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44

Seismic Design Practice in Japan

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Nomenclature

The following symbols are used in this chapter. The section number in parentheses after definition of a symbol refers to the section where the symbol first appears or is defined.

a	space of tie reinforcement (Section 44.4.4)
A_{CF}	sectional area of carbon fiber (Figure 44.19)
A_h	area of tie reinforcements (Section 44.4.4)
A_w	sectional area of tie reinforcement (Section 44.4.4)
b	width of section (Section 44.4.4)
c_B	coefficient to evaluate effective displacement (Section 44.4.7)
c_B	modification coefficient for clearance (Section 44.4.11)
c_{df}	modification coefficient (Section 44.4.2)
c_c	modification factor for cyclic loading (Section 44.4.4)
c_D	modification coefficient for damping ratio (Section 44.4.6)
c_e	modification factor for scale effect of effective width (Section 44.4.4)
c_E	modification coefficient for energy-dissipating capability (Section 44.4.7)
c_P	coefficient depending on the type of failure mode (Section 44.4.2)
c_{pt}	modification factor for longitudinal reinforcement ratio (Section 44.4.4)

c_R	factor depending on the bilinear factor r (Section 44.4.2)
c_W	corrective coefficient for ground motion characteristics (Section 44.4.9)
c_Z	modification coefficient for zone (Section 44.4.3)
d	effective width of tie reinforcements (Section 44.4.4)
d	height of section (Section 44.4.4)
D	a width or a diameter of a pier (Section 44.4.4)
D_E	coefficient to reduce soil constants according to F_L value (Section 44.4.11)
E_c	elastic modules of concrete (Section 44.4.4)
E_{CF}	elastic modulus of carbon fiber (Figure 44.19)
E_{des}	gradient at descending branch (Section 44.4.4)
F_L	liquefaction resistant ratio (Section 44.4.9)
$F(u)$	restoring force of a device at a displacement u (Section 44.4.7)
h	height of a pier (Section 44.4.4)
h_B	height of the center of gravity of girder from the top of bearing (Figure 44.13)
h_B	equivalent damping of a Menshin device (Section 44.4.7)
h_i	damping ratio of ith mode (Section 44.4.6)
h_{ij}	damping ratio of jth substructure in ith mode (Section 44.4.6)
h_{Bi}	damping ratio of ith damper (Section 44.4.7)
h_{Pi}	damping ratio of ith pier or abutment (Section 44.4.7)
h_{Fui}	damping ratio of ith foundation associated with translational displacement (Section 44.4.7)
$h_{F\theta i}$	damping ratio of ith foundation associated with rotational displacement(Section 44.4.7)
H	distance from a bottom of pier to a gravity center of a deck (Section 44.4.7)
H_0	shear force at the bottom of footing (Figure 44.12)
I	importance factor (Section 44.5.2)
k_{hc}	lateral force coefficient (Section 44.4.2)
k_{hc}	design seismic coefficient for the evaluation of liquefaction potential (Section 44.4.9)
k_{hc0}	standard modification coefficient (Section 44.4.3)
k_{hcm}	lateral force coefficient in Menshin design (Section 44.4.7)
k_{he}	equivalent lateral force coefficient (Section 44.4.2)
k_{hem}	equivalent lateral force coefficient in Menshin design (Section 44.4.7)
k_{hp}	lateral force coefficient for a foundation (Section 44.4.2)
k_j	stiffness matrix of jth substructure (Section 44.4.6)
K	stiffness matrix of a bridge (Section 44.4.6)
K_B	equivalent stiffness of a Menshin device (Section 44.4.7)
K_{Pi}	equivalent stiffness of ith pier or abutment (Section 44.4.7)
K_{Fui}	translational stiffness of ith foundation (Section 44.4.7)
$K_{F\theta i}$	rotational stiffness of ith foundation (Section 44.4.7)
L	shear stress ratio during an earthquake (Section 44.4.9)
L_A	redundancy of a clearance (Section 44.4.11)
L_E	clearance at an expansion joint (Section 44.4.11)
L_P	plastic hinge length of a pier (Section 44.4.4)
M_0	moment at the bottom of footing (Figure 44.12)
P_a	lateral capacity of a pier (Section 44.4.2)
P_s	shear capacity in consideration of the effect of cyclic loading (Section 44.4.4)
P_{s0}	shear capacity without consideration of the effect of cyclic loading (Section 44.4.4)
P_u	bending capacity (Section 44.4.2)
r	bilinear factor defined as a ratio between the first stiffness (yield stiffness) and the second stiffness (postyield stiffness) of a pier (Section 44.4.2)
r_d	modification factor of shear stress ratio with depth (Section 44.4.9)
R	dynamic shear strength ratio (Section 44.4.9)
R	priority (Section 44.5.2)
R_D	dead load of superstructure (Section 44.4.11)
R_{heq} and R_{veq}	vertical reactions caused by the horizontal seismic force and vertical force (Section 44.4.11)
R_L	cyclic triaxial strength ratio (Section 44.4.9)
R_U	design uplift force applied to the bearing support (Section 44.4.11)
s	space of tie reinforcements (Section 44.4.4)
S	earthquake force (Section 44.5.2)
S_c	shear capacity shared by concrete (Section 44.4.4)
S_{I} and S_{II}	acceleration response spectrum for Type-I and Type-II ground motions (Section 44.4.6)

S_{I0} and S_{II0} standard acceleration response spectrum for Type-I and Type-II ground motions (Section 44.4.6)
S_E seat length (Section 44.4.11)
S_{EM} minimum seat length (cm) (Section 44.4.11)
S_s shear capacity shared by tie reinforcements (Section 44.4.4)
T natural period of fundamental mode (Table 44.3)
ΔT difference of natural periods (Section 44.4.11)
T_1 and T_2 natural periods of the two adjacent bridge systems (Section 44.4.11)
u_B design displacement of isolators (Section 44.4.7)
u_{Be} effective design displacement (Section 44.4.7)
u_{Bi} design displacement of ith Menshin device (Section 44.4.7)
u_G relative displacement of ground along the bridge axis (Section 44.4.11)
u_R relative displacement (cm) developed between a superstructure and a substructure (Section 44.4.11)
V_0 vertical force at the bottom of footing (Figure 44.12)
V_T structural factor (Section 44.5.2)
V_{RP1} design specification (Section 44.5.2)
V_{RP2} pier structural factor (Section 44.5.2)
V_{RP3} aspect ratio (Section 44.5.2)
V_{MP} steel pier factor (Section 44.5.2)
V_{FS} unseating device factor (Section 44.5.2)
V_F foundation factor (Section 44.5.2)
w_v weighting factor on structural members (Section 44.5.2)
W equivalent weight (Section 44.4.2)
W elastic strain energy (Section 44.4.7)
W_P weight of a pier (Section 44.4.2)
W_U weight of a part of superstructure supported by the pier (Section 44.4.2)
ΔW energy dissipated per cycle (Section 44.4.7)
α safety factor (Section 44.4.4)
α, β coefficients depending on shape of pier (Section 44.4.4)
α_m safety factor used in Menshin design (Section 44.4.7)
δ_y yield displacement of a pier (Section 44.4.2)
δ_R residual displacement of a pier after an earthquake (Section 44.4.2)
δ_{Ra} allowable residual displacement of a pier (Section 44.4.2)
δ_u ultimate displacement of a pier (Section 44.4.4)
ε_c strain of concrete (Section 44.4.4)
ε_{cc} strain at maximum strength (Section 44.4.4)
ε_G ground strain induced during an earthquake along the bridge axis (Section 44.4.11)
ε_s strain of reinforcements (Section 44.4.4)
ε_{sy} yield strain of reinforcements (Section 44.4.4)
θ angle between vertical axis and tie reinforcement (Section 44.4.4)
θ_{pu} ultimate plastic angle (Section 44.4.4)
μ_a allowable displacement ductility factor of a pier (Section 44.4.2)
μ_m allowable ductility factor of a pier in Menshin design (Section 44.4.7)
μ_R response ductility factor of a pier (Section 44.4.2)
ρ_s tie reinforcement ratio (Section 44.4.4)
σ_c stress of concrete (Section 44.4.4)
σ_{cc} strength of confined concrete (Section 44.4.4)
σ_{CF} stress of carbon fiber (Figure 44.19)
σ_{ck} design strength of concrete (Section 44.4.4)
σ_s stress of reinforcements (Section 44.4.4)
σ_{sy} yield strength of reinforcements (Section 44.4.4)
σ_v total loading pressure (Section 44.4.9)
σ_v' effective loading pressure (Section 44.4.9)
τ_c shear stress capacity shared by concrete (Section 44.4.4)
ϕ_{ij} mode vector of jth substructure in ith mode (Section 44.4.6)
ϕ_i mode vector of a bridge in ith mode (Section 44.4.6)
ϕ_y yield curvature of a pier at bottom (Section 44.4.4)
ϕ_u ultimate curvature of a pier at bottom (Section 44.4.4)

44.1 Introduction

Japan is one of the most seismically disastrous countries in the world and has often suffered significant damage from large earthquakes. More than 3000 highway bridges have suffered damage since the 1923 Kanto earthquake. The earthquake disaster prevention technology for highway bridges has been developed based on such bitter damage experiences. Various provisions for designing bridges have been developed to prevent damage due to the instability of soils such as soil liquefaction. Furthermore, design detailings including unseating prevention devices are implemented. With progress in improving seismic design provisions, damage to highway bridges caused by the earthquakes has been decreasing in recent years.

