6. SEISMIC DESIGN

6.1 Introduction

The extensive damage of bridges in the recent earthquakes, such as 1989 Loma Prieta, 1994 Northridge and 1995 Kobe (Hyogo-ken nanbu) earthquakes together with the research triggered as a consequence of the recent earthquakes have led to the significant advance in bridge seismic design and retrofitting. Near field ground motions developed in the Northridge earthquake and Kobe earthquake were included in the 1996 Japanese and 1999 Caltrans design codes, respectively. The traditional seismic coefficient method was replaced with the ductility design method, and linear/nonlinear dynamic response analysis is used on routine basis in design of bridges with complex structural response. A new treatment of liquefaction and liquefaction-induced lateral ground movement was included in the Japanese Specifications. Ductility evaluation of reinforced concrete/steel single/frame columns has been extensively conducted in various regions. The displacement-based design was included in the 1999 Caltrans code.

This chapter describes the effect of recent earthquakes on the development of new design practice in those countries, with placing emphasis on the Japanese experience in the aftermath of the 1995 Kobe earthquake.

6.2 Practice of Seismic Design in Japan

1) Past history of seismic design

The first seismic provisions for bridges were formulated in 1926 after experiencing the destructive damage in the 1923 Kanto Earthquake. Since the first stipulations, the seismic regulations have been reviewed and amended many times. The Design Specifications of Steel Highway Bridges were first issued in 1939 and were revised in 1956 and 1964. The seismic design related requirements were extremely limited in those days due to lack of scientific knowledge to earthquake engineering. Only the seismic lateral force of 20% of the gravity force was included, and no other seismic design related provisions were presented in these Specifications. The 20% gravity force was used as a basic design force for long time.

The first comprehensive seismic design provisions were issued in 1971 as the "Guide Specifications for Seismic Design of Highway Bridges." The 1964 Niigata Earthquake triggered development of the Guide Specifications. It was described in the Guide Specifications that the lateral force shall be determined depending on zone, importance and ground condition in the static lateral force method (seismic coefficient method) and structural response shall be further considered in the modified static lateral force method (modified seismic coefficient method). Evaluation of soil liquefaction was incorporated in view of the damage caused by the 1964 Niigata Earthquake. Design detailings to increase the seismic safety such as the devices for preventing unseating of a superstructure from substructures were introduced. Several independent design methods that had been developed for substructures were first unified between 1964 and 1971 in the form of "Guide Specifications of Substructures." Consequently, seismic safety was considerably upgraded in the bridges

designed after 1971. The year of 1971 was important in not only US but also Japan from the upgrading point of view for seismic design of highway bridges.

The 1971 Guide Specifications of Substructures and the 1971 Guide Specifications for Seismic Design were revised in 1980 in the form of “Part IV Substructures” and "Part V Seismic Design" of "Design Specifications of Highway Bridges". The Part V was essentially the same with the 1971 Guide Specifications for Seismic Design, but an updated evaluation method for predicting soil liquefaction as well as a practical design method for foundations in liquefying sands was included in Part V.

The Design Specifications were revised in 1990. Various major revisions were included in the Part V Seismic Design reflecting the progress of bridge earthquake engineering. The first was a unification of the static lateral force method (seismic coefficient method) and the modified static lateral force method (modified seismic coefficient method). This included the revision of the lateral force coefficient. The second was an introduction of check of strength and ductility capacities of reinforced concrete piers. Depending on the failure mechanism, strength and ductility capacities of reinforced concrete piers were evaluated. This was the first requirement in Japan to check the nonlinear behavior of bridges after yielding of structural members. Although this provision has not been mandatory, this significantly contributed to the enhancement of the ductility of piers. The third was an introduction of the static frame method to evaluate the lateral force of multi-span continuous bridges. This enabled to consider three-dimensional behavior of a bridge in the equivalent static analysis. The fourth was the provisions of design response spectra for dynamic response analysis.

Because of the unsatisfactory performance of highway bridges in the 1995 Kobe earthquake, the Design Specifications were extensively revised in 1996. Main features of the seismic design related to the seismic performance goals, design force, design of reinforced concrete columns and foundations, and treatment of liquefaction and liquefaction-induced ground movement are introduced in the following.

2) Seismic performance goals

It is essential to have a clear vision on the performance goals in seismic design. The seismic performance goals are determined depending on the importance and type of ground motions as shown in Table 6.1.

In the 1995 Kobe earthquake, even engineers in the organizations with jurisdiction to the construction and maintenance of bridges suffered damage to their family and properties. Due to the lack of information, it was almost days after the earthquake when the approximate damage situation was identified based on the inspection from outside the structures. If damage occurred at only a bridge, temporary shoring of the structure was possible in a couple of days after the earthquake. However, since extensive damage occurred in a wide area, it was unable to conduct the temporary shoring for a number of bridges shortly after the earthquake. Stock of the structural members for replacement, such as bearings, expansion joints and cable restrainers, were not available. It took long time to fabricate them after order. For example, structural steel plates were not available for reconstruction of steel girders; hence the structural steel plates that were ordered prior to the earthquake by other organization for construction of steel bridges at other locations were used for the reconstruction of steel girders in the damaged area. Based on such experience, it was obvious that bridges in important routes should not suffer damage to the extent that requires repairing immediately after an earthquake. If this occurred, repair within a month or even months of the earthquake is difficult. Prior to the Kobe earthquake, there was an augment whether bridges should have higher seismic performance than buildings or not. It is now obvious that bridges in important routes should have higher seismic performance than buildings, because restoration due to collapse of buildings cannot be conducted if the bridges having the same seismic performance

with the buildings suffer damage.

Table 6.1 Seismic Performance Goals (JRA (1996))

Ground Motions		Ordinary Bridges	Important Bridges
GM with high probability of occurrence		Functional	Functional
GM with low probability of occurrence	Type I GM	Prevent critical damage	Retain limited damage
	Type II GM		

3) Design ground motions

To achieve the seismic performance goals, three ground motions are used in seismic design as

Function-Evaluation

$$S_F = k_Z \cdot k_D \cdot \begin{cases} S_1 \cdot T^{1/3} & (0 < T \le T_1) \\ S_2 & (T_1 \le T \le T_2) \\ S_3 / T & (T_2 \le T) \end{cases} \tag{6.1}$$

Safety-Evaluation

$$S_{SI} = k_Z \cdot k_D \cdot \begin{cases} S_4 T^{1/3} & (0 < T \le T_3) \\ S_5 & (T_3 \le T \le T_4) \\ S_6 / T & (T_4 \le T) \end{cases} \quad \text{(Type-I ground motion)} \tag{6.2}$$

$$S_{SII} = k_Z \cdot k_d \cdot \begin{cases} S_7 \cdot T^{2/3} & (0 < T \le T_5) \\ S_8 & (T_5 < T \le T_6) \\ S_9 / T^{5/3} & (T_6 < T) \end{cases} \quad \text{(Type II ground motion)} \tag{6.3}$$

in which S_i (i=1-6) and T_i (i=1-4) are shown in Table 6.2, k_Z is the zone factor (1.0, 0.85 and 0.7 depending on regions), and k_D is the damping modification factor , and it is provided based on Eq. (1.23) as

$$k_D = \frac{1.5}{40\xi + 1} + 0.5 \tag{6.4}$$

Fig. 6.1 shows the response acceleration S_F, S_{SI} and S_{SII} by Eqs. (6.1), (6.2) and (6.3), respectively. The response spectrum for function evaluation S_F by Eq. (6.1) has been used in the seismic coefficient method. The Type-I and Type-II ground motions for the safety evaluation, S_{SI} and S_{SII}, by Eqs. (6.2) and (6.3) represent the ground motions induced in Tokyo in the 1923 Kanto earthquake and the near-field ground motions induced in the 1995 Kobe earthquake, respectively. Eq. (6.2) was derived based on the attenuation equation by Eq. (1.29) for a combination of M=8 and Δ=50 km. On the other hand, Eq. (6.3) was evaluated based on the response accelerations measured during the 1995 Kobe earthquake. The response acceleration S_{SII} by Eq. (6.3) was determined based on the envelops of the measured near-field ground motions as shown in Fig. 6.2.

An important feature of S_{SI} by Eq. (6.2) is the “capping” of the response accelerations

at short natural periods. The capping was often used in the evaluation of design response spectra at short natural periods because it was difficult to consider high response accelerations in design computation. The high response accelerations sometimes result in unrealistic sections if nonlinear dynamic behavior of bridges such as radiational energy dissipation and pounding between structural elements is realistically included in analysis. The evidence that the seismic damage of bridges with short natural periods was seldom in spite of the high demand in analysis validated the capping of response accelerations at short natural periods.

