SEISMIC DESIGN OF BUILDING FOUNDATIONS IN JAPAN

1. Introduction

An overview of seismic design of foundations described in the literature[1)-3)] and the allowable unit stress of piles and others under Notification No. 1113 are presented in Sections 2 to 5. In Section 6, the fundamentals and issues related to the design of foundations to withstand a great earthquake are described. Finally, in Section 7, the characteristics of overseas legal provisions concerning the seismic design of foundations are outlined and comparisons are made with Building Standard Law of Japan (BSLJ).

2. Basic concept

The building foundations that may be subjected to seismic force should be designed so as to maintain structural safety equivalent to or exceeding that of the upper structure. It is important to conduct an investigation according to the state of the site with respect to possible ground deformation such as liquefaction and landslide, and measures such as appropriate soil improvement should be taken as needed.

It should be noted that under BSLJ Article 20, structural calculations including seismic design are not mandatory for the foundations of small buildings. Buildings of this type are covered in BSLJ Article 6-1 as follows:

a) Wooden buildings with a total floor area of 500 m^2 or less and up to 2 stories.
b) Non-wooden buildings with a total floor area of 200 m^2 or less and 1 story.

3. Design external forces during an earthquake

3.1 General external forces

The design horizontal force P_h of a foundation is obtained by adding the horizontal force on the underground part of building including the foundation to the horizontal shear force P_{ho} (Equation 3.1.1) of the lowest story for upper ground part of building design under BSLJ Enforcement Ordinance 88 (Equation 3.1.2). The horizontal force of an underground structure is obtained by multiplying the sum of the fixed load of the foundation, the live load, and the weight of soil on the foundation slab by the horizontal seismic coefficient of Equation 3.1.3, according to Article 88-4.

$$P_{ho} = Ci * Ws = Z * Ai * Rt * Co * Ws \qquad (3.1.1)$$

Ci : seismic story shear coefficient of the aboveground part of a building at a given height
W_s : weight of superstructure (kN)
Z : zoning factor between 0.7 and 1.0, established by the Minister of Construction
Ai : a value indicating vertical distribution of the seismic story shear coefficient according to the vibration characteristics of the building
Rt : a value indicating the vibration characteristics of the building
Co : standard shear coefficient ($Co \geqq 0.2$ with special exceptions)

$$P_h = P_{ho} + k * W_g \qquad (3.1.2)$$

$$k = 0.1 * (1 - H_f / 40) Z \quad (3.1.3)$$

P_h : design horizontal force of foundation (kN)
P_{ho} : horizontal shear force of the lowest story (kN)
W_g : weight of foundation (kN)
k : seismic coefficient of underground part of a building
H_f : depth of each part of the underground structure of the building from the ground surface; 20 (m) at depths of > 20 m
Z : zoning factor between 0.7 and 1.0, established by the Minister of Construction

3.2 Reduction in horizontal force due to foundation slab embedment effect in pile foundation

The design horizontal force on a pile foundation is obtained by subtracting the resistant force due to the embedment effect of the foundation slab from the total horizontal force on the foundation. This resistant force includes passive resistance of the underground exterior wall as well as frictional resistance of the side of the exterior wall and the foundation bed.

For a pile foundation, the horizontal force P_{hp} at the bottom of the foundation slab is usually calculated using the following equation.

$$P_{hp} = P_h * (1 - \alpha) \quad (3.2.1)$$

$$\alpha = 1 - 0.2 \frac{\sqrt{H}}{\sqrt[4]{D_f}} \quad (3.2.2)$$

P_{hp} : horizontal force at the bottom of the foundation slab
α : allocation ratio of horizontal force at foundation slab embedment (Max. 0.7)
H : height of superstructure (m)
D_f : depth of embedment of the foundation (m) (in principle, $D_f \geqq 2$ m)

Fig. 3.2.1. shows the relationship between α and H and D_f. Equation 3.2.2 is based on a trial calculation in which frictional resistance in the side is taken into consideration assuming that the passive structure of embedment is an elastic spring.[1)-3)]

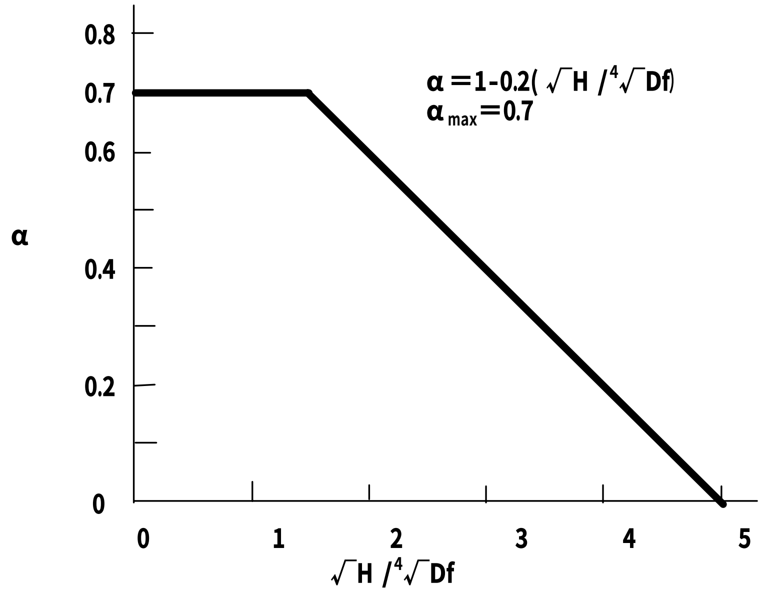

Figure 3.2.1 Relation among α, H and Df

Equation 3.2.2 is applicable at $D_f \geqq 2$ m, because it is possible that the passive structure depends on backfilling and exerts limited resistant force at a reduced D_f. However, Equation 3.2.2 could probably be adopted even at $D_f < 2$ m if backfilling has been desirably obtained.