However, the Hyogo-ken Nanbu earthquake of January 17, 1995 caused destructive damage to highway bridges. Collapse and near collapse of superstructures occurred at nine sites, and other destructive damage occurred at 16 sites [1]. The earthquake revealed that there are a number of critical issues to be revised in the seismic design and seismic retrofit of bridges [2,3].

This chapter presents technical developments for seismic design and seismic retrofit of highway bridges in Japan. The history of the earthquake damage and development of the seismic design methods is first described. The damage caused by the 1995 Hyogo-ken Nanbu earthquake, the lessons learned from the earthquake, and the seismic design methods introduced in the 1996 *Seismic Design Specifications for Highway Bridges* are then described. Seismic performance levels and design methods as well as ductility design methods for reinforced concrete piers, steel piers, foundations, and bearings are described. Then the history of the past seismic retrofit practices is described. The seismic retrofit program after the Hyogo-ken-Nanbu earthquake is described with emphasis on the seismic retrofit of reinforced concrete piers as well as research and development on the seismic retrofit of existing highway bridges.

44.2 History of Earthquake Damage and Development of Seismic Design Methods

A year after the 1923 Great Kanto earthquake, consideration of the seismic effect in the design of highway bridges was initiated. The Civil Engineering Bureau of the Ministry of Interior promulgated "The Method of Seismic Design of Abutments and Piers" in 1924. The seismic design method has been developed and improved through bitter experience in a number of past earthquakes and with progress of technical developments in earthquake engineering. Table 44.1 summarizes the history of provisions in seismic design for highway bridges.

In particular, the seismic design method was integrated and upgraded by compiling the "Specifications for Seismic Design of Highway Bridges" in 1971. The design method for soil liquefaction and unseating prevention devices was introduced in the Specifications. It was revised in 1980 and integrated as "Part V: Seismic Design" in Design Specifications of Highway Bridges. The primitive check method for ductility of reinforced concrete piers was included in the reference of the Specifications. It was further revised in 1990 and ductility check of reinforced concrete piers, soil liquefaction, dynamic response analysis, and design detailings were prescribed. It should be noted here that the detailed ductility check method for reinforced concrete piers was first introduced in the 1990 Specifications.

However, the Hyogo-ken Nanbu earthquake of January 17, 1995, exactly 1 year after the Northridge earthquake, California, caused destructive damage to highway bridges as described earlier. After the earthquake the Committee for Investigation on the Damage of Highway Bridges Caused by the Hyogo-ken Nanbu Earthquake (chairman, Toshio Iwasaki, Executive Director, Civil Engineering Research Laboratory) was established in the Ministry of Construction to investigate the damage and to identify the factors that caused the damage.

TABLE 44.1 History of Seismic Design Methods

		1926 Details of Road Structure (draft) Road Law, MIA	1939 Design Specifications of Steel Highway Bridges (draft) MIA	1956 Design Specifications of Steel Highway Bridges, MOC	1964 Design Specifications of Substructures (Pile Foundations), MOC	1964 Design Specifications of Steel Highway Bridges, MOC	1966 Design Specifications of Substructures (Survey and Design), MOC	1968 Design Specifications of Substructures (Piers and Direct Foundations), MOC	1970 Design Specifications of Substructures (Caisson Foundations), MOC	1971 Specifications for Seismic Design of Highway Bridges, MOC	1972 Design Specifications of Substructures (Cast-in-Piles), MOC	1975 Design Specifications of Substructures (Pile Foundations), MOC	1980 Design Specifications of Highway Bridges, MOC	1990 Design Specifications of Highway Bridges, MOC
Seismic loads	Seismic coefficient	Largest seismic loads Varied dependent on the site	$k_h = 0.2$	$k_h = 0.1–0.35$ Varied dependent on the site and ground condition						$k_h = 0.1–0.3$ Standardization of seismic coefficient provision of modified seismic coefficient method			Revision of application range of modified seismic coefficient method	$k_h = 0.1–0.3$ Integration of seismic coefficient method and modified one.
	Dynamic earth pressure	Equations proposed by Mononobe and Okabe were supposed to be used					Provision of dynamic earth pressure							
	Dynamic hydraulic pressure	Less effect on piers except high piers in deep water					Provision of hydraulic pressure			Provision of dynamic hydraulic pressure				
Reinforced concrete column	Bending at bottom	Supposed to be designed in a similar way provided in current design Specifications						Provisions of Definite Design Method						
	Shear	Less effect on RC piers except those with smaller section area such as RC frame and hollow section						Check of shear strength					Provision of definite design method, decreasing of allowable shear stress	
	Termination of Main Reinforcement at Midheight												Elongation of anchorage length of terminated reinforcement at midheight	
	Bearing capacity for lateral force							Less effect on RC piers with larger section area					Ductility check	Check for bearing capacity for lateral force
Footing								Provisions of definite design method (designed as a cantilever plate)					Provisions of effective width and check of shear strength	
Pile foundation		Bearing capacity in vertical direction was supposed to be checked				Provisions of Definite Design Method (bearing capacity in vertical and horizontal directions)						Special Condition (Foundation on Slope, Consolidation Settlement, Lateral Movement)	Provisions of Design Details for Pile Head	
Direct foundation		Stability (overturning and slip) was supposed to be checked						Provisions of Definite Design Method (bearing capacity, stability analysis)						
Caisson foundation				Supposed to be designed in a similar way provided in Design Specification of Caisson Foundation of 1969					Provisions of Definite Design Method					
Soil Liquefaction										Provisions of soil layers of which bearing capacity shall be ignored in seismic design			Provisions of evaluation method of soil liquefaction and the treatment in seismic design	Consideration of effect of fine sand content
Bearing support	Bearing support		Provisions of Design Methods for steel bearing supports (bearing, roller, anchor bolt)							Provision of transmitting method of seismic load at bearing				
	Devices preventing falling-off of superstructure							Provision of bearing seat length S		Provisions of stopper at movable bearings, devices for preventing superstructure from falling (seat length S, connection of adjacent decks)			Provisions of stopper at movable bearings, devices for preventing superstructure from falling (seat length S_E devices)	

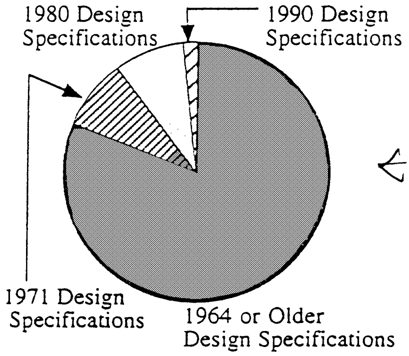


FIGURE 44.1 Design specifications referred to in design of Hanshin Expressway [2].

On February 27, 1995, the Committee approved the "Guide Specifications for Reconstruction and Repair of Highway Bridges Which Suffered Damage Due to the Hyogo-ken Nanbe Earthquake," [4], and the Ministry of Construction announced on the same day that the reconstruction and repair of the highway bridges which suffered damage in the Hyogo-ken Nanbu earthquake should be made by the Guide Specifications. It was decided by the Ministry of Construction on May 25, 1995 that the Guide Specifications should be tentatively used in all sections of Japan as emergency measures for seismic design of new highway bridges and seismic strengthening of existing highway bridges until the Design Specifications of Highway Bridges is revised.

In May, 1995, the Special Sub-Committee for Seismic Countermeasures for Highway Bridges (chairman, Kazuhiko Kawashima, Professor of the Tokyo Institute of Technology) was established in the Bridge Committee (chairman, Nobuyuki Narita, Professor of the Tokyo Metropolitan University), Japan Road Association, to draft the revision of the Design Specifications of Highway Bridges. The new Design Specifications of Highway Bridges [5,6] was approved by the Bridge Committee, and issued by the Ministry of Construction on November 1, 1996.

44.3 Damage of Highway Bridges Caused by the Hyogo-ken Nanbu Earthquake

The Hyogo-ken Nanbu earthquake was the first earthquake to hit an urban area in Japan since the 1948 Fukui earthquake. Although the magnitude of the earthquake was moderate (M7.2), the ground motion was much larger than anticipated in the codes. It occurred very close to Kobe City with shallow focal depth.

Damage was developed at highway bridges on Routes 2, 43, 171, and 176 of the National Highway, Route 3 (Kobe Line) and Route 5 (Bay Shore Line) of the Hanshin Expressway, and the Meishin and Chugoku Expressways. Damage was investigated for all bridges on national highways, the Hanshin Expressway, and expressways in the area where destructive damage occurred. The total number of piers surveyed reached 3396 [1]. Figure 44.1 shows Design Specifications referred to in the design of the 3396 highway bridges. Most of the bridges that suffered damage were designed according to the 1964 Design Specifications or the older Design Specifications. Although the seismic design methods have been improved and amended several times since 1926, only a requirement for lateral force coefficient was provided in the 1964 Design Specifications or the older Specifications.