Another feature of Eqs. (6.1), (6.2) and (6.3) is that the importance of bridges in not reflected in the evaluation of the design response accelerations. It was often assumed that the design response acceleration depends on the importance of bridges. However intensities of ground motions does not depend on whether the bridges are important or not. Since the damage levels of bridges under the design ground motions should be determined so that the seismic performance goals are achieved, the importance is counted in the evaluation of design displacement ductility factor of columns. This will be described later.

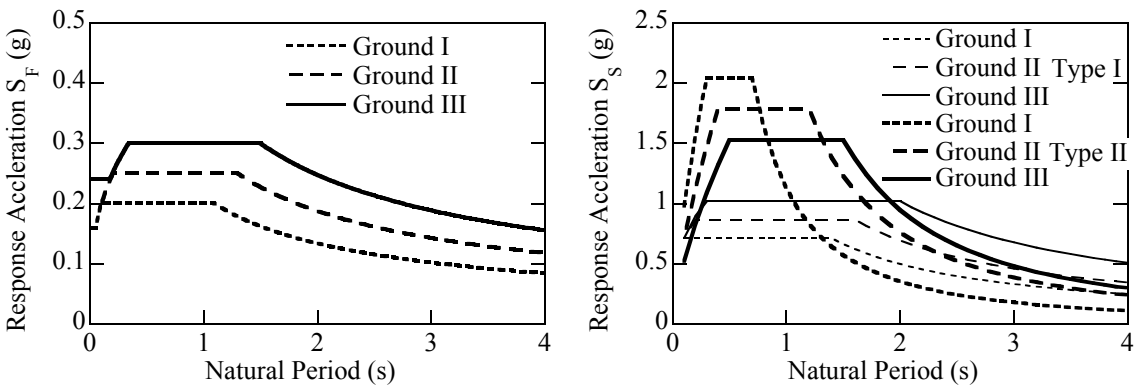
Table 6.2 Parameters in Eqs. (6.1), (6.2) and (6.3)

(a) Function-Evaluation Ground Motion

Soil Group	T_1 (s)	T_2 (s)	S_1 (m/s^2)	S_2 (m/s^2)	S_3 (m/s^2)
I	0.1	1.1	4.31	2.0	2.2
II	0.2	1.3	4.27	2.5	3.25
III	0.34	1.5	4.3	3.0	4.5

(b) Safety-Evaluation Ground Motions

Soil Group	Type I Ground Motion					Type II Ground Motion				
	T_3 (s)	T_4 (s)	S_4 (m/s^2)	S_5 (m/s^2)	S_6 (m/s^2)	T_5 (s)	T_6 (s)	S_7 (m/s^2)	S_8 (m/s^2)	S_9 (m/s^2)
I	0	1.4	-	7.0	9.8	0.3	0.7	44.63	20.0	11.04
II	0.18	1.6	15.05	8.5	13.6	0.4	1.2	32.24	17.5	23.71
III	0.29	2.0	15.11	10.0	20.0	0.5	1.5	23.81	15.0	29.48



(a) Function Evaluation (b) Safety Evaluation

Fig. 6.1 Response Accelerations by Eqs. (6.1), (6.2) and (6.3)

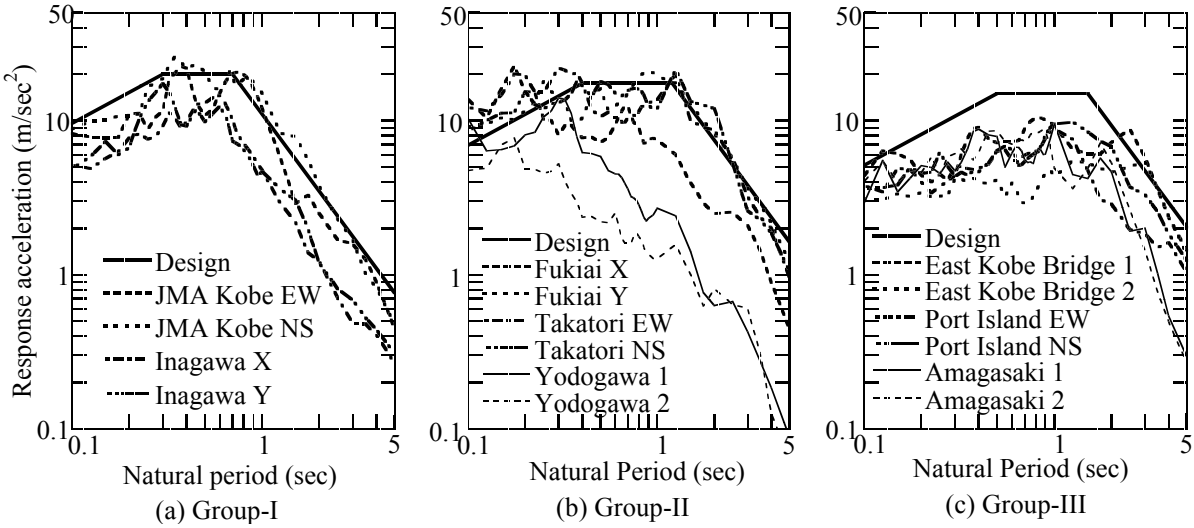


Fig. 6.2 Safety Evaluation Response Accelerations S_{SII} Determined from the 5% Damped Response Accelerations of the Record in the 1995 Kobe Earthquake

In addition to the response accelerations with 5% damping ratio, elastic lateral force for the equivalent static analysis that takes account of the modification of damping ratio of bridges is provide. The damping ratios of bridges for the fundamental modes depend on various factors. However based on forced loading tests, there exists an empirical relation between the small-amplitude fundamental natural period of a bridge T and the fundamental damping ratio ξ of a bridge as [Kuribayashi and Iwasaki, 1972]

$$\xi = 0.02/T \tag{6.5}$$

Fig. 6.3 shows a relation between the measured fundamental natural periods and the damping ratios of the fundamental modes. Because a damping ratio of a bridge generally increases as the oscillation amplitude increases, the actual damping ratio of the bridge may be slightly larger than the value estimated by Eq. (6.5).

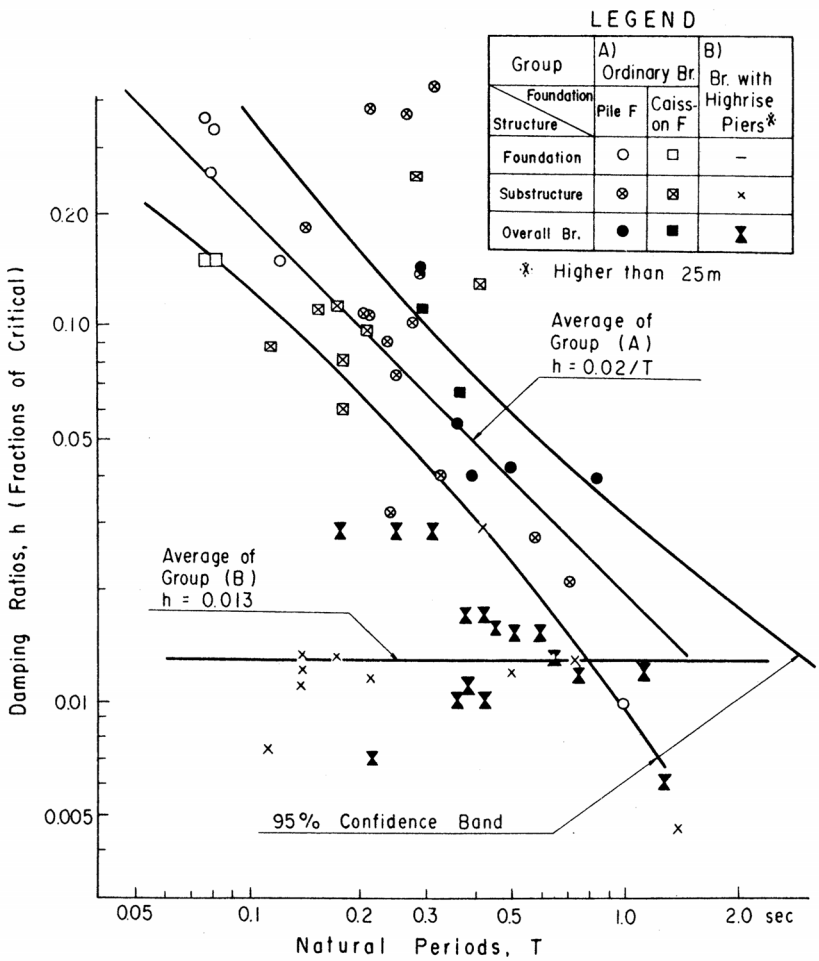


Fig. 6.3 Natural Period Dependence of the Damping Rations

Substituting Eq. (6.5) into Eq. (6.4), one obtains the elastic response accelerations for the equivalent static analysis S_s as

$$S_S(T) = S(T, \xi = 0.02/T) = c_D(\xi = 0.02/T) \cdot S(T, 0.05) \tag{6.6}$$

in which $S(T, 0.05)$ represents the response acceleration with 5% damping ratio. Substituting Eqs. (6.1), (6.2) and (6.3) into $S(T, 0.05)$ in Eq. (6.6), one obtains the following response acceleration for the equivalent static analysis after some modifications

