\beta H_t}{Bk_{CH}}$
Ground level displacement y_0	$y_0=\dfrac{1+\beta h_0}{2EI\beta^3}H_t$	$y_0=\dfrac{1+\beta h}{4EI\beta^3}H_t$	$y_0=y_t$	$y_0=y_t$
Pile head inclination θ_t	$\theta_t=\dfrac{(1+\beta h)^2}{2EI\beta^2}H_t+\dfrac{1+\beta h}{EI\beta}M_t$	$\theta_t=0$	$\theta_t=\dfrac{H_t}{2EI\beta^2}$	$\theta_t=0$
Flexural moment of pile members M	$M_z=-M_t-H_tz$ $M_x=-\dfrac{H_t}{\beta}e^{-\beta x}\left[\beta h_0\cos\beta x+(1+\beta h_0)\sin\beta x\right]$	$M_z=\dfrac{H_t}{2\beta}(1+\beta h-2\beta z)$ $M_x=\dfrac{H_t}{2\beta}e^{-\beta x}\left[(1-\beta h)\cos\beta x-(1+\beta h)\sin\beta x\right]$	$M_x=-\dfrac{H_t}{\beta}e^{-\beta x}\sin\beta x$	$M_x=-\dfrac{H_t}{\beta}e^{-\beta x}(\cos\beta x-\sin\beta x)$
Shear strength of pile members S	$S_z=-H_t$ $S_x=-H_te^{-\beta x}\left[\cos\beta x-(1+2\beta h_0)\sin\beta x\right]$	$S_z=-H_t$ $S_x=-H_te^{-\beta x}\left[\cos\beta x-\beta h\sin\beta x\right]$	$S_x=-H_te^{-\beta x}(\cos\beta x-\sin\beta x)$	$S_x=-H_te^{-\beta x}\cos\beta x$
Pile head flexural moment $\overline{M}_0$	$\overline{M}_0=-M_t$	$\overline{M}_0=\dfrac{1+\beta h}{2\beta}H_t=\dfrac{K_2}{K_1}H_t$	$\overline{M}_0=0$	$\overline{M}_0=\dfrac{H_t}{2\beta}$
Maximum flexural moment of embedded parts $M_{s,\max}$	$M_{s,\max}=-\dfrac{H_t}{2\beta}\sqrt{(1+2\beta h_0)^2+1}\cdot\exp(-\beta l_m)$	$M_{s,\max}=-\dfrac{H_t}{2\beta}\sqrt{1+(\beta h)^2}\cdot\exp(-\beta l_m)$	$M_{s,\max}=-0.3224\dfrac{H_t}{\beta}$	$M_{s,\max}=-0.2079\dfrac{H_t}{\beta}$
Depth at which $M_{s,\max}$ occurs ℓ_m	$\ell_m=\dfrac{1}{\beta}\tan^{-1}\dfrac{1}{1+2\beta h}$	$\ell_m=\dfrac{1}{\beta}\tan^{-1}\dfrac{1}{\beta h}$	$\ell_m=\dfrac{\pi}{4\beta}=\dfrac{0.7854}{\beta}$	$\ell_m=\dfrac{\pi}{2\beta}=\dfrac{1.5708}{\beta}$
Depth of 1st steady point ℓ_0	$\ell_0=\dfrac{1}{\beta}\tan^{-1}\dfrac{1+\beta h_0}{\beta h_0}$	$\ell_0=\dfrac{1}{\beta}\tan^{-1}\dfrac{\beta h+1}{\beta h\quad 1}$	$\ell_0=\dfrac{\pi}{2\beta}=\dfrac{1.5708}{\beta}$	$\ell_0=\dfrac{3\pi}{4\beta}=\dfrac{2.3562}{\beta}$
Depth of deflection angle zero point L	$L=\dfrac{1}{\beta}\tan^{-1}\left[-(1+2\beta h_0)\right]$	$L=\dfrac{1}{\beta}\tan^{-1}(-\beta h)$	$L=\dfrac{3\pi}{4\beta}=\dfrac{2.3562}{\beta}$	$L=\dfrac{\pi}{\beta}=\dfrac{3.1416}{\beta}$
Pile head rigidity factor K_1,K_2,K_3,K_4	$K_1=\dfrac{12EI\beta^3}{(1+\beta h)^3+2}$ $K_2=K_3=K_1\dfrac{1+\beta h}{2\beta}$ $K_4=\dfrac{4EI\beta}{1+\beta h}\dfrac{(1+\beta h)^3+1/2}{(1+\beta h)^3+2}$		$K_1=4EI\beta^3$ $K_2=K_3=2EI\beta^2$ $K_4=2EI\beta$	