Figure 44.2 compares damage of piers (bridges) on the Route 3 (Kobe Line) and Route 5 (Bay Shore Line) of the Hanshin Expressway. Damage degree was classified as A_s (collapse), A (nearly collapse), B (moderate damage), C (damage of secondary members), and D (minor or no damage). Substructures on Route 3 and Route 5 were designed with the 1964 Design Specifications and the 1980 Design Specifications, respectively. It should be noted in this comparison that the intensity of

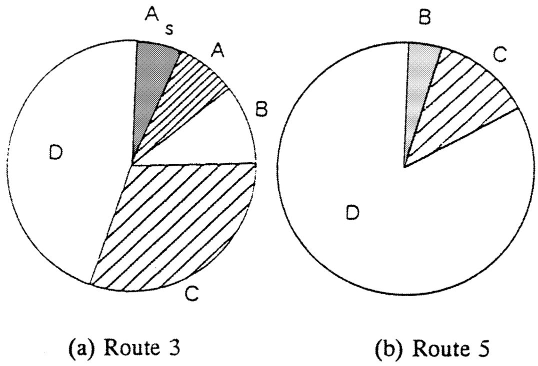


FIGURE 44.2 Comparison of damage degree between Route 3 (a) and Route 5 (b) (As: collapse, A: near collapse, B: moderate damage, C: damage of secondary members, D: minor or no damage) [2].

ground shaking in terms of response spectra was smaller at the Bay Area than the narrow rectangular area where JMA seismic intensity was VII (equivalent to modified Mercalli intensity of X-XI). Route 3 was located in the narrow rectangular area, while Route 5 was located in the Bay Area. Keeping in mind such differences in ground motion, it is apparent in Figure 44.2 that about 14% of the piers on Route 3 suffered As or A damage while no such damage was developed in the piers on Route 5.

Although damage concentrated on the bridges designed with the older Design Specifications, it was thought that essential revision was required even in the recent Design Specifications to prevent damage against destructive earthquakes such as the Hyogo-ken Nanbu earthquake. The main modifications were as follows:

1. To increase lateral capacity and ductility of all structural components in which seismic force is predominant so that ductility of a total bridge system is enhanced. For such purpose, it was required to upgrade the "Check of Ductility of Reinforced Concrete Piers," which has been used since 1990, to a "ductility design method" and to apply the ductility design method to all structural components. It should be noted here that "check" and "design" are different; the check is only to verify the safety of a structural member designed by another design method, and is effective only to increase the size or reinforcements if required, while the design is an essential procedure to determine the size and reinforcements.
2. To include the ground motion developed at Kobe in the earthquake as a design force in the ductility design method.
3. To specify input ground motions in terms of acceleration response spectra for dynamic response analysis more actively.
4. To increase tie reinforcements and to introduce intermediate ties for increasing ductility of piers. It was decided not to terminate longitudinal reinforcements at midheight to prevent premature shear failure, in principle.
5. To adopt multispan continuous bridges for increasing number of indeterminate of a total bridge system.
6. To adopt rubber bearings for absorbing lateral displacement between a superstructure and substructures and to consider correct mechanism of force transfer from a superstructure to substructures.
7. To include the Menshin design (seismic isolation).
8. To increase strength, ductility, and energy dissipation capacity of unseating prevention devices.
9. To consider the effect of lateral spreading associated with soil liquefaction in design of foundations at sites vulnerable to lateral spreading.

TABLE 44.2 Seismic Performance Levels

		Importance of Bridges		Design Methods	
Type of Design Ground Motions		Type-A (Standard Bridges)	Type-B (Important Bridges)	Equivalent Static Lateral Force Methods	Dynamic Analysis
Ground motions with high probability to occur		Prevent Damage		Seismic coefficient method	Step by Step analysis or Response spectrum analysis
Ground motions with low probability to occur	Type I (plate boundary earthquakes)	Prevent critical damage	Limited damage	Ductility design method	
	Type II (Inland earthquakes)				

44.4 1996 Seismic Design Specifications of Highway Bridges

44.4.1 Basic Principles of Seismic Design

The 1995 Hyogo-ken Nanbu earthquake, the first earthquake to be considered that such destructive damage could be prevented due to the progress of construction technology in recent years, provided a large impact on the earthquake disaster prevention measures in various fields. Part V: Seismic Design of the Design Specifications of Highway Bridges (Japan Road Association) was totally revised in 1996, and the design procedure moved from the traditional seismic coefficient method to the ductility design method. The revision was so comprehensive that the past revisions of the last 30 years look minor.

A major revision of the 1996 Specifications is the introduction of explicit two-level seismic design consisting of the seismic coefficient method and the ductility design method. Because Type I and Type II ground motions are considered in the ductility design method, three design seismic forces are used in design. Seismic performance for each design force is clearly defined in the Specifications.

Table 44.2 shows the seismic performance level provided in the 1996 Design Specifications. The bridges are categorized into two groups depending on their importance: standard bridges (Type A bridges) and important bridges (Type B bridges). The seismic performance level depends on the importance of the bridge. For moderate ground motions induced in earthquakes with a high probability of occurrence, both A and B bridges should behave in an elastic manner without essential structural damage. For extreme ground motions induced in earthquakes with a low probability of occurrence, Type A bridges should prevent critical failure, whereas Type B bridges should perform with limited damage.

In the ductility design method, two types of ground motions must be considered. The first is the ground motions that could be induced in plate boundary-type earthquakes with a magnitude of about 8. The ground motion at Tokyo in the 1923 Kanto earthquake is a typical target of this type of ground motion. The second is the ground motion developed in earthquakes with magnitude of about 7 to 7.2 at very short distance. Obviously, the ground motions at Kobe in the Hyogo-ken Nanbu earthquake is a typical target of this type of ground motion. The first and the second ground motions are called Type I and Type II ground motions, respectively. The recurrence time of Type II ground motion may be longer than that of Type I ground motion, although the estimation is very difficult.

The fact that lack of near-field strong motion records prevented serious evaluation of the validity of recent seismic design codes is important. The Hyogo-ken Nanbu earthquake revealed that the history of strong motion recording is very short, and that no near-field records have yet been measured by an earthquake with a magnitude on the order of 8. It is therefore essential to have sufficient redundancy and ductility in a total bridge system.

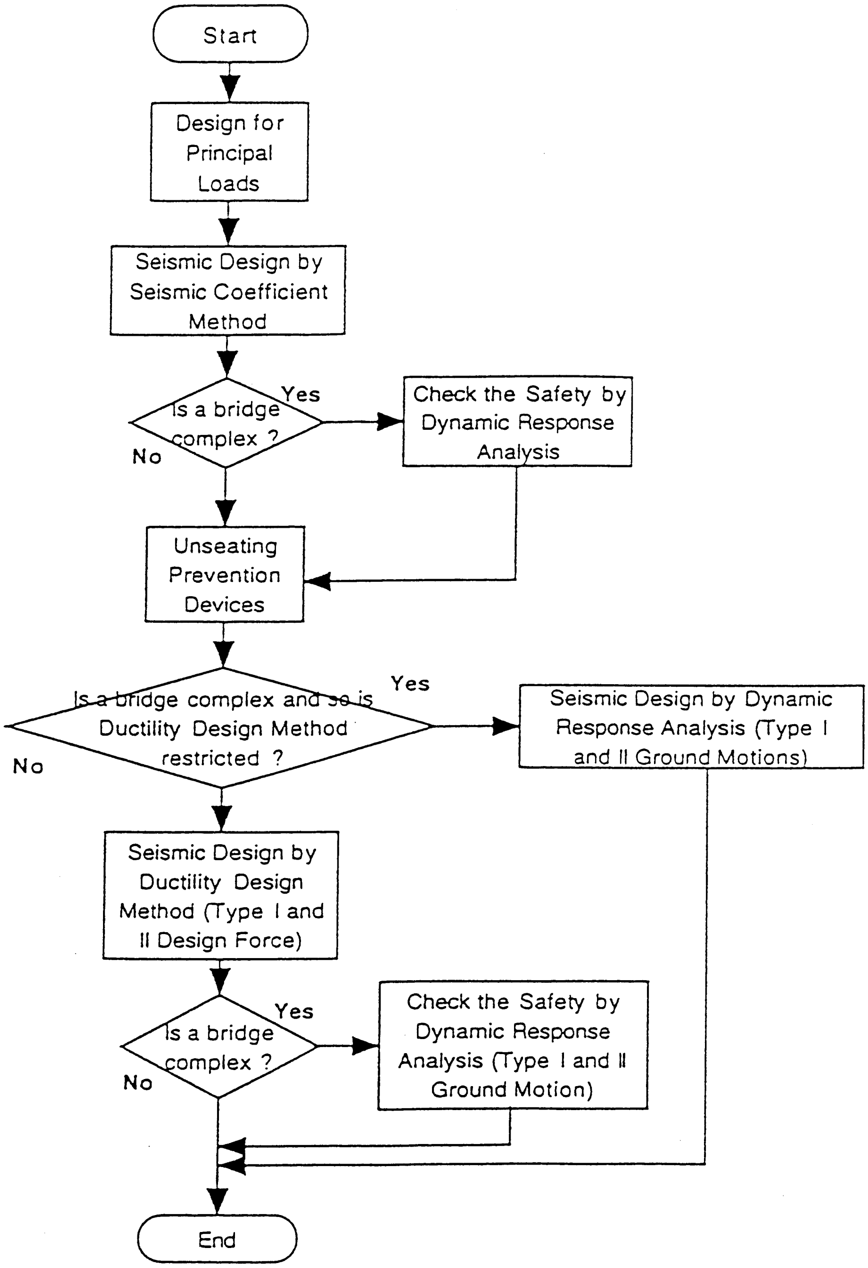


FIGURE 44.3 Flowchart of seismic design.