According to measured earth pressure during earthquakes, incremental earth pressure on underground exterior walls at the time of an earthquake is constant in the depth direction of those walls, or has a reverse triangle-like distribution in which the upper earth pressure is greater than the lower earth pressure.[4)]

4. Design of spread foundations

In the case of spread foundations, the contact pressure caused by vertical and horizontal forces on the foundations must be within the allowable bearing capacity of the ground for a short period. It should also be confirmed that the foundations will not slide out, if necessary.

The allowable bearing capacity of the ground for a short period is prescribed in Notification No. 1113 (Table 4.1.1).

Table 4.1.1 Allowable unit stress of ground

	Allowable unit stress of ground under forces generated for a long period	Allowable unit stress of ground under forces generated for a short period
(1)	$qa = \frac{1}{3}(i_c \alpha C N_C + i_\gamma \beta \gamma_1 B N\gamma + i_q \gamma_2 D_f Nq)$	$qa = \frac{2}{3}(i_c \alpha C N_C + i_\gamma \beta \gamma_1 B N\gamma + i_q \gamma_2 D_f Nq)$
(2)	$qa = qt + \frac{1}{3} N' \gamma_2 D_f$	$qa = 2qt + \frac{1}{3} N' \gamma_2 D_f$
(3)	$qa = 30 + 0.6\overline{Nsw}$	$qa = 60 + 1.2\overline{Nsw}$

Qa: allowable unit stress of the ground (unit: kilo-Newton/m^2)

$i_c = i_q = (1 - \theta / 90)^2$
$i_r = (1 - \theta / \phi)^2$

θ: inclination to the vertical of load acting on the foundation (if θ>φ, θ=φ.) (unit: °)

φ: internal friction angle obtained according to properties of the ground (unit: °)

α and β: coefficients according to the shape of the foundation load surface

Shape of the foundation load surface / Coefficient	Circular	Shapes other than circular
α	1.2	1.0 + 0.2 *B/L*
β	0.3	0.5 – 0.2 *B/L*

C: cohesion of the ground under the foundation load surface (unit: kilo-Newton/m^2)
B: short side or short diameter of the foundation load surface (unit: m)
Nc, *Nr* and *Nq*: coefficient of capacity according to the internal friction angle of the ground

Internal friction angle / Coefficient of capacity	0°	5°	10°	15°	20°	25°	28°	32°	36°	40° or more
Nc	5.1	6.5	8.3	11.0	14.8	20.7	25.8	35.5	50.6	75.3
Nr	0	0.1	0.4	1.1	2.9	6.8	11.2	22.0	44.4	93.7
Nq	1.0	1.6	2.5	3.9	6.4	10.7	14.7	23.2	37.8	64.2

γ_1: unit weight of ground under the foundation load surface (kN/m^3)
γ_2: mean unit weight of ground above the foundation load surface (kN/m^3)
D_f: depth from the lowest ground surface to the foundation load surface (m)
qt : the smaller of two values: 1/2 of the yield load or 1/3 of the ultimate stress (kN/m^2)
N' : coefficient according to the category of ground under the foundation load surface

Category of ground / Coefficient	Dense sandy ground	Sandy ground (excluding dense sandy ground)	Cohesive ground
N'	12	6	3

$\overline{Nsw}$: mean value of the half-rotation number per 1 m at a vertical distance within 2 m from the bottom of the foundation (if $\overline{Nsw}$>150, $\overline{Nsw}$=150)

For spread foundations, it can usually be expected that a coefficient of friction of 0.3 to 0.5 between the foundation bed and the ground will not cause sliding, but it is important to investigate sliding when there is a horizontal force due to one-sided earth pressure constantly acting on the foundations.

Notification No. 1113-2 provides a formula for the allowable unit stress of the ground considering the gradient of the load (Table 4.1.1(1)), and it should be noted that the allowable unit stress will change as a result of an oblique angle. Horizontal force on a building acts to change the contact pressure, whereas contact pressure will not often be negative in the primary design. However, in the case of tower-like buildings and those with buoyancy, great care should be taken regarding the distribution of contact pressure and stiffness of footing beams, because uplift of the foundation bed could be generated by primary design external force.

5. Design of pile foundations

5.1 Investigation of vertical force

In design with respect to the vertical force acting on piles, the acting force must be within the allowable bearing capacity.

5.2 Investigation of horizontal force

The bending moment and displacement of a pile are determined considering the pile as an elastic bearing beam. Usually, the displacement of a pile head y_o caused by horizontal force, bending moment of a pile head M_o, maximum bending moment M_{max} of a pile in the ground, and its depth L_m can be calculated using the following equations.

$$y_0 = \frac{Q}{4EI\beta^3} R_{y0} \quad \text{(m)} \qquad (5.2.1)$$

$$M_0 = \frac{Q}{2\beta} R_{M0} \quad (\text{kN} \bullet \text{m}) \qquad (5.2.2)$$

$$M_{max} = \frac{Q}{2\beta} R_{M\,max} \quad (\text{kN} \bullet \text{m}) \qquad (5.2.3)$$

$$l_m = \frac{1}{\beta} R_{lm} \quad \text{(m)} \qquad (5.2.4)$$

However,

$$\beta = \sqrt[4]{\frac{k_h B}{4EI}} \quad (\text{m}^{-1}) \qquad (5.2.5)$$

$$R_{y0} = 2 - \alpha_r \qquad (5.2.6)$$

$$R_{M0} = \alpha_r \qquad (5.2.7)$$

$$R_{M\,max} = exp\left[-tan^{-1}\left(\frac{1}{1-\alpha_r}\right)\right]\sqrt{(1-\alpha_r)^2+1} \qquad (5.2.8)$$

$$R_{lm} = tan^{-1}\left(\frac{1}{1-\alpha_r}\right) \qquad (5.2.9)$$

Q : horizontal force of a pile head (kN)
k_h : coefficient of horizontal subgrade reaction (kN/m^3)
B : pile diameter (m)
E : Young's modulus of a pile (kN/m^2)
I : moment of second order of a pile (m^4)
α_r : fixing ratio of a pile head joint (1 for fixed, 0 with pin)

$\beta L \geqq 3.0$ is assumed for the length of a pile L (m). At $\beta L < 3.0$, a separate calculation for short piles will be needed. Although the fixing ratio of pile head joints should be established based on special investigational experiments, if the fixing ratio is not identified then the joint is considered to be fixed. Recently, a seismically isolated device used at the pile head level and a jointing system that realizes a pile head pin have been developed.