<u>Functional -evaluation</u>

$$S_{esF} = k_Z \cdot \begin{cases} S_{s1} \cdot T^{1/3} & 0 < T \le T_1 \\ S_{s2} & T_1 \le T \le T_2 \\ S_{s3} / T^{2/3} & T_2 \le T \end{cases} \tag{6.7}$$

Safety-evaluation

$$S_{esSI} = k_Z \cdot \begin{cases} S_{s4} T^{1/3} & 0 < T \le T_3 \\ S_{s5} & T_3 \le T \le T_4 \\ S_{s6} / T^{2/3} & T_4 \le T \end{cases} \quad \text{(Type-I ground motion)} \qquad (6.8)$$

$$S_{esSII} = k_Z \cdot \begin{cases} S_{s7} T^{2/3} & 0 < T \le T_5 \\ S_{s8} & T_5 \le T \le T_6 \\ S_{s9} / T^{4/3} & T_6 \le T \end{cases} \quad \text{(Type-II ground motion)} \qquad (6.9)$$

in which $S_{si}(i = 1-9)$ = spectral acceleration (m/s), and are given in Table 6.3. The natural periods $T_i(i = 1-6)$ are the same with those in Eqs. (6.1), (6.2) and (3). However, the gradients at the constant velocity range in Eqs. (6.7), (6.8) and (6.9) are smaller than those in Eqs. (6.1), (6.2) and (6.3) by a factor of $T^{1/3}$ as a consequence of smaller damping ratio at the constant response velocity range.

Fig. 6.4 shows the response accelerations S_{esF}, S_{esSI} and S_{esSII} by Eqs. (6.7), (6.8) and (6.9), respectively.

Table 6.3 Parameters in Eqs. (6.7), (6.8) and (6.9)

Soil Group	Function-Evaluation			Safety-Evaluation					
				Type-I Ground Motion			Type-II Ground Motion		
	S_{s1}(m/s^2)	S_{s2}(m/s^2)	S_{s3}(m/s^2)	S_{s4}(m/s^2)	S_{s5}(m/s^2)	S_{s6}(m/s^2)	S_{s7}(m/s^2)	S_{s8}(m/s^2)	S_{s9}(m/s^2)
I	4.22	1.96	2.09	-	6.86	8.58	43.7	19.6	12.15
II	4.18	2.45	2.92	14.8	8.33	11.37	31.8	17.15	21.85
III	4.21	2.94	3.85	14.8	9.8	15.58	23.3	14.7	25.2

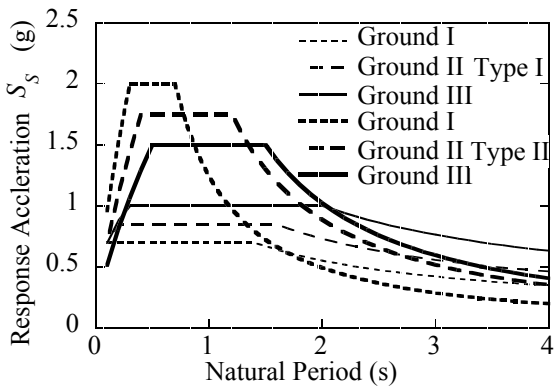


Fig. 6.4 Response Accelerations for the Equivalent Static Analysis by Eqs. (6.7), (6.8) and (6.9)

4) Design of a Bridge System

5) Design of a Reinforced Concrete Colunmn

a) Evaluation of Response Modification Factors

A bridge is designed assuming a principal plastic hinge at the bottom of piers so that the following requirement is satisfied.

$$P_a \geq \frac{S_{es} \cdot W}{R} \tag{6.10}$$

in which P_a =lateral capacity of a pier, S_{es} = response accelerations for the equivalent static analysis by Eqs. (6.8) and (6.9), W =tributary weight, and R = response modification factor. Assuming the equal energy principle, the response modification factor may be written from Eq. (1.44) as

$$R = \sqrt{2\mu_r - 1} \tag{6.11}$$

in which, μ_r is the response displacement ductility factor of a column.

Since response displacement ductility factor is not known at the first stage of design, an iteration is required if one uses Eq. (6.11). Consequently the response modification factor in Eq. (6.10) is generally estimated in design as

$$R = \sqrt{2\mu_a - 1} \tag{6.12}$$

in which μ_a is the design (allowable) displacement ductility factor of the pier. Eq. (6.12) is validated if one designs the bridge so that the response displacement ductility factor is close to the design displacement ductility factor. However if the response displacement ductility factor is sufficiently smaller than the design displacement ductility factor, Eq. (6.12) overestimates the response modification factor. The maximum value of the design displacement ductility factor μ_a of a single reinforced concrete column must be 8. Consequently, the largest response modification factor based on Eq. (6.12) is 3.8.

b) Evaluation of Strength and Ductility Capacity

Strength and design displacement ductility factor μ_a are determined depending on the failure mode as shown in Fig. 6.5. The flexural strength is evaluated from the fiber element analysis (refer to Chapter 4). An elastic-perfect plastic model generally idealizes the stress vs. strain relation of reinforcing bars, and the stress vs. strain relation of confined concrete is provided from Eq. (4.?) as [Hoshikuma et al. 1997]

$$f_c = \begin{cases} E_c \varepsilon_c \left\{ 1 - \frac{1}{n} \left(\frac{\varepsilon_c}{\varepsilon_{cc}} \right)^{n-1} \right\} & (0 \leq \varepsilon_c \leq \varepsilon_{cc}) \\ f_{cc} - E_{des}(\varepsilon_c - \varepsilon_{cc}) & (\varepsilon_{cc} \leq \varepsilon_c \leq \varepsilon_{cu}) \end{cases} \tag{6.13}$$

where,

$$n = E_c \varepsilon_{cc} / (E_c \varepsilon_{cc} - f_{cc}) \tag{6.14}$$

in which f_{cc} and ε_{cc} =strength of confined concrete and strain corresponding to f_{cc}, E_c = elastic modulus of concrete, and E_{des} = gradient at descending branch. In Eq. (7), f_{cc}, ε_{cc} and E_{des} are provided as

$$f_{cc} = f_{c0} + 3.8\alpha\rho_s f_{sy} \tag{6.15}$$

$$\varepsilon_{cc} = 0.002 + 0.033\beta \frac{\rho_s f_{sy}}{f_{c0}} \tag{6.16}$$

$$E_{des} = 11.2 \frac{f_{c0}^{\ 2}}{\rho_s \cdot f_{sy}} \tag{6.17}$$

in which f_{c0} = design strength of concrete, f_{sy} = yield strength of reinforcements, α and β = shape factors (α =1.0 and β =1.0 for circular piers, and α =0.2 and β =0.4 for rectangular piers), and ρ_s = volumetric ratio of tie reinforcements.

The ultimate displacement d_u is defined as the displacement at the gravity center of superstructure when concrete compression strain at the out-most reinforcements reaches the following ultimate strain ε_{cu};

$$\varepsilon_{cu} = \begin{cases} \varepsilon_{cc} & \text{(Type - I ground motion)} \\ \varepsilon_{cc} + 0.2 f_{cc} / E_{des} & \text{(Type - II ground motion)} \end{cases} \tag{6.18}$$