② Estimation of k_{CH} in Chang's method

(a) Terzaghi's proposal [86)]

Terzaghi proposed the following values for the coefficient of lateral subgrade reaction in cohesive or sandy soil:

1) In case of cohesive soil

$$k_{CH} = \frac{0.2}{B}\bar{k}_{CH1} \tag{2.4.41}$$

where

k_{CH} : coefficient of lateral subgrade reaction (kN/m³)
B : pile width (m)
$\bar{k}_{CH1}$: value shown in **Table 2.4.7**

$$E_s = k_{CH}B = 0.2\bar{k}_{CH1} \tag{2.4.42}$$

2) In case of sandy soil

$$k_{CH} = n_h\frac{x}{B} \tag{2.4.43}$$

where

x : depth (m)
B : pile width (m)
n_h : value listed in **Table 2.4.8**

$$E_s = k_{CH}B = n_h x \tag{2.4.44}$$

In sandy soil, E_s is a function of depth and thus cannot be applied directly to Chang's method. For such cases, Chang states that E_s can be taken the value at the depth of one third of ℓ_{y1} which is the depth of the first zero-displacement point. However, ℓ_{y1} itself is a function of E_s, thus repeated calculations have to be made to obtain the value of E_s. Reference 87) describes the method of calculation without the repetition calculation.

Terzaghi assumes that the value of k_{CH} is inversely proportional to the pile width B, as shown in equations (**2.4.43**) and (**2.4.44**). Other opinions suggest that pile width is irrelevant to k_{CH} (see (4) ⑤).

Table 2.4.7 Coefficient of Lateral Subgrade Reaction

Consistency of cohesive soil	Hard	Very hard	Solid
Unconfined compressive strength q_u (kN/m²)	100–200	200–400	400 or greater
Range of $\bar{k}_{CH1}$ (kN/m²)	16,000–32,000	32,000–64,000	64,000 or greater
Proposed value of $\bar{k}_{CH1}$ (kN/m³)	24,000	48,000	96,000

Table 2.4.8 Value of n_h

Relative density of sand	Loose	Medium	Dense
n_h for dry or wet sand (kN/m³)	2,200	6,600	17,600
n_h for submerged sand (kN/m³)	1,300	4,400	10,800

(b) Yokoyama's proposal

Yokoyama collected the results of lateral loading tests on steel piles conducted in Japan and performed reverse calculations for k_{CH}, and obtained **Fig. 2.4.18** by comparing the results and the mean N-values at depths down to β^{-1} from the ground level.[88)] In this case, $Es = k_{CH}B$ is assumed to be valid for both sandy soil and cohesive soil, and k_{CH} itself is assumed not to be affected by B. Although the values of k_{CH} obtained by reverse calculation from the measured values decrease as loading increases, **Fig. 2.4.18** is prepared using k_{CH} when the ground surface displacement is 1cm. **Fig. 2.4.18** may be used when making rough estimates of the value of E_s from soil conditions alone without conducting loading tests in-*situ*.