5.3 Horizontal bearing power of a pile

5.3.1 Calculation methods for horizontal bearing power

The methods of calculating the horizontal bearing power of a pile include the ultimate subgrade reaction, linear elastic subgrade reaction, and nonlinear elastic subgrade reaction methods, with the linear elastic subgrade reaction method usually being used in primary design.

In the linear elastic subgrade reaction method, a pile is considered to be a beam on an elastic bearing and the subgrade reaction is assumed to be directly proportional to the displacement of the beam, based on the differential equation in Equation 6.4.10. In this method, the coefficient of horizontal subgrade reaction k_h should be appropriately established. Equations 5.2.1-5.2.9 were obtained at m = 0 in Equation 5.3.1.1.

$$EI\frac{d^4y}{dx^4} + k_h B x^m y = 0 \qquad (5.3.1.1)$$

EI : flexural rigidity of a pile (kN/m^2)
y : horizontal displacement (m)
x : depth from the ground surface (m)
k_h : coefficient of horizontal subgrade reaction (kN/m^3)
B : pile diameter (m)
m : constant

5.3.2 Coefficient of horizontal subgrade reaction k_h

The coefficient of horizontal subgrade reaction k_h can be determined by either (1) an evaluation method using the horizontal loading test, or (2) an estimation method according to the results of soil investigation.

The former is preferable for an important structure, because direct evaluation of the relationship between the horizontal displacement and load at the site concerned can provide high precision. In the latter method, the following equation is commonly applied. However, since this equation assumes that the horizontal displacement of the pile head is about 1 cm or less, nonlinearity of k_h should be appropriately taken into consideration for excessive pile head displacement obtained by calculation.

$$k_h = 80 * E_0 * (B * 100)^{-3/4} = 2.5 * E_0{}^* B^{*-3/4} \qquad (5.3.2.1)$$

k_h : coefficient of horizontal subgrade reaction (kN/m^3)

E_0 : modulus of deformation of the ground. The value used should be one of the following; however, that of cohesive soil should not be estimated from the N value, but determined using either ① or ②:

① modulus of deformation of the ground measured in a boring hole (kN/m^2)

② modulus of deformation determined using the uniaxial or triaxial compression test (kN/m^2)

③ modulus of deformation estimated at E_0 = 7N based on the mean N value (kN/m^2).

B : diameter of a pile (m)

The range of soil investigation to determine k_h is often 1/**β** below the foundation bed, a depth that predominantly influences the horizontal resistance of a pile.

5.3.3. Flexural rigidity of a pile

The flexural rigidity of a pile is determined assuming a pile to be an elastic material. Preferably, the experimentally obtained Young's modulus E is used, but if there is no experimental result the following table is useful.

Table 5.3.3.1 Young's modulus E of a pile (kN/m^2)

Steel		2.05×10^8
PC steel		1.96×10^8
Concrete	Cast-in-place concrete pile	$3.35 \times 10^7 \times (Fc/60)^{1/3}$
	Centrifugal reinforced concrete pile	3.5×10^7
	Prestressed concrete pile	4.0×10^7
	Prestressed high-strength concrete pile	4.0×10^7

The moment of second order of a pile can be considered for the entire section (total sectional area without a corrosion allowance for steel pipe piles, and that of concrete and bar for concrete piles) assuming the pile center to be a neutral axis.

5.3.4 Fixing ratio of pile head joints

The fixing ratio of pile head joints is commonly defined as α_r in the following equation.

$$\alpha_r = \frac{M_0}{M_{0f}} = \frac{2\beta M_0}{Q} \quad (5.3.4.1)$$

Q : horizontal force of the pile head

M_0 : bending moment of the pile head

M_{0f} : bending moment of the pile head estimated when the pile head is fixed

β : value in Equation 5.2.5

The value of α_r is 1 for a fixed pile head and 0 for a pile head pin.

5.4 Investigation of unit stress of piles

In seismic design with respect to a medium earthquake, the unit stress on a pile must be within the allowable unit stress of a pile for a short period. The unit stress of a pile that is subjected to an axial force and bending moment is usually investigated according to the following assumptions.

① The unit stress of the pile is investigated based on elasticity theory.

② The section of the pile that is subjected to the bending moment remains flat, and the unit stress of each point is proportional to the distance from the neutral axis.

5.4.1 Cast-in-place concrete pile

The unit stress of a cast-in-place concrete pile is investigated so that the following equations and inequality are satisfied.

1) Axial force and bending moment

a) When the bar on the tension side of the pile section reaches the allowable tensile unit stress:

$$(\varepsilon_0 - \phi_0\gamma_t)E_s = {}_s\!\int_t \qquad (5.4.1.1)$$

b) When the bar on the compression side of the pile section reaches the allowable compressive unit stress:

$$(\varepsilon_0 + \phi_0\gamma_c)E_s = {}_s\!\int_c \qquad (5.4.1.2)$$

c) When the concrete on the compression side of the pile section reaches the allowable compressive unit stress:

$$(\varepsilon_0 + \phi_0\gamma_0)E_c = {}_c\!\int_c \qquad (5.4.1.3)$$

${}_s\!\int_t$: allowable tensile unit stress of a bar (N/m^2)

${}_s\!\int_c$: allowable compressive unit stress of a bar (N/m^2)

${}_c\!\int_c$: allowable compressive unit stress of concrete (N/m^2)

γ_t : distance between the center of the pile and the outermost bar on the tension side (mm)

γ_c : distance between the center of the pile and the outermost bar on the compression side (mm)

γ_0 : radius of the pile (mm)

ε_0 : axial strain at the center of the pile (positive for compression)

ϕ_0 : curvature of the pile associated with the bending moment (1/mm)

E_s : Young's modulus of the bar (N/m^2)

E_c : Young's modulus of concrete (N/m^2)

2) Shear stress

$$\kappa \frac{\boldsymbol{Q}}{\boldsymbol{A}_s} \leqq \int_s \tag{5.4.1.4}$$

$\int_s$: allowable shear unit stress of concrete (N/mm^2)

$\boldsymbol{Q}$: design shear force (N)

$\boldsymbol{A}_s$: sectional area of the pile (mm^2)

κ : distribution coefficient of shear unit stress, 4/3

5.4.2 Centrifugal prestressed concrete pile

The unit stress of centrifugal prestressed concrete pile is investigated so that the following inequality is satisfied, converting the effect of PC steel to that of concrete.