44.4.2 Design Methods

Bridges are designed by both the seismic coefficient method and the ductility design method as shown in Figure 44.3. In the seismic coefficient method, a lateral force coefficient ranging from 0.2 to 0.3 has been used based on the allowable stress design approach. No change has been made since the 1990 Specifications in the seismic coefficient method.

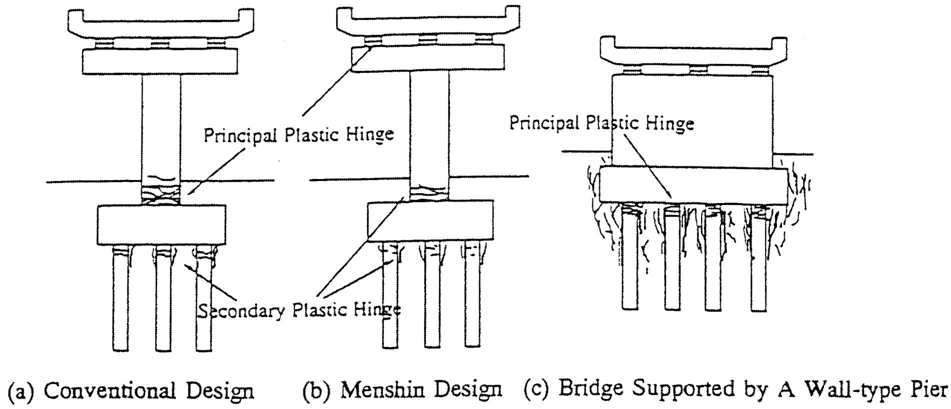


FIGURE 44.4 Location of primary plastic hinge. (a) Conventional design; (b) Menshin design; (c) bridge supported by a wall-type pier.

In the ductility design method, assuming a principal plastic hinge is formed at the bottom of pier as shown in Figure 44.4a and that the equal energy principle applies, a bridge is designed so that the following requirement is satisfied:

$$P_a > k_{he} W \tag{44.1}$$

where

$$k_{he} = \frac{k_{hc}}{\sqrt{2\mu_a - 1}} \tag{44.2}$$

$$W = W_U + c_P W_P \tag{44.3}$$

in which P_a = lateral capacity of a pier, k_{he} = equivalent lateral force coefficient, W = equivalent weight, k_{hc} = lateral force coefficient, μ_a = allowable displacement ductility factor of a pier, W_U = weight of a part of superstructure supported by the pier, W_P = weight of a pier, and c_P = coefficient depending on the type of failure mode. The c_P is 0.5 for a pier in which either flexural failure or shear failure after flexural cracks are developed, and 1.0 is for a pier in which shear failure is developed. The lateral capacity of a pier P_a is defined as a lateral force at the gravity center of a superstructure.

In Type B bridges, residual displacement developed at a pier after an earthquake must be checked as

$$\delta_R < \delta_{Ra} \tag{44.4}$$

where

$$\delta_R = c_R \left(\mu_R - 1\right)(1 - r)\delta_y \tag{44.5}$$

$$\mu_R = 1/2 \left\{\left(k_{hc} \cdot W/P_a\right)^2 + 1\right\} \tag{44.6}$$

in which δ_R = residual displacement of a pier after an earthquake, δ_{Ra} = allowable residual displacement of a pier, r = bilinear factor defined as a ratio between the first stiffness (yield stiffness) and the second stiffness (postyield stiffness) of a pier, c_R = factor depending on the bilinear factor r, μ_R = response ductility factor of a pier, and δ_y = yield displacement of a pier. The δ_{Ra} should be 1/100 of the distance between the bottom of a pier and the gravity center of a superstructure.

In a bridge with complex dynamic response, the dynamic response analysis is required to check the safety of a bridge after it is designed by the seismic coefficient method and the ductility design method. Because this is only for a check of the design, the size and reinforcements of structural members once determined by the seismic coefficient method and the ductility design methods may be increased if necessary. It should be noted, however, that under the following conditions in which the ductility design method is not directly applied, the size and reinforcements can be determined based on the results of a dynamic response analysis as shown in Figure 44.3. Situations when the ductility design method should not be directly used include:

1. When principal mode shapes that contribute to bridge response are different from the ones assumed in the ductility design methods
2. When more than two modes significantly contribute to bridge response
3. When principal plastic hinges form at more than two locations, or principal plastic hinges are not known where to be formed
4. When there are response modes for which the equal energy principle is not applied

In the seismic design of a foundation, a lateral force equivalent to the ultimate lateral capacity of a pier P_u is assumed to be a design force as

$$k_{hp} = c_{df}\, P_u / W \tag{44.7}$$

in which k_{hp} = lateral force coefficient for a foundation, c_{df} = modification coefficient (= 1.1), and W = equivalent weight by Eq. (44.3). Because the lateral capacity of a wall-type pier is very large in the transverse direction, the lateral seismic force evaluated by Eq. (44.7) in most cases becomes excessive. Therefore, if a foundation has sufficiently large lateral capacity compared with the lateral seismic force, the foundation is designed assuming a plastic hinge at the foundation and surrounding soils as shown in Figure 44.4c.

44.4.3 Design Seismic Force

Lateral force coefficient k_{hc} in Eq. (44.2) is given as

$$k_{hc} = c_z \cdot k_{hc0} \tag{44.8}$$

in which c_Z = modification coefficient for zone, and is 0.7, 0.85, and l.0 depending on the zone, and k_{hc0} = standard modification coefficient. Table 44.3 and Figure 44.5 show the standard lateral force coefficients k_{hc0} for Type I and Type II ground motions. Type I ground motions have been used since 1990 (1990 Specifications), while Type II ground motions were newly introduced in the 1996 Specifications. It should be noted here that the k_{hc0} at stiff site (Group I) has been assumed smaller than the k_{hc0} at moderate (Group II) and soft soil (Group III) sites in Type I ground motions as well as the seismic coefficients used for the seismic coefficient method. Type I ground motions were essentially estimated from an attenuation equation for response spectra that is derived from a statistical analysis of 394 components of strong motion records. Although the response spectral accelerations at short natural period are larger at stiff sites than at soft soil sites, the tendency has not been explicitly included in the past. This was because damage has been more developed at soft sites than at stiff sites. To consider such a fact, the design force at stiff sites is assumed smaller than

TABLE 44.3 Lateral Force Coefficient k_{hc0} in the Ductility Design Method

Soil Condition	Lateral Force Coefficient k_{hc0}		
		Type I Ground Motion	
Group I (stiff)		$k_{hc0} = 0.7$ for $T \leqq 1.4$	$k_{hc0} = 0.876T^{2/3}$ for $T > 1.4$
Group II (moderate)	$k_{hc0} = 1.51T^{1/3}$ ($k_{hc0} \geqq 0.7$) for $T < 0.18$	$k_{hc0} = 0.85$ for $0.18 \leqq T \leqq 1.6$	$k_{hc0} = 1.16T^{2/3}$ for $T > 1.6$
Group III (soft)	$k_{hc0} = 1.51T^{1/3}$ ($k_{hc0} \geqq 0.7$) for $T < 0.29$	$k_{hc0} = 1.0$ for $0.29 \leqq T \leqq 2.0$	$k_{hc0} = 1.59T^{2/3}$ for $T > 2.0$
		Type II Ground Motion	
Group I (stiff)	$k_{hc0} = 4.46T^{2/3}$ for $T \leqq 0.3$	$k_{hc0} = 2.00$ for $0.3 \leqq T \leqq 0.7$	$k_{hc0} = 1.24T^{4/3}$ for $T > 0.7$
Group II (moderate)	$k_{hc0} = 3.22T^{2/3}$ for $T < 0.4$	$k_{hc0} = 1.75$ for $0.4 \leqq T \leqq 1.2$	$k_{hc0} = 2.23T^{4/3}$ for $T > 1.2$
Group III (soft)	$k_{hc0} = 2.38T^{2/3}$ for $T < 0.5$	$k_{hc0} = 1.50$ for $0.5 \leqq T \leqq 1.5$	$k_{hc0} = 2.57T^{4/3}$ for $T > 1.5$

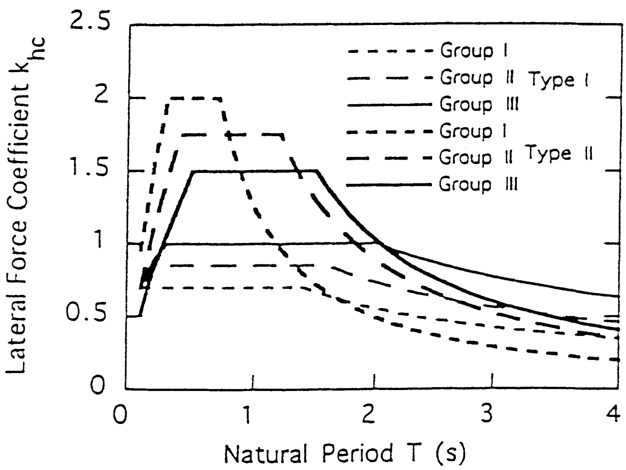


FIGURE 44.5 Type I and Type II ground motions in the ductility design method.

that at soft sites even at short natural period. However, being different from such a traditional consideration, Type II ground motions were determined by simply taking envelopes of response accelerations of major strong motions recorded at Kobe in the Hyogo-ken Nanbu earthquake.