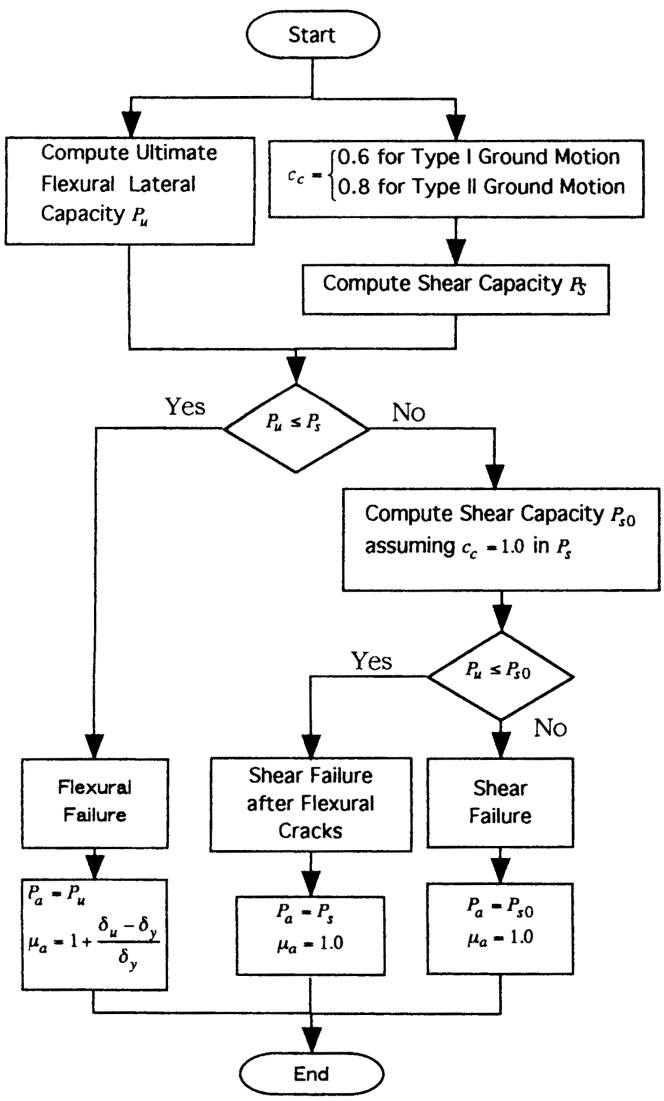


Fig. 6.5 Flowchart of Seismic Design

It is noted in Eq. (6.18) that the ultimate concrete strain depends on the type of ground motions. The deterioration of reinforced concrete columns at the plastic hinge region is larger under the cyclic loading with larger number of inelastic excursions (Type-I ground motion) as compared to the cyclic loading with smaller number of inelastic excursions (Type-II ground motion) [for example, Takemura and Kawashima, 1997]. Taking account of the ground motion-dependent deterioration of columns, the ultimate concrete compression strain is ε_{cc} under the Type-I ground motion, while it is the strain corresponding to the stress 20% deteriorated from the maximum stress f_{cc}. Consequently, the ultimate displacement of a column is larger under the Type-II loading than the Type-I loading.

The ultimate displacement d_u of a pier is evaluated from yielding and ultimate curvatures ϕ_y and ϕ_u as [Priestley and Park 1987, Priestley at al. 1996]

$$d_u = d_y + (\phi_u - \phi_y)(h - L_p / 2)L_p \tag{6.19}$$

in which h=height of the pier and L_p=plastic hinge length

$$L_p = 0.2h - 0.1D\,;\;\; 0.1D \le L_p \le 0.5D) \tag{6.20}$$

On the other hand, shear strength of a reinforced concrete pier P_s is evaluated as (refer to Eq. (4.?)

$$P_s = V_c + V_s \tag{6.21}$$

where

$$V_c = k_c \cdot k_e \cdot k_{pt} \cdot v_c \cdot b \cdot h \tag{6.22}$$

$$V_s = A_w \cdot f_{sy} \cdot d \cdot (\sin\theta + \cos\theta)/(1.15 \cdot a) \tag{6.23}$$

in which V_c and V_s=shear strength of concrete and transverse reinforcement, v_c=shear strength of concrete (e.g., 0.35 MPa and 0.37 MPA for concrete with design strength of 24 MPa and 30 MPa, respectively), k_c=cyclic loading effect factor , k_e=effective height factor (e.g., 1.0, 0.7, 0.6 and 0.5 for effective height of < 1m, 3m, 5m and > 10m, respectively), k_{pt}=modification factor depending on tension bars ratio p_t (e.g., 0.9, 1.0, 1.2 and 1.5 for tension reinforcement ratio of 0.2%, 0.3%, 0.5% and > 1.0%), b and h=effective width and height, and A_w=area of reinforcing bars with an interval a and an angle θ. The factor k_c in Eq. (6.22) represents the deterioration of shear strength under cyclic loading. It is 1.0, 0.6 and 0.8 under a static loading, Type-I ground motion and Type-II ground motion, respectively. Shear strength of concrete P_s under a static loading ($k_c = 1.0$) is denoted P_{s0}.

Based on the flexural strength P_{fu}, shear strength P_s and shear strength under static loading P_{s0}, failure mode of a pier is decided to be one of the flexural failure, shear failure after flexural damage and shear failure as

$$\text{Failure Mode} = \begin{cases} \text{flexural failure} \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots P_u \le P_s \\ \text{shear failure after flexural damage} \cdots\cdots P_s \le P_u \le P_{s0} \\ \text{shear failure} \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots P_{s0} \le P_u \end{cases} \tag{6.24}$$

, and the lateral capacity P_a and the design displacement ductility factor μ_a in Eq. (6.12) are given as

$$P_a = \begin{cases} P_u & \cdots\cdots \text{ flexural failure + shear failure after flexural damage} \\ P_{s0} & \cdots\cdots \text{shear failure} \end{cases} \tag{6.25}$$

$$\mu_a = \begin{cases} 1 + \dfrac{d_u - d_y}{\alpha \cdot d_y} \cdots\cdots \text{flexural failure} \\ 1 \cdots\cdots\cdots\cdots\cdots\cdots \text{shear failure after flexural damage + shear failure} \end{cases} \tag{6.26}$$

in which α =safety factor depending on importance of bridges and the type of ground motion (α =3.0 and 2.4 for important and ordinary bridges, respectively, under the Type-I ground motions, and α =1.5 and 1.2 for important and ordinary bridges, respectively, under the Type-II ground motions), d_y and d_u =yielding and ultimate displacement of the pier. Larger α -value is assigned for bridges with higher importance under cyclic loading with long duration (Type-I ground motion). Thus, in addition to the determination of ultimate displacement d_u, the ground motion characteristics are also accounted in the evaluation of the allowable displacement ductility factor of a pier μ_a.

c) Residual Displacement

A number of reinforced concrete piers that suffered flexural failure at their based did not collapse, but tilted and were left with large residual displacement at the top. About 100 piers with a tilt angle of more than 1 degree (1.75% drift) were demolished and new piers were built due to the difficulty of setting the superstructures back to the original alignments and levels. Residual displacements in piers have been considered to be secondary importance to maximum ductility demand in seismic design of structures. Residual displacement should be considered independently from maximum ductility because a wide range of residual displacement may occur for the same ductility demand. Hence, a requirement of residual displacement d_R developed at a pier after an earthquake was at the first time included in the 1995 Specifications as

$$d_R \le d_{Ra} \tag{6.27}$$

where

$$d_R = c_R(\mu_r - 1)(1 - r)d_y \tag{6.28}$$

$$\mu_r = 1/2 \cdot \left[\{S_s / (g \cdot P_a)\}^2 + 1\right] \tag{6.29}$$

in which d_{Ra} =allowable residual displacement, r =bilinear factor defined as a ratio between the first stiffness (yield stiffness) and the second stiffness (post-yield stiffness) of the pier, c_R =factor depending on the bilinear factor r, μ_r =response displacement ductility factor of the pier, and d_y =yielding displacement of the pier. c_R was set 0.5 for reinforced concrete piers based on the residual displacement response spectrum by Eq. (1.57). The d_{Ra} is 1% of the distance between bottom of the pier and the gravity center of the superstructure (1% drift).

d) Design Detailings

Design detailings are important in seismic design. For example, the size of tie bars has to be 13 mm or larger in diameter, and they are provided in each 150 mm or shorter. In tall piers (> 30 m), it can be extended to 300 mm at the region outside 4 times the plastic hinge length, but the space should not be suddenly changed. Crossties with the same size with longitudinal bars are required at 150 mm space or shorter (same spacing with the tie bars). They are placed at distance of no larger than 1 m in the horizontal direction.

e) Comparison of Pre-Kobe and Post-Kobe Codes

It is interesting to evaluate how much changes were induced as a consequence of the revisions of codes. A few bridges were designed according to the past and current codes, assuming that only pier size and reinforcements were changed by keeping the superstructure, pier height and soil condition the same. Fig. 6.6 compares the section of an 11 m high reinforced concrete pier that supports a part (4.97 MN weight) of 3-span continuous steel girder bridge with 3@40 m long. Ground condition was Group-II (moderate). The pier designed according to the 1964 Specifications, which were used in design of most bridges that suffered damage in the Kobe earthquake including the collapsed 18-span bridge, is 2.4 m wide (transverse direction) and 1.9 m high (longitudinal direction). It is reinforced by 88 deformed bar with 29 mm diameter (D29) in double (reinforcement ratio=1.24%), and D13 ties are provided in each 300 mm and 600 mm for the outside and inside bars, respectively. Thus the volumetric ratio of ties ρ_s is only 0.08% and 0.1% in longitudinal and transverse directions, respectively. On the other hand, the pier design according to the 1996 Design Specifications is 2.9 m wide and 2.2 m high. It is reinforced by 116 D38 bards in double (reinforcement ratio=2.07%) and D22 ties are provided in each 150 mm for both outside and inside bars. The tie reinforcement ratio ρ_s is thus 1.15% and 1.08% in longitudinal and transverse directions, respectively.