1) Axial force and bending moment

In the presence of an axial force and bending moment, all edge unit stresses on the bending compression and bending tension sides under compressive and tensile forces should satisfy the following inequality.

$$-\int_b \leqq \frac{\boldsymbol{N}}{\boldsymbol{A}_e} + \sigma_e + \frac{\boldsymbol{M}}{\boldsymbol{I}_e} \boldsymbol{y} \leqq \int_c \tag{5.4.2.1}$$

However: $\boldsymbol{A}_e = \boldsymbol{A}_c + \eta \boldsymbol{A}_S$ (5.4.2.2)

$\boldsymbol{I}_e = \boldsymbol{I}_c + \eta \boldsymbol{I}_S$ (5.4.2.3)

$\eta = \boldsymbol{E}_S / \boldsymbol{E}_C$ (5.4.2.4)

$\int_b$: allowable bending tensile unit stress of concrete (N/mm^2)

$\int_c$: allowable compressive unit stress of concrete (N/mm^2)

$\boldsymbol{N}$: design axial force (N); positive for compressive force and negative for tensile force

$\boldsymbol{M}$: design bending moment (N • mm); positive value

$\boldsymbol{y}$: radius of the pile (mm); positive for bending compression side and negative for bending tension side

σ_e : effective prestress (N/mm^2)

$\boldsymbol{A}_e$: equivalent sectional area of concrete (mm^2)

$\boldsymbol{A}_C$: sectional area of concrete (mm^2)

$\boldsymbol{A}_S$: sectional area of PC steel (mm^2)

$\boldsymbol{I}_e$: equivalent moment of second order of concrete for the pile center (mm^4)

$\boldsymbol{I}_C$: moment of second order of concrete for the pile center (mm^4)

$\boldsymbol{I}_S$: moment of second order of PC steel for the pile center (mm^4)

η : ratio of Young's modulus of PC steel to that of concrete

$\boldsymbol{E}_C$: Young's modulus of concrete (N/mm^2)

E_S : Young's modulus of PC steel (N/mm^2)

2) Shear force

Shear force is investigated using the following inequality according to Article 71 of "Criteria for Design and Construction of Prestressed Concrete and Commentary Thereon" by the Architectural Institute of Japan.

$$\tau_{max} \leqq \frac{1}{2}\sqrt{(\sigma_g + 2\sigma_d)^2 - \sigma_g^{\ 2}} \qquad (5.4.2.5)$$

τ_{max} : maximum shear unit stress (N/mm^2)

Q : design shear force (N)

t : thickness of the pile (mm)

S_0 : geometrical moment of area for the neutral axis of a unilateral pile section of the neutral axis of the pile (mm^3), which can be expressed as $S_0 = \frac{2}{3}(\gamma_0^{\ 3} - \gamma_i^{\ 3})$

I : moment of second order for the neutral axis of the pile (mm^4), which can be expressed as $I = \frac{\pi}{4}(\gamma_0^{\ 4} - \gamma_i^{\ 4})$

σ_g : axial unit stress (N/mm^2),

which can be expressed as $\sigma_g = \sigma_e + \frac{N}{A_e}$

σ_e : effective prestress (N/mm^2)

N : design axial force (N); positive for compressive force, negative for tensile force

A_e : equivalent sectional area of concrete (mm^2)

σ_d : allowable diagonal tensile unit stress of concrete (N/mm^2)

γ_0 : external radius of the pile (mm)

γ_i : internal radius of the pile (mm)

5.4.3 Steel pipe pile

The unit stress of steel pipe pile is investigated so that the following inequalities are satisfied.

1) Compressive force and bending moment

The following inequalities are applied to a pile that is subjected to a compressive force and bending moment.

$$\frac{\frac{N}{A_e}}{\int_c} + \frac{\frac{M}{I_e/\gamma_{ec}}}{\int_b} \leqq 1 \qquad \text{(compression side)} \qquad (5.4.3.1)$$

$$-\frac{\frac{N}{A_e}}{\int_t}+\frac{\frac{M}{I_e/\gamma_{et}}}{\int_t}\leqq 1 \qquad \text{(bending tension side)} \qquad (5.4.3.2)$$

2) Tensile force and bending moment

The following inequalities are applied to a pile that is subjected to a tensile force and bending moment.

$$\frac{\frac{T}{A_e}}{\int_t}+\frac{\frac{M}{I_e/\gamma_{et}}}{\int_t}\leqq 1 \qquad \text{(tension side)} \qquad (5.4.3.3)$$

$$-\frac{\frac{T}{A_e}}{\int_b}+\frac{\frac{M}{I_e/\gamma_{ec}}}{\int_b}\leqq 1 \qquad \text{(bending compression side)} \qquad (5.4.3.4)$$

3) Shear force

The following inequality is applied to a pile that is subjected to shear force.

$$\kappa\frac{\frac{Q}{A_e}}{\int_S}\leqq 1 \qquad (5.4.3.5)$$

γ_{et} : distance between the center and outermost part of the pile on the tension side excluding corrosion allowance (mm)

γ_{ec} : distance between the center and outermost part of the pile on the compression side excluding corrosion allowance (mm)

A_e : sectional area excluding corrosion allowance (mm^2)

I_e : moment of second order excluding corrosion allowance (mm^4)

N : design axial compressive force (N)

T : design axial tensile force (N)

M : design bending moment (N•mm)

Q : design shear force (N)

$\int_c$: allowable compressive unit stress of steel (N/mm^2)

$\int_b$: allowable bending unit stress of steel (N/mm^2)

$\int_t$: allowable tensile unit stress of steel (N/mm^2)

$\int_S$: allowable shear unit stress of steel (N/mm^2)

κ : distribution coefficient of shear stress, 2.0

5.5 Allowable unit stress of a pile

5.5.1 Concept and properties of allowable unit stress

The allowable unit stress of a pile is prescribed in Notification No. 1113. The following are the features of the provisions.