Although the acceleration response spectral intensity at short natural period is higher in Type II ground motions than in Type I ground motions, the duration of extreme accelerations excursion is longer in Type I ground motions than Type II ground motions. As will be described later, such a difference of the duration has been taken into account to evaluate the allowable displacement ductility factor of a pier.

44.4.4 Ductility Design of Reinforced Concrete Piers

44.4.4.1 Evaluation of Failure Mode

In the ductility design of reinforced concrete piers, the failure mode of the pier is evaluated as the first step. Failure modes are categorized into three types based on the flexural and shear capacities of the pier as

1. $P_u \leqq P_s$ bending failure
2. $P_s \leq P_u \leqq P_{s0}$ bending to shear failure
3. $P_{s0} \leq P_u$ shear failure

in which P_u = bending capacity, P_s = shear capacity in consideration of the effect of cyclic loading, and P_{s0} = shear capacity without consideration of the effect of cyclic loading.

The ductility factor and capacity of the reinforced concrete piers are determined according to the failure mode as described later.

44.4.4.2 Displacement Ductility Factor

Th allowable displacement ductility factor of a pier μ_a in Eq. (44.2) is evaluated as

$$\mu_a = 1 + \frac{\delta_u - \delta_y}{\alpha \delta_y} \tag{44.9}$$

in which α = safety factor, δ_y = yield displacement of a pier, and δ_u = ultimate displacement of a pier. As well as the lateral capacity of a pier P_a in Eq. (44.1), the δ_y and δ_u are defined at the gravity center of a superstructure. In a reinforced concrete single pier as shown in Figure 44.4a, the ultimate displacement δ_u is evaluated as

$$\delta_u = \delta_y + \left(\phi_u - \phi_y\right) L_P \left(h - L_P/2\right) \tag{44.10}$$

in which ϕ_y = yield curvature of a pier at bottom, ϕ_u = ultimate curvature of a pier at bottom, h = height of a pier, and L_P = plastic hinge length of a pier. The plastic hinge length is given as

$$L_P = 0.2\,h - 0.1\,D \left(0.1\,D \leqq L_P \leqq 0.5D\right) \tag{44.11}$$

in which D is a width or a diameter of a pier.

The yield curvature ϕ_y and ultimate curvature ϕ_u in Eq. (44.10) are evaluated assuming a stress–strain relation of reinforcements and concrete as shown in Figure 44.6. The stress σ_c – strain ε_c relation of concrete with lateral confinement is assumed as

$$\sigma_c = \begin{cases} E_c \varepsilon_c \left(1 - \dfrac{1}{n}\left(\dfrac{\varepsilon_c}{\varepsilon_{cc}}\right)^{n-1}\right) & \left(0 \leqq \varepsilon_c \leqq \varepsilon_{cc}\right) \\ \sigma_{cc} - E_{des}\left(\varepsilon_c - \varepsilon_{cc}\right) & \left(\varepsilon_{cc} < \varepsilon_c \leqq \varepsilon_{cu}\right) \end{cases} \tag{44.12}$$

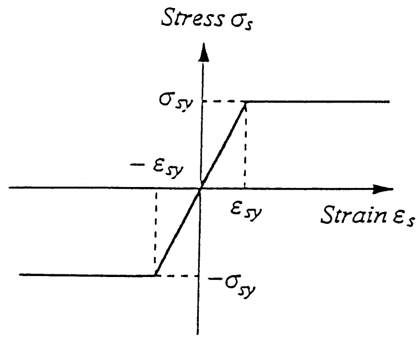
$$n = \frac{E_c \varepsilon_{cc}}{E_c \varepsilon_{cc} - \sigma_{cc}} \tag{44.13}$$

in which σ_{cc} = strength of confined concrete, E_c = elastic modules of concrete, ε_{cc} = strain at maximum strength, and E_{des} = gradient at descending branch. In Eq. (44.12), σ_{cc}, ε_{cc}, and E_{des} are determined as

$$\sigma_{cc} = \sigma_{ck} + 3.8\alpha\rho_s\sigma_{sy} \tag{44.14}$$

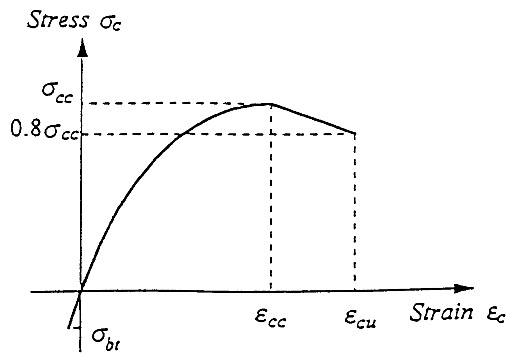
$$F = cV^k \tag{44.15}$$

$$E_{des} = 11.2\frac{\sigma_{ck}^2}{\rho_s \sigma_{sy}} \tag{44.16}$$



(a)

(b) Concrete



(b)

FIGURE 44.6 Stress and strain relation of confined concrete and reinforcing bars. (a) Steel (b) concrete.

in which σ_{ck} = design strength of concrete, σ_{sy} = yield strength of reinforcements, α and β = coefficients depending on shape of pier ($\alpha = 1.0$ and $\beta = 1.0$ for a circular pier, and $\alpha = 0.2$ and $\beta = 0.4$ for a rectangular pier), and ρ_s = tie reinforcement ratio defined as

$$\rho_s = \frac{4A_h}{sd} \leqq 0.018 \tag{44.17}$$

in which A_h = area of tie reinforcements, s = space of tie reinforcements, and d = effective width of tie reinforcements.

The ultimate curvature ϕ_u is defined as a curvature when concrete strain at longitudinal reinforcing bars in compression reaches an ultimate strain ε_{cu} defined as

$$\varepsilon_{cu} = \begin{cases} \varepsilon_{cc} & \text{for Type I ground motions} \\ \varepsilon_{cc} + \dfrac{0.2\sigma_{cc}}{E_{des}} & \text{for Type II ground motions} \end{cases} \tag{44.18}$$

It is important to note that the ultimate strain ε_{cu} depends on the types of ground motions; the ε_{cu} for Type II ground motions is larger than that for Type I ground motions. Based on a loading test,

TABLE 44.4 Safety Factor α in Eq. 44.9

Type of Bridges	Type I Ground Motion	Type II Ground Motion
Type B	3.0	1.5
Type A	2.4	1.2

TABLE 44.5 Modification Factor on Scale Effect for Shear Capacity Shared by Concrete

Effective Width of Section d (m)	Coefficient c_c
$d \leqq 1$	1.0
$d = 3$	0.7
$d = 5$	0.6
$d \geqq 10$	0.5

it is known that a certain level of failure in a pier such as a sudden decrease of lateral capacity occurs at smaller lateral displacement in a pier subjected to a loading hysteresis with a greater number of load reversals. To reflect such a fact, it was decided that the ultimate strain ε_{cu} should be evaluated by Eq. (44.18), depending on the type of ground motions. Therefore, the allowable ductility factor μ_a depends on the type of ground motions; the μ_a is larger in a pier subjected to Type II ground motions than a pier subjected to Type I ground motions.

It should be noted that the safety factor α in Eq. (44.9) depends on the type of bridges as well as the type of ground motions as shown in Table 44.4. This is to preserve higher seismic safety in the important bridges, and to take account of the difference of recurrent time between Type I and Type II ground motions.

44.4.4.3 Shear Capacity

Shear capacity of reinforced concrete piers is evaluated by a conventional method as

$$P_s = S_c + S_s \tag{44.19}$$

$$S_c = c_c c_e c_{pt} \tau_c bd \tag{44.20}$$

$$S_s = \frac{A_w \sigma_{sy} d\,(\sin\theta + \cos\theta)}{1.15a} \tag{44.21}$$

in which P_s = shear capacity; S_c = shear capacity shared by concrete; S_s = shear capacity shared by tie reinforcements, τ_c = shear stress capacity shared by concrete; c_c = modification factor for cyclic loading (0.6 for Type I ground motions; 0.8 for Type II ground motions); c_e = modification factor for scale effect of effective width; c_{pt} = modification factor for longitudinal reinforcement ratio; b and d = width and height of section, A_w = sectional area of tie reinforcement; σ_{sy} = yield strength of tie reinforcement, θ = angle between vertical axis and tie reinforcement, and a = space of tie reinforcement.

The modification factor on the scale effect of effective width, c_e, was based on experimental study of loading tests of beams with various effective heights and was newly introduced in the 1996 Specifications. Table 44.5 shows the modification factor on scale effect.