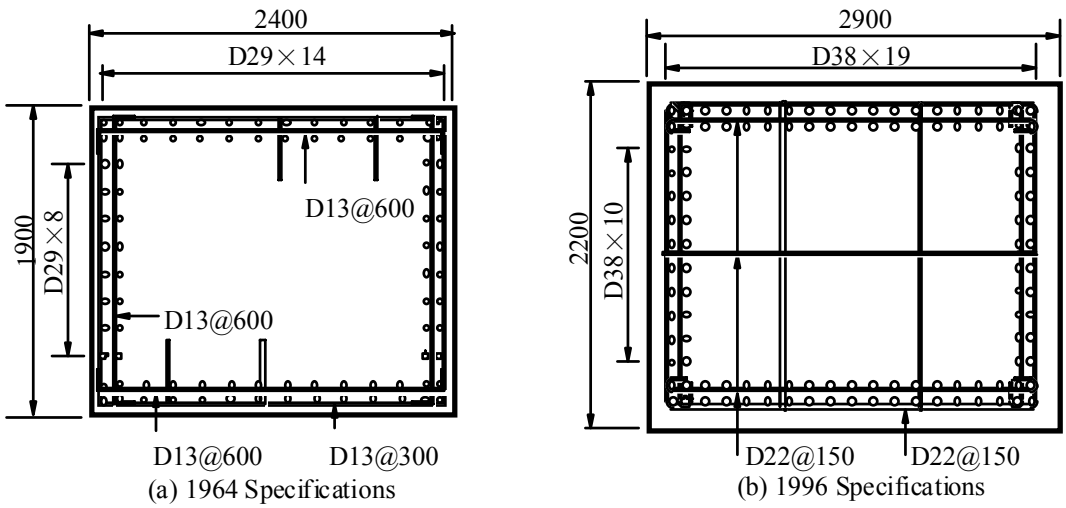


Fig. 6.6 Comparison of Column Sections Designed According to the 1964 and 1996 Design Specifications

If the pier designed according to the 1964 design specifications is evaluated according to the current design specifications, it fails in shear based on Eq. (6.24), because the shear strength by Eq. (6.21) is 1.78 MN while the flexural strength is 1.87 MN in longitudinal direction. On the other hand, the shear and flexural strengths of the pier designed according to the current Specifications are 11.33 MN and 4.32 MN, respectively in longitudinal direction. Hence, it fails in flexure. Since the ultimate displacement ductility factor $\mu_u = d_u / d_y$ computed from the standard moment-curvature analysis under the Type-II ground motion (Type-II ground motion was predominant in this example) is 5.4, the design displacement ductility factor μ_a is 3.93 from Eq. (6.12). Natural period of the pier-superstructure system taking account of foundation flexibility is 0.73 second. Since the Type-II response acceleration for safety-evaluation at this natural period is 17.5 m/s^2 from Eq. (6.9), dividing this value by the seismic modification factor $R = \sqrt{2\mu_a - 1}$ =2.62 (refer to Eq. (6.12)), the

equivalent response acceleration S_{esSII}/R is 6.7 m/s^2. Since the tributary weight of this pier is 6.15 MN, the lateral force demand is 6.7$\times$6.15 MN/g=4.1 MN. Since the flexural strength of the pier (4.32 MN) is larger than the demand, this pier is evaluated to be safe in the longitudinal direction. Similar evaluation shows that the pier is evaluated also to be safe in the transverse direction. The residual displacement from Eq. (6.28) is 80 mm and 90 mm in the longitudinal and the transverse directions, respectively. They are smaller than the 1% drift (110 mm), and satisfy Eq. (6.27).

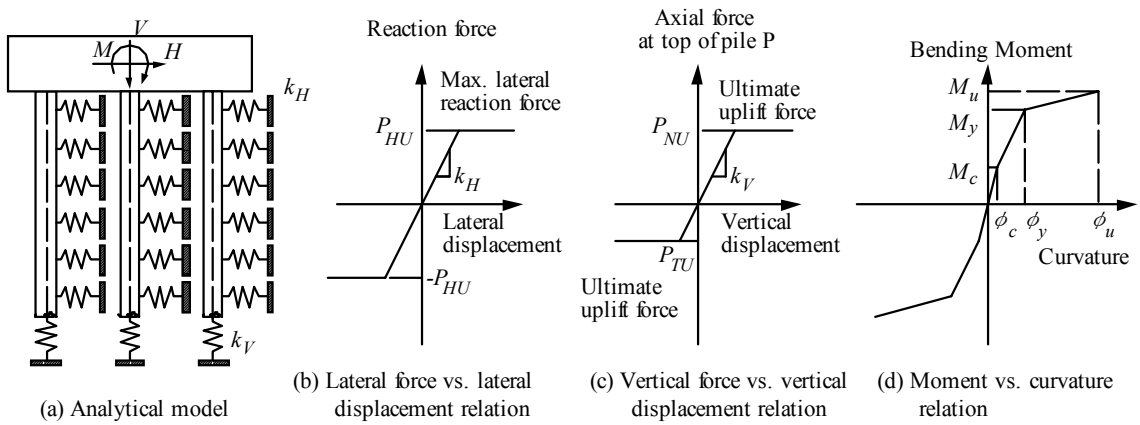
6) Design of a Foundation

Pushover analysis is used in design of foundations under the safety-evaluation ground motions. In the pushover analysis, a foundation is modeled as a structure supported by ground. For example, in a pile foundation, the piles and footing are idealized to be supported by nonlinear soil springs as shown in Fig. 6.7. As well as the plastic deformation of piles, the nonlinear deformation of soils around the piles, and uplift and pull-down of piles are taken into account. Evaluation of spring stiffness and strength are presented in the Design Specifications [JRA 1996].

Design lateral force of a foundation P_F is evaluated from the lateral capacity of the pier P_a by Eq. (6.25) as

$$P_F = k_F P_a \tag{6.30}$$

in which k_F is the over-strength factor (=1.1). Under the design lateral force, the foundation is so designed that piles do not yield. The yielding of a foundation is defined depending on the type of foundations. In a pile foundation, the yielding is defined as the stage when either of the following occurs first;



(a) Analytical model (b) Lateral force vs. lateral displacement relation (c) Vertical force vs. vertical displacement relation (d) Moment vs. curvature relation

Fig. 6.7 Analytical Model for Design of Pile Foundation under the Safety-Evaluation Ground Motion

- All piles yield, or
- Compression force, which applies to the piles on a same line, reaches the strength of ground.

The above conditions result in a sudden deterioration of the lateral resistance of the pile

foundation. Although the upward force, which applies to piles, is also important for the lateral resistance, it is not included as the stage of the yield, because this does not cause critical deterioration due to the dead weight effect.

7) Liquefaction and Liquefaction-Induced Ground Movement

Extensive soil liquefaction occurred over a wide area in offshore-reclaimed lands and natural deposits in the Kobe earthquake. Liquefaction occurred at not only sandy layers but also gravel layers with the averaged grain size over 2mm. Gravel layers was considered free from liquefaction for long time in the past. Liquefaction resulted in the ground movement in horizontal direction near shorelines. It was found from aerial photographs that the liquefaction-induced ground movement reached as large as 3-4 m [Hamada et al. 1995]. Several bridges suffered extensive damage due to liquefaction-induced ground movement [MOC 1996, Tamura et al. 2000].

After experiencing the Kobe earthquake, the evaluation of liquefaction potential and its treatment were extensively revised. The F_L-method [Iwasaki 1985] was revised so that it is evaluated under the safety-evaluation ground motion. The type of ground motion dependency of the shear strength of soil was included. Soils which require the evaluation of liquefaction potential based on the F_L-method was extended to include gravel layers; saturated ground within 20 m from the ground surface and where water table is within 10 m from the ground surface; soil layers with fine content FC < 35%, or soils with FC > 35% and the plasticity index I_p<15; and soil layers with D_{50} < 10 mm and D_{10} < 1mm, in which D_{50} and D_{10} are the grain size corresponding to 50% and 10% on the accumulation grain size curve.