1) Safety factor of concrete

In the case of an upper structure, a safety factor of 3 for a long period and basically 2 for a short period are adopted for concrete, while a safety factor of 4 for compressive unit stress for a long period and 2 for a short period have traditionally been adopted for concrete piles. In addition, since the quality of cast-in-place concrete piles depends on their construction, a safety factor of 4.5 is adopted for a long period when there is no reliable maintenance or confirmation of the use of water and slurry. The allowable shear unit stress for temporary loading and bond stress of concrete of cast-in-place piles is 1.5 times the allowable unit stress for a long period, and double that for general concrete.

2) Corrosion of steel pipe

The following are pointed out with regard to the corrosion of steel pipe piles based on measurements of various grounds.[1),6)]

① In the corrosion of steel piles, the mean value of the annual double-side corrosion rate measured for 10 years is 0.0106 mm when determined mechanically with no consideration of established conditions.

② Of all the experimental piles, the maximum annual double-side corrosion rate is 0.0297 mm. The standard deviation of the measured annual corrosion rate is 0.005 mm; therefore, the maximum corrosion rate will not exceed the standard deviation of the mean value plus a 4-fold larger value.

③ The annual corrosion rate decreases with the passage of time after installation of a pile.

Considering these findings, the external corrosion allowance is 1 mm in principle. However, corrosion can be expected to progress extremely rapidly in the vicinity of a chemical plant or under other special ground conditions such as hot spring areas from which chemicals including sulfur well out, and special investigation will be needed.

5.5.2 Allowable unit stresses of various piles

1) Cast-in-place concrete piles

The allowable unit stresses of concrete in cast-in-place concrete piles are shown in Table 5.5.2.1.

Table 5.5.2.1　Allowable unit stress of concrete in cast-in-place concrete piles

Method of execution		Allowable unit stress for a long period (N/mm^2)			Allowable unit stress for a long period (N/mm^2)		
		Compression	Shearing	Adhesion	Compression	Shearing	Adhesion
(1)	A method without water or slurry during excavation and cases where the strength and shape were confirmed.	F/4	Smaller value of F/40 or 3/4(0.49+F/100)	Smaller value of 3/40F or 3/4(1.35+F/25)	Double the value of the allowable unit stress of compression for a long period	1.5 times the value of the allowable unit stress of shearing and adhesion, respectively, for a long period	
(2)	Cases other than those in (1)	Smaller value of F/4.5 or 6	Smaller value of F/4.5 or 3/4(0.49+F/100)	Smaller value of F/15 or 3/4(1.35+F/25)			
F：specified concrete strength (N/mm^2)							

2) Prefabricated concrete piles

The allowable unit stresses of prefabricated concrete piles are shown in Tables 5.5.2.2 to 5.5.2.6.

Table 5.5.2.2　Allowable unit stresses of concrete in centrifugal reinforced concrete piles and vibration-filled concrete piles(RC piles)

Allowable unit stress for a long period (N/mm^2)			Allowable unit stress for a short period (N/mm^2)		
Compression	Shearing	Adhesion	Compression	Shearing	Adhesion
Smaller value of F/4 or 11	Smaller value of 3/4(0.49+F/100) or 0.7	Smaller value of 3/4(1.35+F/25) or 2.3	Double the value of the allowable unit stress of compression for a long period	1.5 times the value of the allowable unit stress of shearing and adhesion, respectively, for a long period	
F：specified concrete strength (N/mm^2)					

Table 5.5.2.3　Allowable unit stresses of concrete in SC piles

Allowable unit stress of compression for a long period (N/mm^2)	Allowable unit stress of compression for a short period (N/mm^2)
F/4	Double the value of the allowable unit stress of compression for a long period
F：specified concrete strength (N/mm^2)	

Table 5.5.2.4　Allowable unit stresses of prestressed concrete piles(PC piles)

Allowable unit stress for a long period (N/mm^2)			Allowable unit stress for a short period (N/mm^2)		
Compression	Bending tension	Diagonal tension	Compression	Bending tension	Diagonal tension
Smaller value of F/4 or 15	Smaller value of $\sigma c/4$ or 2	Smaller value of 0.07/4*F or 0.9	Double the value of the allowable unit stress of compression and bending tension, respectively, for a long period		1.5 times the value of the allowable unit stress of diagonal tension for a long period
F: specified concrete strength (N/mm^2), σ_c : effective prestress(N/mm^2)					

Table 5.5.2.5 Allowable unit stresses of concrete in PHC piles

Effective prestress	Allowable unit stress for a long period (N/mm^2)			Allowable unit stress for a short period (N/mm^2)		
	Compression	Bending tension	Diagonal tension	Compression	Bending tension	Diagonal tension
4	20	1.0	1.2	40	2.0	1.8
8	24	2.0		42.5	4.0	
10	24	2.5		42.5	5.0	

Table 5.5.2.6 Effective prestress and specified design strength of PHC piles

Effective prestress (N/mm^2)	Specified concrete strength (N/ mm^2)
4	80 or more
8	85 or more
10	

Tables 5.5.2.1 to 5.5.2.1 show the allowable unit stresses for general piles. For special piles, on the other hand, if tests using piles demonstrate safety in terms of structural resistance, the allowable unit stress can be determined from the results of these tests. These tests using piles should be performed for each of the structural method, construction method, and allowable unit stress of a pile.

3) Allowable unit stresses of steel used as a pile

Table 5.5.2.7 shows the allowable unit stresses of steel in steel pipe piles. A corrosion allowance should be considered appropriately in the design. The reduction ratio R_c of allowable unit stress considering the radius thickness ratio is provided for the allowable unit stress of compression and bending.