44.4.4.4 Arrangement of Reinforcement

Figure 44.7 shows a suggested arrangement of tie reinforcement. Tie reinforcement should be deformed bars with a diameter equal or larger than 13 mm, and it should be placed in most bridges

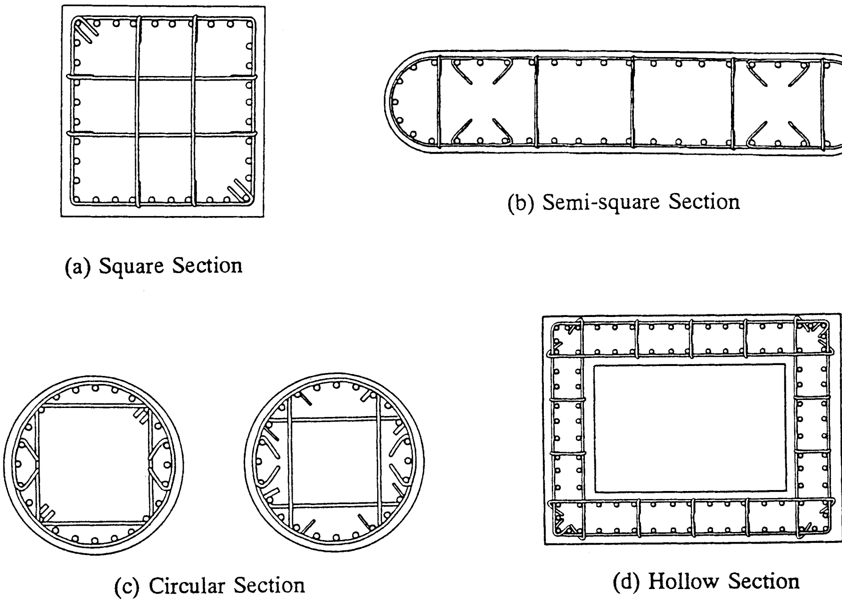


FIGURE 44.7 Confinement of core concrete by tie reinforcement. (a) Square section; (b) semisquare section; (c) circular section; (d) hollow section.

at a distance of no longer than 150 mm. In special cases, such as bridges with pier height taller than 30 m, the distance of tie reinforcement may be increased at height so that pier strength should not be sharply decreased at the section. Intermediate ties should be also provided with the same distance with the ties to confine the concrete. Space of the intermediate ties should be less than 1 m.

44.4.4.5 Two-Column Bent

To determine the ultimate strength and ductility factor for two-column bents, it is modeled as a frame model with plastic hinges at both ends of a lateral cap beam and columns as shown in Figure 44.8. Each elastic frame member has the yield stiffness which is obtained based on the axial load by the dead load of the superstructure and the column. The plastic hinge is assumed to be placed at the end part of a cap beam and the top and bottom part of each column. The plastic hinges are modeled as spring elements with a bilinear moment–curvature relation. The location of plastic hinges is half the distance of the plastic hinge length off from the end edge of each member, where the plastic hinge length L_P is assumed to be Eq. (44.11).

When the two-column bent is subjected to lateral force in the transverse direction, axial force developed in the beam and columns is affected by the applied lateral force. Therefore, the horizontal force–displacement relation is obtained through the static push-over analysis considering axial force N/moment M interaction relation. The ultimate state of each plastic hinge is obtained by the ultimate plastic angle θ_{pu} as

$$\theta_{pu} = \left(\phi_u/\phi_y - 1\right)L_P\phi_y \tag{44.22}$$

in which ϕ_u = ultimate curvature and ϕ_y = yield curvature.

The ultimate state of the whole two-bent column is determined so that all four plastic hinges developed reach the ultimate plastic angle.

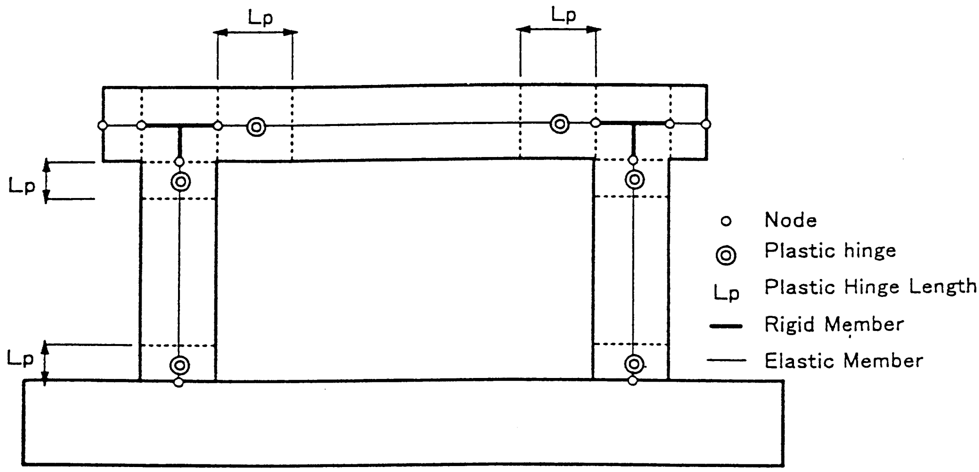


FIGURE 44.8 Analytical idealization of a two-column bent.

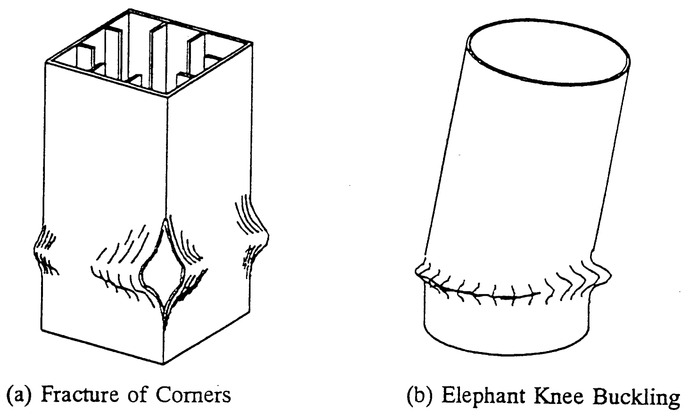


FIGURE 44.9 Typical brittle failure modes of steel piers. (a) Fracture of corners; (b) elephant knee buckling.

44.4.5 Ductility Design of Steel Piers

44.4.5.1 Basic Concept

To improve seismic performance of a steel pier, it is important to avoid specific brittle failure modes. Figure 44.9 shows the typical brittle failure mode for rectangular and circular steel piers. The following are the countermeasures to avoid such brittle failure modes and to improve seismic performance of steel piers:

1. Fill the steel column with concrete.
2. Improve structural parameters related to buckling strength.
   - Decrease the width–thickness ratio of stiffened plates of rectangular piers or the diameter–thickness ratio of steel pipes;
   - Increase the stiffness of stiffeners;
   - Reduce the diaphragm spacing;
   - Strengthen corners using the corner plates;
3. Improve welding section at the corners of rectangular section
4. Eliminate welding section at the corners by using round corners.

44.4.5.2 Concrete-Infilled Steel Pier

In a concrete-infilled steel pier, the lateral capacity P_a and the allowable displacement ductility factor μ_a in Eqs. (44.1) and (44.2) are evaluated as

$$P_a = P_y + \frac{P_u - P_y}{\alpha} \tag{44.23}$$

$$\mu_a = \left(1 + \frac{\delta_u - \delta_y}{\alpha\delta_y}\right)\frac{P_u}{P_a} \tag{44.24}$$

in which P_y and P_u = yield and ultimate lateral capacity of a pier; δ_y and δ_u = yield and ultimate displacement of a pier; and α = safety factor (refer to Table 44.4). The P_a and the μ_a are evaluated idealizing that a concrete-infilled steel pier resists flexural moment and shear force as a reinforced concrete pier. It is assumed in this evaluation that the steel section is idealized as reinforcing bars and that only the steel section resists axial force. A stress vs. strain relation of steel and concrete as shown in Figure 44.10 is assumed. The height of infilled concrete has to be decided so that bucking is not developed above the infilled concrete.

44.4.5.3 Steel Pier without Infilled Concrete

A steel pier without infilled concrete must be designed with dynamic response analysis. Properties of the pier need to be decided based on a cyclic loading test. Arrangement of stiffness and welding at corners must be precisely evaluated so that brittle failure is avoided.

44.4.6 Dynamic Response Analysis

Dynamic response analysis is required in bridges with complex dynamic response to check the safety factor of the static design. Dynamic response analysis is also required as a "design" tool in the bridges for which the ductility design method is not directly applied. In dynamic response analysis, ground motions which are spectral-fitted to the following response spectra are used;

$$S_I = c_Z \cdot c_D \cdot S_{I0} \tag{44.25}$$

$$S_{II} = c_Z \cdot c_D \cdot S_{II0} \tag{44.26}$$

in which S_{I} and S_{II} = acceleration response spectrum for Type I and Type II ground motions; $S_{\mathrm{I}\,0}$ and $S_{\mathrm{II}\,0}$ = standard acceleration response spectrum for Type I and Type II ground motions, respectively; c_Z = modification coefficient for zone, refer to Eq. (44.8); and c_D = modification coefficient for damping ratio given as

$$c_D = \frac{1.5}{40h_i + 1} + 0.5 \tag{44.27}$$

Table 44.6 and Figure 44.11 show the standard acceleration response spectra (damping ratio h = 0.05) for Type I and Type II ground motions.