In the above soil layers, the liquefaction potential factor F_L is evaluated based on cyclic shear stress ratio R and cyclic shear strength ratio L as

$$F_L = R / L \qquad .31)$$

where

$$R = c_{GM} R_L \qquad (6.32)$$

$$L = r_d k_{hc} \sigma_v / \sigma_v' \qquad (6.33)$$

in which R_L=tri-axial strength ratio of soil, k_{GM}=type of ground motion factor, k_d = acceleration reduction factor in underground ($k_d = 1.0 - 0.015x$ where x is the depth (m)), a_g=acceleration at ground surface in the Type-I and Type-II ground motions, and σ_v and σ_v'=total and effective overburden pressure. a_g is 0.3g, 0.35g and 0.4g at the ground groups I, II and III, respectively, under the Type-I ground motion, and 0.8g, 0.7g and 0.6g at the ground groups I, II and III, respectively, under the Type-II ground motion. Ground motion type dependency of the shear strength of soils is accounted by k_{GM} as

$$k_{GM} = \begin{cases} 1.0 & & \text{(Type - I ground motion)} \\ \left.\begin{array}{ll} 1.0 & (R_L \le 0.1) \\ 3.3R_L + 0.67 & (0.1 < R_L \le 0.4) \\ 2.0 & (0.4 < R_L) \end{array}\right\} & & \text{(Type - II ground motion)} \end{cases} \qquad (6.34)$$

The tri-axial strength ratio R_L in Eq. (6.32) is represented as

$$R_L = \begin{cases} 0.0882\sqrt{N_a / 1.7} & (N_a < 14) \\ 0.0882\sqrt{N_a / 1.7} + 1.6\times10^{-6}\cdot(N_a - 14)^{4.5} & (14 \le N_a) \end{cases} \tag{6.35}$$

where

$$N_a = \begin{cases} c_1 \dfrac{1.7N}{\sigma_v' + 0.7} + c_2 & \text{(Sand)} \\ \{1 - 0.36\log_{10}(D_{50}/2)\}N_1 & \text{(Gravels)} \end{cases} \tag{6.36}$$

in which N, N_a and $N_1 = N$-value of standard penetration test, corrected N-value accounting for the effect of grain size, and N-value corresponding to overburden soil pressure of 0.1 MPa (1kgf/cm^2), and coefficients c_1 and c_2 are given as

$$c_1 = \begin{cases} 1 & (0 \le FC < 10\%) \\ (FC + 40)/50 & (10\% \le FC < 60\%) \\ FC/20 - 1 & (60\% \le FC) \end{cases} \tag{6.37}$$

$$c_2 = \begin{cases} 0 & (0\% \le FC < 10\%) \\ (FC - 10)/18 & (10\% \le FC) \end{cases} \tag{6.38}$$

The F_L-value is evaluated usually at each 1 m. The soil layers where F_L is less than 1.0 is evaluated to be liquefiable under the design ground motion. When it is estimated that liquefaction occurs from Eq. (6.31), both spring stiffness k_{SL} and bearing capacity of soils p_{SL} in liquefying layers are reduced from the original spring stiffness k_S and bearing capacity p_S as

$$k_{SL} = c_L k_S \tag{6.39}$$

$$p_{SL} = c_L p_S \tag{6.40}$$

in which c_L is soil parameter reduction factor associated with liquefaction, and is provided depending on F_L-value, depth and cyclic shear strength ratio R as shown in Table 6.4.

Table 6.4 Soil Parameter Reduction Factor Associated with Liquefaction c_L in Eqs. (6.39) and (6.40)

F_L-value	Depth x from Ground Surface (m)	Cyclic Shear Strength Ratio R	
		$R \le 0.3$	R>0.3
$F_L \le 1/3$	0≤x≤10	0	1/6
	10<x<20	1/3	1/3
1/3<F_L ≤2/3	0≤x≤10	1/3	2/3
	10<x<20	2/3	2/3
2/3<F_L ≤1	0≤x≤10	2/3	1.0
	10<x<20	1.0	1.0

Typical soil profile at the sites where bridges suffered damage due to the liquefaction-induced ground movement consisted of reclaimed top soil resting on loose sandy layers as shown in Fig. 6.8. Liquefaction occurred in the loose sandy layers, while the topsoil

did not liquefy. The topsoil however moved associated with the lateral movement of the loose sandy layers. It was often that footings were embedded in the topsoil. Hence, a large lateral force applied to the footings from in the topsoil. Although the mechanism of lateral ground movement is still under investigation, it was found in the Kobe earthquake that it was triggered by failure of revetment along the shoreline, and thus the amount of ground movement reduced as the distance from the failed revetment increased. It was also found that ground movements were larger when the sea bottom was deep from the ground surface.

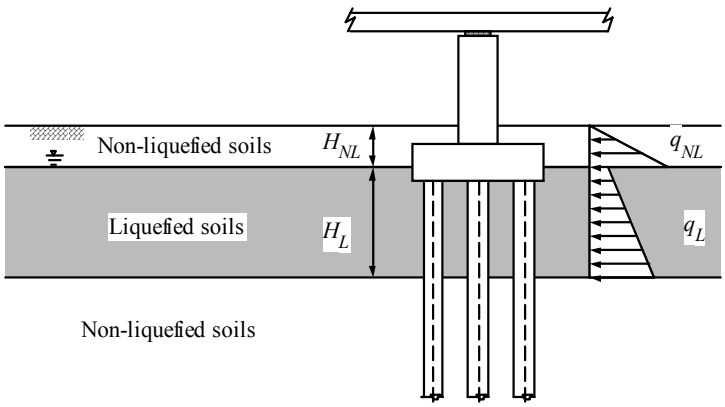


Fig. 6.8 Treatment of Liquefaction-Induced Ground Movement

When the ground movement occurs, force associated with the ground movement applies to a part of foundation in touch with the moving ground. To represent such force mechanism, it is appropriate to idealize the foundation to be a structure supported by soil springs, and prescribe the movement of ground at the end of each soil spring. In such an analytical mode, it is important how accurately amount of ground movement can be predicted. Since the evaluation of maximum ground movement was difficult, an analytical model in which the force associated with ground movement applies to the foundation was included in the 1996 Design Specifications [JRA 1996]. The foundation is idealized as a structure supported by soil springs. The reduction of soil springs based on Eq. (25), if any, is accounted in the model. Then distributing forces apply to a part of the foundation in non-liquefying ground (top soil) and liquefying ground as shown in Figure 7. The distributing forces which apply to a part of the foundation in non-liquefying and liquefying soils, q_{NL} (kN/m) and q_L (kN/m), respectively, are provided as

$$q_{NL} = c_s \cdot c_{NL} \cdot K_P \cdot \gamma_{NL} \cdot x \qquad (0 \le x \le H_{NL}) \qquad (6.41)$$

$$q_L = c_s \cdot c_L \cdot \{\gamma_{NL} \cdot H_{NL} + \gamma_L \cdot (x - H_{NL})\} \qquad (H_{NL} \le x \le H_{NL} + H_L) \qquad (6.42)$$

in which K_P=passive earth-pressure coefficient, γ_{NL} and γ_L=unit weight of non-liquefying and liquefying soils, H_{NL} and H_L=thickness of non-liquefying and liquefying soils, x=depth from the ground surface, c_s=modification factor depending on distance from shoreline, and c_{NL} and c_L=modification factor for ground movement-induced force in non-liquefying and liquefying soils, respectively. Based on several back-analyses of bridges that suffered damage from the ground movement in the Kobe earthquake and shaking table tests, the modification factor c_L was recommended to be 0.3 [MOC 1995, Tamura and Azuma 1997, Tamura et al. 2000]. Based on the empirical evaluation of the damage in the

Kobe earthquake and a series of shaking table tests, c_s=1.0, 0.5 and 0.0 corresponding to $s \leq$50m, 50<$s \leq$100m and 199m<s, respectively, where s is the distance from shoreline, and c_{NL} =0.0, $(0.2P_L - 1)/3$ and 1.0 corresponding to $P_L \leq 5$, 5< $P_L \leq 20$ and 20< P_L, respectively, where the liquefaction index P_L defined as

$$P_L = \int_0^{20} (1 - F_L)(10 - 0.5x)dx \tag{6.43}$$

6.3 Features of Recent Seismic Design Codes

1) Seismic Design Codes

As a consequence of the recent earthquakes including the 1989 Loma Prieta, 1994 Northridge and 1995 Kobe earthquakes, seismic design codes for highway bridges have been revised. As described in the previous sections, Design Specifications of Highway Bridges were fully revised in 1996 in Japan. In 1994, Part 2 Bridges in Euro code 8 Design provisions for earthquake resistance of structures was proposed as the European Pre-standard [Pinto 1995]. In New Zealand, the Transit New Zealand Bridge Manual was revised in 1995 [TNZ 1995, Chapman 1995]. In the United States, the American Association of Highway and Transportation Officials (AASHTO) published two codes for the design of highway bridges: Standard Specifications for Highway Bridges and LRFD Bridge Design Specifications. The Department of Transportation of the State of California (Caltrans) has developed independent seismic design specifications that are similar to, but not the same as, the AASHTO provisions. ATC-32 recommendation was published to improve Caltrans seismic design practice [ATC 1996]. Caltrans recently developed the Seismic Design Methodology [Caltrans 1999a] and the Seismic Design Criteria [Caltrans 1999b].

Recent progress in those seismic design codes for highway bridges is introduced in this section. Since different notations and acronyms are used in equations and tables in these codes, they are modified from the original definitions for simplicity.