Table 5.5.2.7 Allowable unit stresses of steel pipe piles

Allowable unit stress of compression for a long period (N/mm^2)				Allowable unit stress of compression for a short period (N/mm^2)
Compression	Tension	Bending	Shearing	1.5 times the respective values of allowable unit stress for a long period
F*R$_f$/1.5	F/1.5	F*R$_f$/1.5	F/(1.5$\sqrt{3}$)	

$$R_f = 0.80 + 2.5(T - C) / R$$

R_f : reduction factor (not to exceed 1.0)

T : thickness of the steel pile (mm)

C : corrosion allowance (to be 1 or more excluding cases where effective corrosion prevention measures are taken) (mm)

R : radius of the pile (mm)

F : material strength of the steel (usually, 235 N/mm^2 or 270 N/mm^2 for steels used for foundations)

In the case of jointed piles, the allowable unit stress of a pile was previously reduced superposedly depending on the number of joints, but in the amendment, this reduction was eliminated considering the elimination of a reduction for welded joints of general steel and preexisting experimental results. Welded joints of steel pipe pile for which no reduction ratio is required are limited to those corresponding to JIS-A5525, considering dimensional roundness (tongue-and-groove intersection). The reduction method for other joints should be established based on the results of tests for each jointing method considering executability.

Apart from steel pipe, the allowable unit stresses of other materials such as deformed bars should be based on Table 5.5.2.8. However, the reduction factor is 1. The allowable unit stresses of PC steel bar are pursuant to Table 5.5.2.8.

Table 5.5.2.8 Allowable unit stresses of PC steel bar

First tension	$0.70f_1$ or $0.80f_2$, whichever is smaller

f_1: specified tensile strength of PC steel, f_2: specified yield point strength of PC steel bar

6. Design technique for pile foundations during a severe earthquake

There have been many reports on damage to pile foundations caused by Hyogoken Nanbu earthquake in 1995(Photo 6.1.1-6.1.4). Many of these cases consisted of bending failure of piles. Shear failures of pile caps and footing beams were observed, but bending compression failure and shear failure near the pile head were the major forms of damage in concrete piles.

Because there may have been no cases to date in which lives were lost as a result of serious damage to pile foundations and because design methods have not yet been established, structural calculations for foundations with respect to severe earthquake ground motion are not mandatory at present. However, it is important to confirm the state of the foundations of buildings with a tendency to fall, such as tower-like buildings, in terms of great earthquake ground motion.

To assess the aseismic performance of a building during a great earthquake, the ultimate strength and plastic deformation performance of each member should be grasped. In order to assess the deformation of each part during a great earthquake, static or dynamic analysis will be needed. The relationship between the horizontal load-carrying capacity and plastic deformation performance of the frame is often expressed as a structural characteristics factor (D_s) when assessing aseismic performance by static analysis. In stress analysis for great earthquake ground motion, ① the nonlinearity of the ground, ② the fixing ratio of the pile head joints, and ③ the nonlinearity of the piles should be considered.

With regard to item ①, Chang's equation(Equation 5.3.2.10), in which the ground is considered to be elastic, can not be applied as it is.

With regard to item ②, although only fixed pile heads are generally assumed, in actuality pile heads often fall between the fixed and pin types. Factors reducing the fixing ratio of a pile head joint may include rotation of the pile at the joint of the pile and pile cap, and rotation of the pile cap. A pile cap will rotate in association with deformation of the footing beam. A difference in vertical displacement between adjacent pile heads will cause tilting of the footing beam, which will be accompanied by rotation of the pile cap. Therefore, a decrease in the stiffness of the footing beam and in the vertical stiffness of a pile will cause a decrease in the fixing ratio of the pile head joint.

And with regard to item ③, the stiffness of a pile becomes nonlinear in association with defects such as cracking or plasticization of the pile. The nonlinearity may be considered to be a decrease in the stiffness of the pile, or it may be substituted by a decrease in the stiffness of the rotation spring of the pile head and the pile treated as elastic.

Elastoplastic incremental analysis using elastoplastic spring, representing nonlinearity of the ground and pile, is commonly used in stress analysis of pile foundations. In structural calculations for a great earthquake, the horizontal load-carrying capacity and required horizontal load-carrying capacity of the pile foundation are appropriately established ensuring that the former exceeds the latter as the basic principle.

The horizontal load-carrying capacity of a pile foundation is often considered as story shear force when any pile falls into a two-hinge state or when a pile, pile cap, or footing beam is destroyed other than by flexural yielding. The two-hinge state represents a condition in which flexural yielding occurs in the pile head followed by flexural yielding of the underground part of the pile. Failure types other than flexural yielding may include shear failure, bending compressive destruction, axial reinforcement breaking, and local buckling for a pile; various shear failures and breaking of various reinforcing bars for a pile cap; and shear failure for a footing beam.

How the required horizontal load-carrying capacity of a pile foundation is established depends on the designer's theory, and there is a concept whereby the story shear force of a pile head when the upper structure reaches the horizontal load-carrying capacity is considered to be the required horizontal load-carrying capacity of the pile foundation. This concept is quite clear-cut and is required to ensure the horizontal load-carrying capacity of the upper structure, but if the horizontal load-carrying capacity of the upper structure is very large as in wall construction, the required horizontal load-carrying capacity of the pile foundation will be much larger than the design horizontal force during a moderate earthquake. Some useful methods for the design of pile foundations for a severe earthquake are described in the literature.[7]

7. Seismic design of foundations and building standard laws in various countries

In developed countries, building standards are generally systematized for various combinations of loads including seismic load, while in many developing countries, seismic criteria such as a seismic design code are prescribed separately. Seismic criteria in each country are summarized in the literature,[8] and basic items such as base shear coefficients can be seen on the following Web site: http://iisee.kenken.go.jp.[9]

Table 7.1 shows examples of provisions for the seismic design of foundations in various countries. Compared with the seismic design of foundations in Japan, the following points should be noted.

① In assessing seismic load, the set depth by ground type is at the level of the tip of a pile in bearing piles in Japan, while in some countries (e.g., Canada) the depth is set at the level of the pile head.