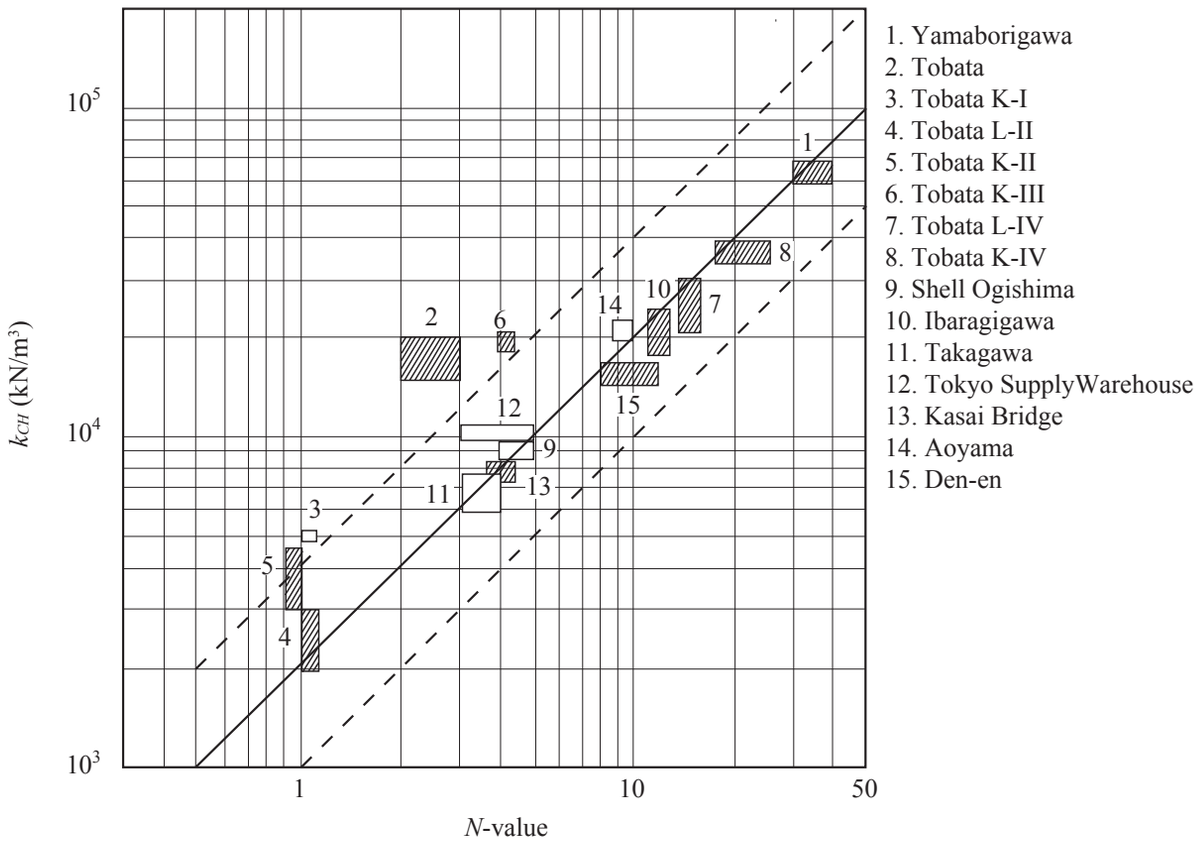


Fig. 2.4.18 Values of kCH obtained by Reverse Calculation from Horizontal Loading Tests on Piles

(c) Relationship between k_c, k_s, and k_{CH} [89), 90)]

From **Fig. 2.4.14**, **Fig. 2.4.15**, and **Fig. 2.4.18**, the relationships between the SPT-*N*-values or $\overline{N}$-values shown in the respective figures and the corresponding coefficients of subgrade reaction are as shown in **Table 2.4.9**. As can be understood from these results, there are largely dispersed relationships between k_{CH} and the *N*-value. These results are due to the fact that the value of k_{CH} cannot be determined from the soil conditions alone.

Hence, the relationship between k_c and k_{CH} and that between k_s and k_{CH} can be obtained in such a way that ground surface displacement was equal under the same loading conditions. Then, substituting the relational equations of k_c, k_s, and the *N*-value or $\overline{N}$-value, the following equations can be obtained.

$$
\begin{aligned}
k_{CH} &= 103(EI/D)^{0.207}\, y_0^{-0.398} \cdot h^{-0.035} \cdot \overline{N}^{0.519} \text{ (free pile head)} \\
k_{CH} &= 114(EI/D)^{0.216}\, y_0^{-0.392} \cdot h^{-0.088} \cdot \overline{N}^{0.513} \text{ (fixed pile head)} \\
k_{CH} &= 719(EI/D)^{-0.001}\, y_0^{-0.499} \cdot h^{0.009} \cdot N^{0.649} \text{ (free pile head)} \\
k_{CH} &= 683(EI/D)^{-0.005}\, y_0^{-0.501} \cdot h^{0.028} \cdot N^{0.651} \text{ (fixed pile head)}
\end{aligned}
\qquad \textbf{(2.4.45)}
$$

Table 2.4.9 Relationships between SPT-*N*-value or $\overline{N}$-value and Respective of Subgrade Reaction

Correlation equation	Correlation coefficient	Coefficient of variation
$k_c = 540N^{0.648}$ (kN/m$^{2.5}$)	0.872	0.111
$k_s = 592\overline{N}^{0.654}$ (kN/m$^{3.5}$)	0.966	0.077
$k_{CH} = 3910N^{0.733}$ (kN/m^3)	0.917	0.754