2) Design Philosophy and Seismic Performance Criteria

In seismic design of structures, it is important to have a clear vision on the seismic performance. Decision on “how do you want the structure to perform in an earthquake ?”, “how much danger can you accept ?” and “what are the reasonable alternate routes ?” [Roberts 1999] is certainly important. The basic concept of design philosophy and seismic performance criteria is more or less similar among the codes, i.e., for small-to-moderate earthquakes bridges should be resisted within the elastic range of the structural components without significant damage, and bridges exposed to shaking from large earthquakes should not cause collapse [Buckle 1996].

In each code, the performance requirements depend on the importance of bridges. The importance is classified into 2-3 categories. In EC and New Zealand codes, design force is factored by the importance factor. In AASHTO and Caltrans, the importance is reflected in the evaluation of response modification factor. In Japan, the importance is reflected in the evaluation of design ductility factor of substructures, and this subsequently affects the evaluation of response modification factor.

The anticipated function of a bridge after a design earthquake depends on the codes. In New Zealand, the bridge must be usable by emergency traffic, although damage has occurred and some temporary repairs may be required. As shown in Table 6.1, in Japan the

performance criteria for important bridge are that the bridge should retain the "limited damage." It is presented in the commentary that the "limited damage" intends not to exceed the stage where the restoring force of piers initiates to deteriorate. Hence, the "limited damage" intends to be almost "immediately functional."

In the ATC-32 and the Caltrans code, the performance criteria is clearly stated as shown in Table 6.5 by classifying them into requirements in service level and damage level. In the service level, "immediate" implies full access to normal traffic almost immediately following the earthquake, and "limited" implies that limited access (reduced lanes, light emergency traffic) is possible within days of the earthquake, and that full service is restorable within months. In the damage level, "repairable damage" intends damage that can be repaired with a minimum risk of losing functionality. "Significant damage" intends a minimum risk of collapse, but that would require closure to repair, while "minimum damage" intends essentially elastic performance. "No collapse", "no major damage", "no secondary injuries or fatalities because emergency equipment cannot get through", "major important structures and lifeline routes must remain operational" are the current Caltrans performance criteria [Roberts 1999]. It is important that the important bridges should have full access to normal traffic immediately following the safety-evaluation earthquake.

Table 6.5 Seismic Performance Criteria of ATC32 and Caltrans (19999

Ground Motions	Service level		Damage level	
	Ordinary Bridges	Important Bridges	Ordinary Bridges	Important Bridges
Functional-Evaluation GM	Immediate	Immediate	Repairable Damage	Minimum Damage
Safety-Evaluation GM	Limited	Immediate	Significant Damage	Repairable Damage

3) Seismic Loads

All the codes listed above except New Zealand represent the design force by 5% damped elastic response spectra. In EC code, the elastic response acceleration S for the reference return period is defined depending on the local soil condition as

$$S = k_I \cdot k_{GC} \cdot a_g \cdot \begin{cases} 1 + \dfrac{T}{T_B}(k_D \beta_0 - 1) & 0 \le T \le T_B \\ k_D \cdot \beta_0 & T_B \le T \le T_C \\ k_D \cdot \beta_0 \cdot \left(\dfrac{T_C}{T}\right) & T_C \le T \le 3s \\ k_D \cdot \beta_0 \cdot \left(\dfrac{T_C}{3}\right) \cdot \left(\dfrac{3}{T}\right)^2 & 3s \le T \end{cases} \tag{6.44}$$

in which k_I=importance factor (1.0, 1.3 and 0.7 for average, greater than average and less than average importance bridges, respectively), k_{GC}=ground condition factor (1.0 for A+B, and 0.9 for C subsoil conditions), k_D=damping modification factor, β_0=spectral acceleration amplification factor (=2.5), a_g=design ground acceleration (g), and T_B and T_C=limits of the constant spectral acceleration branch (T_B=0.1, 0.15 and 0.2 seconds in subsoil class A, B and C, and T_C=0.4, 0.6 and 0.8 seconds for subsoil class A, B and C, respectively). The damping

modification factor k_D in Eq. (6.44) is given in terms of damping ratio ξ as

$$k_D = \sqrt{0.07/(0.02+\xi)} \geq 0.7 \tag{6.45}$$

The design seismic force for averaged-importance bridges (k_I=1.0) usually reflects a design seismic event with a return period of approximately 475 year. Such an event has a probability of exceedance ranging from 0.1 to 0.19 for a design lifetime ranging from 50 to 100 years, respectively.

In New Zealand, the horizontal seismic force for the equivalent static analysis is provided with the inelastic response spectra S_i as

$$S_i = k_Z \cdot k_I \cdot k_{GC} \cdot S_\mu(T) \tag{6.46}$$

in which k_Z = zone factor (0.6-1.2 depending on zones), k_I = importance factor (risk factor; 1.3, 1.15 and 1.0 for bridges with importance category of 1, 2 and 3, respectively), k_{GC} = ground condition factor (structural performance factor; 0.9, 0.8 and 0.67 for rocky or very stiff soil sites, intermediate soil sites, and flexible or deep soil sites, respectively), and $S_\mu(T)$ = basic acceleration coefficient multiplied by gravity acceleration g, and is give in figures. Factoring by k_I, the return period of design earthquakes is about 900 and 650 years for bridges with the important factor of 1.3 and 1.15, respectively.

In the AASHTO, the response acceleration spectrum S is given as

$$S = \frac{1.2 \cdot a_g \cdot k_{GC}}{T^{2/3}} \leq 2.5 a_g \tag{6.47}$$

in which a_g = ground acceleration (g), and k_{GC} = site modification factor (1.0, 1.2, 1.5 and 2.0 for ground condition I, II, III and IV, respectively). a_g is provided as contour for return period of 475 year. The highest value of a_g throughout the US is > 0.8 g in a part of Alaska and California. Furthermore, the maximum probable earthquake with a return period of around 2,500 years is considered.

On the other hand, in the ATC-32 and Caltrans [1999], the new response accelerations for the safety-evaluation have been used to represent the near-field ground motions. They are assessed either deterministically or probabilistically. The deterministic evaluation corresponds to the maximum credible earthquake, while the probabilistically assessed ground motion is one with approximately 1,000-2,000 year return period. The response acceleration S (ARS response curves) is provided in figures depending on peak ground acceleration a_g (0.1-0.7g), site condition (soil profile Type B-E), and earthquake magnitude (6.5$\pm$0.25, 7.25$\pm$0.25, and 8.0$\pm$0.25). The peak ground acceleration a_g is evaluated by CDMG based on a deterministic procedure for maximum credible earthquakes.

To take account of the near-field ground motion effect for bridges within 15 km from an active fault, Caltrans magnifies the response acceleration S by 20% for natural period $T \geq 1.0$ seconds, and a value determined by linear interpolation for $0.5 \leq T \leq 1.0$ seconds. The response acceleration is not magnified for T < 0.5 seconds. Furthermore, to take account of amplification in soft soil deposits, for bridges with natural period equal to or larger than 0.5 seconds constructed on deep soil sites (depth of alluvium soils over 75 m), Caltrans also magnifies the response acceleration S by 20% for $T \geq 1.5$ seconds, and the value determined by linear interpolation for $0.5 \leq T \leq 1.5$ seconds. The response acceleration is not magnified for $T \leq 0.5$ seconds.

A comparison of design linear response acceleration between EC-8, New Zealand, Japan and US was conducted under the following assumptions;

- Compare the largest linear response acceleration of 5% damping ratio corresponding to the largest factors and design accelerations, if available.
- Assume that three ground condition category in EC-8, New Zealand and Japan are close each other. They are called herein "stiff," "moderate," and "soft." Assume also that the soil profile Type-II, III and IV in AASHTO and the soil profile C, D and E in Caltrans correspond to the stiff, moderate and soft soil category.
- In EC-8, assume k_I=1.3 and a_g=0.7g in Eq. (6.44). It should be noted that a_g=0.7g may not be used in European countries, but this was assumed here for comparison. In New Zealand code, assume k_Z =1.2, k_I=1.3 and μ=1 in Eq. (6.46). In the Japanese code, assume k_Z=1.0 in Eqs. (6.2) and (6.3). In AASHTO, assume a_g=0.8g in Eq. (6.47). In Caltrans, consider two magnitude ranges; 7.25$\pm$0.25 and 8.0$\pm$0.25, and assume a_g=0.7g for stiff and moderate sites and a_g=0.4g for soft soil sites.

Fig. 6.9 shows the comparison among the design accelerations in 4 codes. At stiff sites, the general trend of the response accelerations is similar among the codes, except the Type-I ground motion in Japan provides smaller value. At soft soil sites, EC-8 and AASHTO provide larger response accelerations at the constant response acceleration range. The Caltrans, New Zealand and Japan codes provide the similar response. The difference between M=7.25$\pm$0.25 and 8.0$\pm$0.25 is small in the Caltrans. It is noted here that this comparison reflects the assumptions described above.