② In Spain and India, seismic load is assessed based on each ground type as well as a combination of foundation types and the ground (seismic load generally becomes smaller in the order of individual footing, friction pile, and bearing pile).

③ In many countries, the footing beam is investigated with respect to axial compression and tension of about 1/10 the pile head load.

④ In the USA (IBC) and Puerto Rico, the foundation design overturning moment is reduced to 75%, while in Taiwan, it is reduced to 90%.

⑤ In many countries, the ratio of allowable unit stresses of the ground and a pile for long and short periods (bearing capacity) falls between 1 and 1.5.(1.33 in the USA (UBC), 2 in Japan, 1 to 1.5 depending on the ground and foundation in India), so the difference in design seismic force should be taken into consideration.

⑥ There are almost no technical criteria for the seismic force of underground structures other than in Japan and Mexico.

Table 7.2 shows examples of provisions for the seismic design of retaining walls. There are criteria according to the Monobe and Okabe method (earth pressure during an earthquake) and a horizontal seismic coefficient as well as those providing a vertical seismic coefficient.

Criteria for allowable unit stress of the ground for a short period that do not exist in Japan include seismic criteria in Bulgaria (1987) for the allowable condition of estrangement of foundations (edge unit stress up to 4 times the allowable unit stress for a long period at $e \geqq B/3$), and criteria in Costa Rica (1986) whereby the safety factor F is changed based on the ratio R of the minimum contact pressure and maximum contact pressure of the foundation bed (R <

0.25, F = 3.0 and R $\geqq$ 0.25, F = 2.5 at ordinary times; R $<$ 0.25, F = 2 and R $\geqq$ 0.25, F = 1.6 during an earthquake).

There are a number of provisions, similar to Enforcement Ordinance 93 in Japan, establishing values for each ground type for the allowable unit stress of the ground for a long period q_a. It should be noted that adequate investigation is required for organic soil and filled soil because, according to the USA (IBC) and India (National Building Code of India, 1983), estimation using tables is inappropriate.

Table 7.1 Examples of provisions for seismic design of foundations

Country, criteria, year	Examples of provisions
JAPAN BSLJ-2000	Calculation of allowable unit stress. There are provisions on underground seismic intensity. In design practice, a method has been established in which horizontal force is shared between the piles and underground exterior wall utilizing the embedment effect of the underground structure. Avoidance of deleterious settling, deformation, and others, taking liquefaction into account. The allowable bearing capacity of the ground during an earthquake is double that for a short period.
BULGARIA BCDBSSR-1987	Ultimate strength design. Safety factor, 1.2. The bearing capacity of the ground is investigated for each bidirectional moment.
CANADA NBCC-1995	A foundation is designed so that the upper structure will yield earlier. Except for Zone 0, independent piles and footings are connected in not less than two directions. Compressive and tensile connections are designed for the product of the maximum pile cap load and coefficient (not more than 10% of the maximum pile head load). Connections between the pile and pile head are fixed to exert yield strength due to the reinforcing effect.
CHINA NSPRCCSDB 1989	Structures including ① certain masonry structures, ② normal one-story factories or warehouses, ③ smokestacks with a height of 100 m or less in Zones III or IV, and ④ structures for which no seismic design investigation is performed for the upper structure have no need of seismic design of the foundation. In the case of pile foundations bearing only a low pile cap and vertical force with no possible liquefaction, no silt around the pile cap, and no filling with 100 kPa of *qa* for a long period, certain buildings have no need of seismic investigation of the pile foundations. The allowable unit stress of the ground for a short period $qs = qa$ for a long period $\times \delta$ (δ = 1.5 at $qa \geqq$ 300 kPa, δ = 1.0 for loose sand, mean unit stress $P \leqq qs$, maximum edge unit stress $P_{max} \leqq 1.2 \times qs$, alienation is 25% or less of the total).
COLOMBIA CCCSRD-1984	Foundation members such as footings, piles, and caisson are connected so that they can resist at least 25% of the total vertical load.
COSTA RICA SCCC-1986	Ultimate strength design or allowable unit stress design. Footing beam design considering compressive tension of 10% or more of the maximum footing load is performed for individual footings. Liquefaction may occur in ground with 50% or less than 2mm content of grain size and less than 30-40% relative density within 15 m. In stability calculation of a slope at a site, horizontal force is calculated as the product of 1.1 times the slip clod weight and the maximum acceleration.
EGYPT RERDBE-1988	There is no increase in the allowable unit stress of the ground and foundation members during an earthquake in soft clay and loose sand. Foundations are connected in two directions against compressive and tensile force equal to 10% of the vertical load.
EL SALVADOR SDERRES-1989	The footings of piles are connected against horizontal compressive and tensile forces equal to 25% of the axial force of the maximum footing. Liquefaction may occur in the ground with 50% or less of 2 mm or less content and 30-40% or less relative density.
FRANCE SCPS-1982	There are foundation factors in the evaluation of seismic load (1.0 for intermediate mat foundation, 1.15 for soft bearing pile, 1.3 for very soft bearing pile, and 1.3 for soft friction pile).
GREECE ARBC-1984	The allowable bearing capacity during an earthquake is 1.5 times that at ordinary times. The allowable unit stress of materials is increased by 20% (excluding shear). There are foundation factors in the evaluation of seismic load (1 for Categories A and B, 0.9 for Categories C and D with a constant foundation depth/mat foundation/pile with footing beam, 0.8 for those with a non-constant foundation depth).
INDIA CEERDS-2000	The safety factor during an earthquake is 1.5 or more. There are foundation factors in the evaluation of seismic load (1.0 for a bearing pile/mat foundation; 1, 1.2, and 1.5 for an individual footing without a footing beam on rock, intermediate ground, and soft ground, respectively; and 1.2 for a foundation with a footing beam on soft ground). Increased bearing capacity of piles and the ground during an earthquake (50% for bearing pile in solid ground, 25% for other piles, 50% for mat foundation, 25% for foundation with footing beam on soft ground, 0% for individual footings on soft ground).
INDONESIA IEC-1983	Foundations are connected in two directions. Members are designed considering compressive and tensile forces equal to 10% of each maximum vertical load. When the axial load of a connected column is not more than 20% of another load, the design axial load is 10% of the mean of both values. The safety factor for falling is 1.75 or more.
ISRAEL SIS-1995	The axial force of horizontal joints of a foundation is designed considering compressive and tensile forces, or not less than 10% of the maximum axial force of the columns.
ITALY ISC-1986	Foundations are connected to a grid beam. The footing beam is investigated using compression and tension, or 10% of the maximum vertical load at the end. This connection is not required when relative settling of the foundation is not more than 1/1000 and not more than 2 cm. The pile foundation resists the share of design horizontal force considering pile stiffness. In stability analysis of the soil and structure, the whole stress transmitted from the building to the foundations is considered.
MEXICO CTNERD-1995	The inertia force (C/4 $\times$ g; C = base shear coefficient) of foundations is considered. Serviceability limit and ultimate limit. There are detailed provisions on interaction.
NEPAL NBC105 SDBN-1994	Foundations are connected in two or more directions. Members are designed considering compressive and tensile forces, or 10% of the maximum vertical load. When one of the axial loads of a column connected to a footing beam is 20% or less of other loads, the design axial load is 10% of the mean of both values. The allowable bearing capacity during an earthquake is 1.5 times that at ordinary times.
NEW ZEALAND	Rocking of foundations requires special investigation with respect to energy extinction. (This is not applicable to a