It is recommended that at least three ground motions be used per analysis and that an average be taken to evaluate the response.

In dynamic analysis, modal damping ratios should be carefully evaluated. To determine the modal damping ratios, a bridge may be divided into several substructures in which the energy-dissipating mechanism is essentially the same. If one can specify a damping ratio of each substructure for a given mode shape, the modal damping ratio for the ith mode, h_i, may be evaluated as

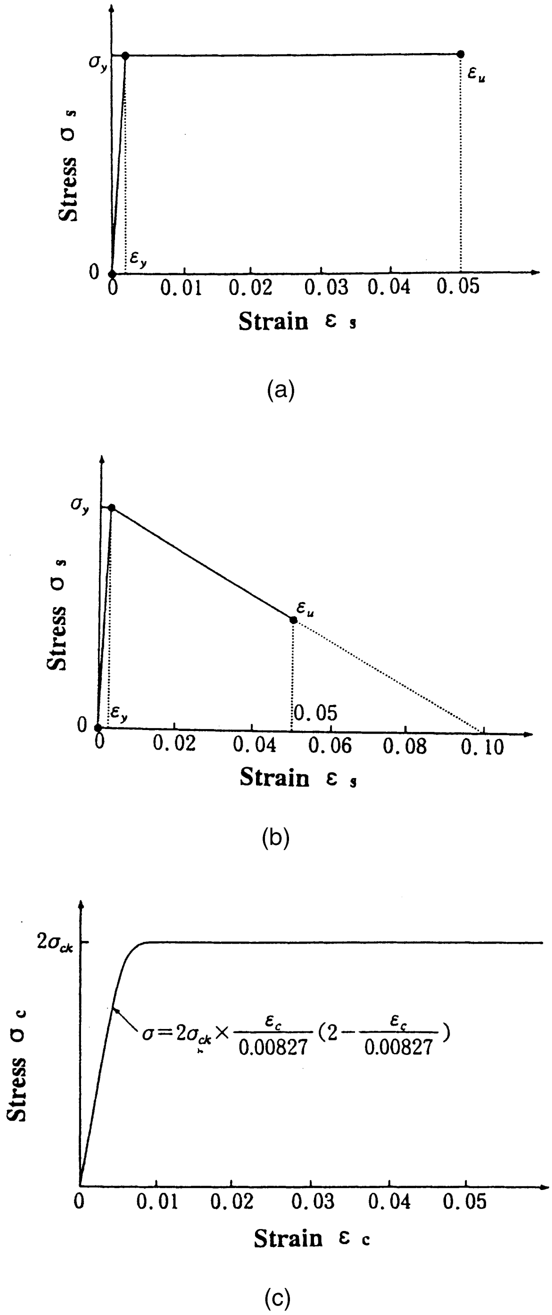


FIGURE 44.10 Stress–strain relation of steel and concrete. (a) Steel (tension); (b) steel (compression); (c) concrete.

$$h_i = \frac{\sum_{j=1}^{n} \phi_{ij}^T \cdot h_{ij} \cdot K_j \cdot \phi_{ij}}{\Phi_i^T \cdot K \cdot \Phi_i} \tag{44.28}$$

TABLE 44.6 Standard Acceleration Response Spectra

Soil Condition	Response Acceleration S_{10} (gal = cm/s^2)		
	Type I Response Spectra S_{10}		
Group I	$S_{10} = 700$ for $T_i = \leqq 1.4$		$S_{10} = 980/T_i$ for $T_i > 1.4$
Group II	$S_{10} = 1505 T_i^{1/3}$ ($S_{10} \geqq 700$) for $T_i < 0.18$	$S_{10} = 850$ for $0.18 \leqq T_i \leqq 1.6$	$S_{10} = 1360/T_i$ for $T_i > 1.6$
Group III	$S_{10} = 1511 T_i^{1/3}$ ($S_{10} \geqq 700$) for $T_i < 0.29$	$S_{10} = 1000$ for $0.29 \leqq T_i \leqq 2.0$	$S_{10} = 2000/T_i$ for $T_i > 2.0$
	Type II Response Spectra S_{110}		
Group I	$S_{110} = 4463 T_i^{2/3}$ for $T_i \leqq 0.3$	$S_{110} = 2000$ for $0.3 \leqq T_i \leqq 0.7$	$S_{110} = 1104/T_i^{5/3}$ for $T_i > 0.7$
Group II	$S_{110} = 3224 T_i^{2/3}$ for $T_i < 0.4$	$S_{110} = 1750$ for $0.4 \leqq T_i \leqq 1.2$	$S_{110} = 2371/T_i^{5/3}$ for $T_i > 1.2$
Group III	$S_{110} = 2381 T_i^{2/3}$ for $T_i < 0.5$	$S_{110} = 1500$ for $0.5 \leqq T_i \leqq 1.5$	$S_{110} = 2948 T_i^{5/3}$ for $T_i > 1.5$

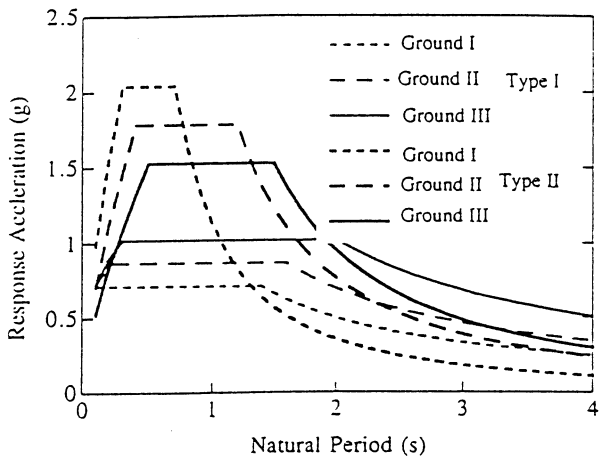


FIGURE 44.11 Type I and Type II standard acceleration response spectra.

TABLE 44.7 Recommended Damping Ratios for Major Structural Components

	Elastic Response		Nonlinear Response	
Structural Components	Steel	Concrete	Steel	Concrete
Superstructure	0.02 ~ 0.03	0.03 ~ 0.05	0.02 ~ 0.03	0.03 ~ 0.05
Rubber bearings	0.02		0.02	
Menshin bearings	Equivalent damping ratio by Eq. 44.26		Equivalent damping ratio by Eq. 44.46	
Substructures	0.03 ~ 0.05	0.05 ~ 0.1	0.1 ~ 0.2	0.12 ~ 0.2
Foundations	0.1 ~ 0.3		0.2 ~ 0.4	

in which h_{ij} = damping ratio of the jth substructure in the ith mode, ϕ_{ij} = mode vector of the jth substructure in the ith mode, k_j = stiffness matrix of the jth substructure, K = stiffness matrix of a bridge, and Φ_i = mode vector of a bridge in the ith mode, which is given as

$$\Phi_i^T = \left\{\phi_{i1}^T, \phi_{i2}^T, \ldots, \phi_{in}^T\right\} \tag{44.29}$$

Table 44.7 shows recommended damping ratios for major structural components.

TABLE 44.8 Modification Coefficient for Energy Dissipation Capability

Damping Ratio for First Mode h	Coefficient c_ε
$h < 0.1$	1.0
$0.1 \leqq h < 0.12$	0.9
$0.12 \leqq h < 0.15$	0.8
$h \geqq 0.15$	0.7

44.4.7 Menshin (Seismic Isolation) Design

44.4.7.1 Basic Principle

Implementation of Menshin bridges should be carefully chosen from the point of view not only of seismic performance but also of function for traffic and maintenance, based on the advantage and disadvantage of increasing natural period. The Menshin design should not be adopted in the following situations:

1. Sites vulnerable to loss of bearing capacity due to soil liquefaction and lateral spreading;
2. Bridges supported by flexible columns;
3. Soft soil sites where potential resonance with surrounding soils could be developed by increasing the fundamental natural period; and
4. Bridges with uplift force at bearings.

It is suggested that the design be made with an emphasis on an increase of energy-dissipating capability and a distribution of lateral force to as many substructures as possible. To concentrate the hysteretic deformation not at piers, but at bearings, the fundamental natural period of a Menshin bridge should be about two times or more longer than the fundamental natural period of the same bridge supported by conventional bearings. It should be noted that an elongation of natural period aiming to decrease the lateral force should not be attempted.

44.4.7.2 Design Procedure

Menshin bridges are designed by both the seismic coefficient method and the ductility design method. In the seismic coefficient method, no reduction of lateral force from the conventional design is made.