4) Analytical Methods and Design Requirements

In the ATC-32, the analytical methods are classified into "Elastic Static Analysis" (ESA), "Elastic Dynamic Analysis" (EDA), "Inelastic Static Analysis" (ISA), and "Inelastic Dynamic Analysis" (INA). In ESA, the elastic response of a bridge is divided by the response modification factor, and the structural components are sized based on the capacity design approach. On the other hand, pushover analysis is used in ISA. The minimum analysis requirements are presented in Table 6.6. The requirements vary with bridge category, configuration type and evaluation level. Configuration Type-I is intended to include bridges with simple response characteristics, including bridges with continuous superstructure, well-balanced spans, supporting bents with appropriately equal stiffness and insignificant vertical response. Configuration Type-II is intended to include bridges with more complex response characteristics that are unlikely to be represented well by ESA, including bridges with intermediate superstructure hinges, irregular configuration, bents of nonuniform stiffness, significant skew or spans likely to be excited by vertical input motion.

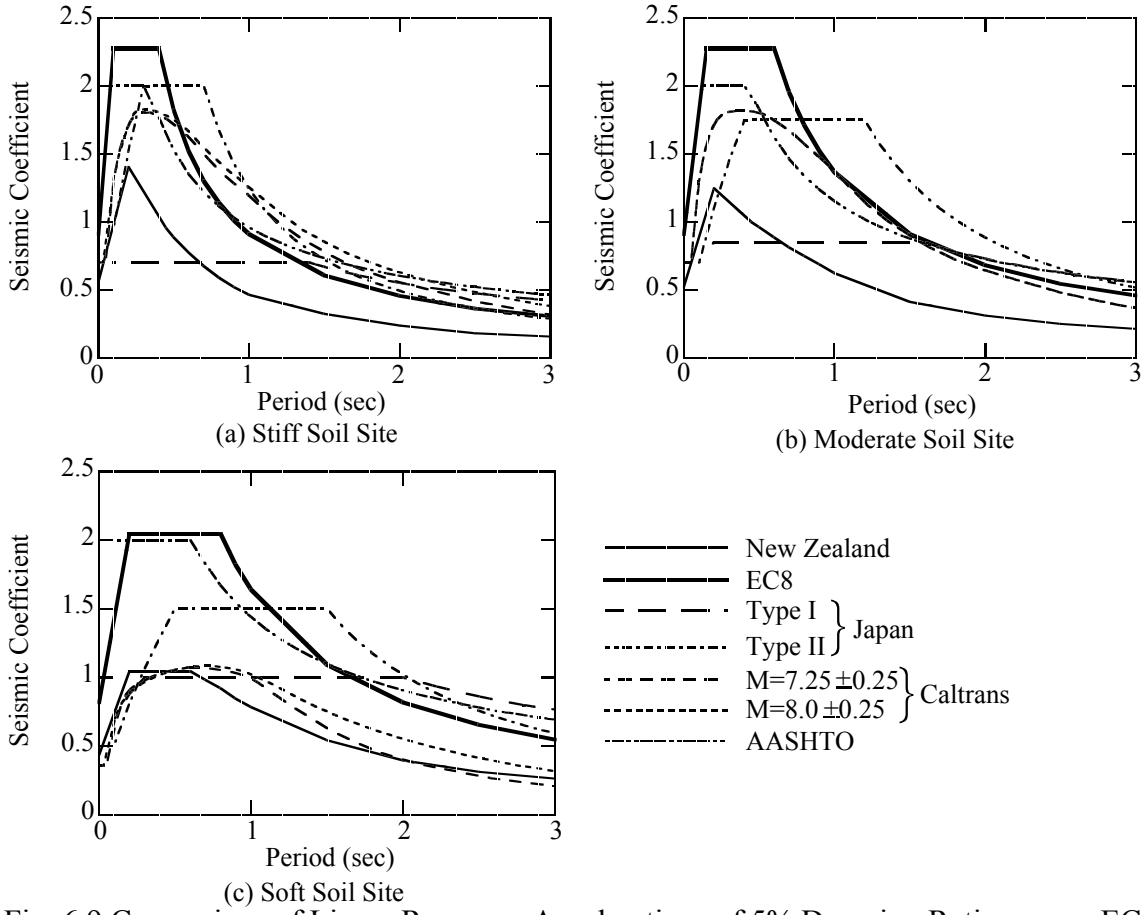


Fig. 6.9 Comparison of Linear Response Accelerations of 5% Damping Ratio among EC-8, New Zealand, Japan and US codes

Table 6.6 Minimum Required Analysis (ATC-32, 1996)

Importance	Configuration	Functional Evaluation	Safety Evaluation
Ordinary Bridge	Type-I	None required	ESA or EDA
	Type-II	None required	EDA
Important Bridge	Type-I	ESA or EDA	ESA or EDA
	Type-II	EDA	EDA, ISA and IDA

If one follows the classification in Table 6.6, EDA and ESA are used in EC-8. Detailed description is provided for three methods in ESA; the rigid deck mode, flexible deck model and individual pier model. They are selected depending on the relative deformation of deck and the interrelation of response of adjacent piers. In New Zealand, ESA, EDA and IDA are used. Dynamic analysis (EDA and IDA) is recommended when it is not appropriate to represent the bridge as a single-degree-of-freedom oscillator. In Japan, analytical methods are also chosen depending on the structural configuration as shown in Table 6.7. Push over analysis is used for almost bridges, in particular for design of moment-resisting frame piers and foundations. Inelastic dynamic response analysis is widely adopted on routine basis. Since the input data required for pushover analysis and EDA are almost the same with IDA, IDA is easily conducted once the data are prepared. Based on the advance of personal computers, movement toward more frequent use of IDA may be accelerated in the future. In Caltrans, ESA, EDA and ISA are used.

Table 6.7 Minimum Required Analysis (JRA, 1999)

Category		Functional Evaluation	Safety Evaluation
Bridges with Simple Response Characteristics		ESA	ISA
Bridges with Complex Response Characteristics	Equivalent Static Analysis is Applicable	ESA and EDA	ISA and IDA
	Equivalent Static Analysis is not Applicable	EDA	IDA

5) Response Modification Factor and Target Displacement Ductility Demand

In ESA, response modification factor is important in evaluating the demand. In EC-8, it is intended that bridges behave under the design seismic action in ductile or limited ductile/essentially elastic. In ductile behavior, the response modification factor (behavior factor q) is 1.0-3.5, 2.0-3.5, 1.0 and 2.0 for reinforced concrete piers, steel piers, abutments and arches, respectively.

In Japanese design specifications, the response modification factors are computed for each bridge by Eq. (6.12) based on the equal energy principle using the design displacement ductility factor of piers. As presented previously, since the design displacement ductility should be equal or less than 8, the response modification factor is equal to or smaller than 3.87. Since the design displacement ductility factor is generally in the range of 4-6 in ordinary reinforced concrete single piers, the response modification factor is in the range of 2.6-3.3.

In New Zealand, response modification factors are not required because nonlinear response accelerations are used. The maximum allowable displacement ductility factor is determined depending on the hysteretic structural behavior and accessibility to the damaged portion. For example, in the ductile or partially ductile structures (Type-I), the maximum allowable displacement ductility factor is 6 when plastic hinges form above the ground or normal water level. But it must be 4 when the plastic hinges form in reasonably accessible position (e.g., less than 2 m below ground surface), and 3 when they form at inaccessible position (e.g., more than 2 m below ground surface).

In AASHTO, the response modification factors are determined depending on the type of piers and the importance. They are 1.5, 1.5-3.5, and 2.0-5.0 in critical, essential, and other bridges, respectively. For single columns, it is 1.5, 2.0 and 3.0 in critical, essential and other bridges, respectively. Special interest is the response modification factor for connections. It is 0.8 for superstructure to abutment connection.

In Caltrans, the design maximum displacement ductility factor is provided. It is 5 for multi-column bents, 4 for single column bents, and 5 and 1 for pier walls in weak and strong direction, respectively. Quantitative definition of the damage corresponding to different performance requirements has not yet been specified in the Version 1.0 Caltrans-SDC. Table 6.8 provides a quantitative strain and ductility limit corresponding to the three damage levels [Duan and Chen 1999].

Table 6.8 Damage Levels, Strain and Ductility [Duan and Chen 199**9]**

Damage Level	Strain		Ductility	
	Concrete	Steel	Curvature μ_ϕ	Displacement μ_d
Significant	ε_{cu}	ε_{sh}	8-10	4-6
Repairable	larger between 0.005 and $2\varepsilon_{cu}/3$	larger between 0.08 and $2\varepsilon_y/3$	4-6	2-4
Minimum	larger between 0.004 and ε_{cu}	larger between 0.03 and $1.5\varepsilon_y$	2-4	1-2

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