NZS-1992 NZS 4203	ductility factor of 2 or less. Careful modeling of foundation stiffness and simulation of nonelastic response is required.) Rocking as dynamic energy extinction is applicable only to stiff and almost elastic upper structures. Investigation of the yield of foundations/ground and energy extinction for floating of spread foundations.
PERU ERSNRC-1977	Attention is to be paid to liquefaction and concentration in granular soil. In individual footings with piles and Zones I and II, connections require reinforcement of tensile force, with 10% or more of the vertical load supported by the footing. Piles require a footing beam (tensile reinforcement, 15% or more of supported load). In design of piles and footings, rotation and deformation due to horizontal force should be considered.
PHILIPPINES NSCP-1992	Appropriate reduction of bearing capacity is required for liquefaction and others. The allowable bearing capacity of the ground during an earthquake is 1.33 times that at ordinary times.
PUERTO RICO PRSD-1987	The overturning moment on foundations can be reduced to 75% (because a slight uplift at the end leads to reduction of the seismic force).
SPAIN SSC-1974/1991	There are foundation factors in the evaluation of seismic load (0.2 for mat foundation on very hard ground, 0.5 for slab foundation on intermediate ground, 0.7 for friction pile on intermediate ground, 2 for friction pile on ultrasoft ground, etc.).
TAIWAN TBC-1991	For discontinuous vertical bearing members, the overturning moment of the lowest layer is assessed as the axial force on the foundations. For continuous vertical bearing members, 10% reduction of the overturning moment is possible. Piles and caisson are connected to a footing beam and designed against axial compression and tension corresponding to 10% of the maximum pile load.
TURKEY EDR-1996	Ground with possible liquefaction (groundwater level within 10 m from the surface of the ground, 35% or less relative density, 10 or less N value, loose sand with Vs < 200 cm/sec). In Zones 1 and 2, oblique piles with a gradient of 1/6 or more should not be used.

Table 7.2 Examples of provisions for seismic design of retaining walls

Country, criteria, year	Examples of provisions
COSTA RICA SCCC-1986	The additional stress associated with earthquakes $P = 0.5\gamma h^2 * 3/4 \alpha_{max}$ (point of action, 0.6 h).
EL SALVADOR SDERRES-1989	Safety factors in the seismic design of a retaining wall are 1.2 for falling and 1.2 for slipping. The method of Mononobe and Okabe or the following equations are used. $P_{dh} = 3/8\gamma h^2 A_{hmax}$ (at 0.6 h from the base), $P_{vh} = 1/2\gamma h^2 Av_{max}$ (at 0.2 h from the bottom of the wall) A_{hmax}: 0.2 in Zone I and 0.1 in Zone II. Av_{max}: 0.1 in Zone I and 0.05 in Zone II.
INDIA CEERDS-2000	The vertical seismic coefficient in calculation of earth pressure during an earthquake is half the horizontal seismic coefficient. Point of action of active and passive earth pressures due to uniformly distributed load: static, 1/2 height; dynamic, 2/3 height.
USA California Hospital Code	Increment of earth pressure during an earthquake $Pd = 0.03Z\triangle h$ (h: height, ft; △: unit weight, lb/ft^3; Z: zone factor (1 for Zone 4)). The point of action of the increment of earth pressure is at 0.5 to 0.67H of wall height.

References

1) Guidelines on Seismic Design of Building Foundations, Ministry of Construction, supervised by the Building Guidance Division of the Housing Bureau, Building Center of Japan, 1984 (in Japanese).
2) Sugimura, Y: Japan foundation design guide, The Journal of CIB, Council for Building Research, Studies and Documentation, Building Research and Practice, Vol.16, No.2, pp.109-121, 1988.
3) Sugimura, Y: Earthquake Resistant Design of Building Foundations–introduction and commentary on "Design Guide for Building Foundations against Seismic Force," BRI Research Paper, No.113, Building Research Institute, 43pp, 1985.
4) Ikuta, Y, et al.: Record of lateral pressure taken during earthquake, Soils & Foundations, Vol.19, No.4, pp.85-92, 1979.
5) Chang, YL: Discussion on "Lateral Pile Loading Tests" by L.B. Feagin, A.S.C.E. Trans. Paper No.1959, pp.2l2-278, 1937.
6) Ohsaki, Y: Corrosion of steel piles driven in soil deposits, Research Report 81-04, Dept. of Architecture, University of Tokyo, 1981.
7) Recommendations for Design of Building Foundations, AIJ, 1988, 2001 (in Japanese).
8) Regulations for seismic design: A world list—1996, International Association for Earthquake Engineering, 1996.
9) Information network of earthquake disaster prevention technologies, IISEENET, BRI, http//iisee.kenken.go.jp, 2002.