In the ductility design method, the equivalent lateral force coefficient k_{hem} in the Menshin design is evaluated as

$$k_{hem} = \frac{k_{hcm}}{\sqrt{2\mu_m - 1}} \tag{44.30}$$

$$k_{hcm} = c_E \cdot k_{hc} \tag{44.31}$$

in which k_{hcm} = lateral force coefficient in Menshin design, μ_m = allowable ductility factor of a pier, c_E = modification coefficient for energy-dissipating capability (refer to Table 44.8), and k_{hc} = lateral force coefficient by Eq. (44.8). Because the k_{hc} is the lateral force coefficient for a bridge supported by conventional bearings, Eq. (44.31) means that the lateral force in the Menshin design can be reduced, as much as 30%, by the modification coefficient c_E depending on the modal damping ratio of a bridge.

Modal damping ratio of a menshin bridge h for the fundamental mode is computed as Eq. (44.32). In Eq. (44.32), h_{Bi} = damping ratio of the ith damper, h_{Pi} = damping ratio of the ith pier or abutment, h_{Fui} = damping ratio of the ith foundation associated with translational displacement, $h_{F\phi i}$ = damping ratio of the *i*th foundation associated with rotational displacement, K_{Pi} = equivalent stiffness of the *i*th pier or abutment, K_{Fui} = translational stiffness of the *i*th foundation, $K_{F\phi i}$ = rotational stiffness of the *i*th foundation, u_{Bi} = design displacement of the *i*th Menshin device, and H = distance from the bottom of a pier to a gravity center of a deck.

In the Menshin design, the allowable displacement ductility factor of a pier μ_m in Eq. (44.30) is evaluated by

$$h = \frac{\sum K_{Bi} \cdot u_{Bi}^2 \left(h_{Bi} + \frac{h_{Pi} \cdot K_{Bi}}{K_{Pi}} + \frac{h_{Fui} \cdot K_{Bi}}{K_{Fui}} + \frac{h_{F\theta i} \cdot K_{Bi} \cdot H^2}{K_{Fei}} \right)}{\sum K_{Bi} \cdot u_{Bi}^2 \left(1 + \frac{K_{Bi}}{K_{Pi}} + \frac{K_{Bi}}{K_{Fui}} + \frac{K_{Bi} \cdot H^2}{K_{F\theta i}} \right)} \tag{44.32}$$

$$\mu_m = 1 + \frac{\delta_u - \delta_y}{\alpha_m \delta_y} \tag{44.33}$$

in which α_m is a safety factor used in Menshin design and is given as

$$\alpha_m = 2\alpha \tag{44.34}$$

where α is the safety factor in the conventional design (refer to Table 44.4). Equation (44.34) means that the allowable displacement ductility factor in the Menshin design μ_m should be smaller than the allowable displacement ductility factor μ_a by Eq. (44.2) in the conventional design. The reason for the smaller allowable ductility factor in the Menshin design is to limit the hysteretic displacement of a pier at the plastic hinge zone so that the principal hysteretic behavior occurs at the Menshin devices, as shown in Figure 44.4b.

44.4.7.2 Design of Menshin Devices

Simple devices that can resist extreme earthquakes must be used. The bearings have to be anchored to a deck and substructures with bolts, and should be replaceable. Clearance has to be provided between a deck and an abutment or between adjacent decks.

Isolators and dampers must be designed for a desired design displacement *uB*. The design displacement u_B is evaluated as

$$u_B = \frac{k_{hem} W_U}{K_B} \tag{44.35}$$

in which k_{hem} = equivalent lateral force coefficient by Eq. (44.31), K_B = equivalent stiffness, and W_U = dead weight of a superstructure. It should be noted that, because the equivalent lateral force coefficient k_{hem} depends on the type of ground motions, the design displacement u_B also depends on the same.

The equivalent stiffness K_B and the equivalent damping ratio h_B of a Menshin device are evaluated as

$$K_B = \frac{F(u_{Be}) - F(-u_{Be})}{2u_{Be}} \tag{44.36}$$

$$h_B = \frac{\Delta W}{2\pi W} \tag{44.37}$$

$$u_{Be} = c_B \cdot u_B \tag{44.38}$$

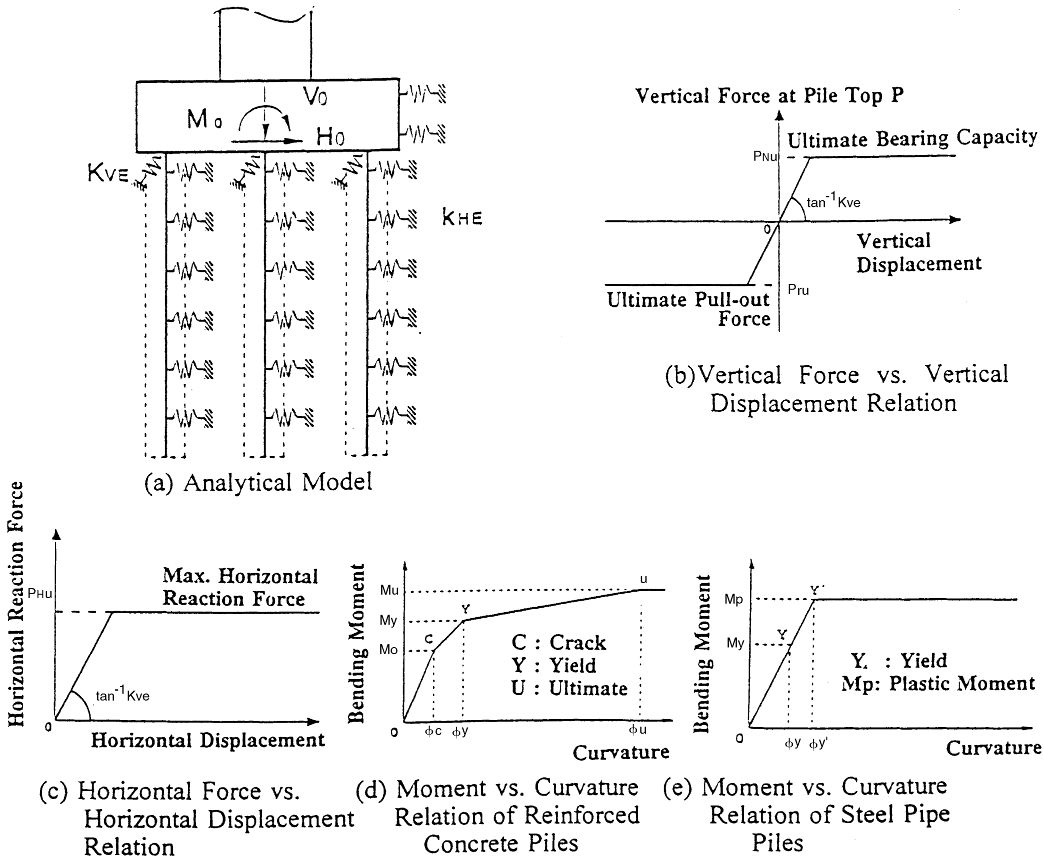


FIGURE 44.12 Idealized nonlinear model of a pile foundation. (a) Analybical model; (b) vertical force vs. vertical displacement relation; (c) horizontal force vs. horizontal displacement relation; (d) moment vs. curvature relation of reinforced concrete piles; (e) moment vs. curvature relation of steel pipe piles.

in which $F(u)$ = restoring force of a device at a displacement u, u_{Be} = effective design displacement, ΔW = energy dissipated per cycle, W = elastic strain energy, and c_B = coefficient to evaluate effective displacement (= 0.7).

44.4.8 Design of Foundations

The evaluation methods of ductility and strength of foundations such as pile foundations and caisson foundations were newly introduced in the 1996 Specifications.

For a pile foundation, a foundation should be so idealized that a rigid footing is supported by piles which are supported by soils. The flexural strength of a pier defined by Eq. (44.7) is to be applied as a seismic force to foundations at the bottom of the footing together with the dead-weight superstructure, pier, and soils on the footing. Figure 44.12 shows the idealized nonlinear model of a pile foundation. The nonlinearity of soils and piles is considered in the analysis.

The safety of the foundation is to be checked so that (1) the foundation does not reach its yield point; (2) if the primary nonlinearity is developed in the foundations, the response displacement is less than the displacement ductility limit; and (3) the displacement developed in the foundation is less than the allowable limit. The allowable ductility and the allowable limit of displacement were noted as 4 in displacement ductility, 40 cm in horizontal displacement, and 0.025 rad in rotation angle.

For a caisson-type foundation, the foundation should be modeled as a reinforced concrete column that is supported by soil spring model; the safety is checked in the same way as the pile foundations.

44.4.9 Design against Soil Liquefaction and Liquefaction~Induced Lateral Spreading

44.4.9.1 Estimation of Liquefaction Potential

Since the Hyogo-ken Nanbu earthquake of 1995 caused liquefaction even at coarse sand or gravel layers which had been regarded as invulnerable to liquefication, a gravel layer was included in the soil layers that require liquefaction potential estimation. Soil layers that satisfy the following conditions are estimated to be potential liquefaction layers:

1. Saturated soil layer which is located within 20 m under the ground surface and in which the groundwater level is less than 10 m deep;
2. Soil layer in which fine particle content ratio FC is equal or less than 35% or the plasticity index I_P is equal to or less than 15;
3. Soil layer in which mean grain size D_{50} is equal or less than 10 mm and 10% grain size D_{10} is equal or less than 1 mm.

Liquefaction potential is estimated by the safety factor against liquefaction F_L as

$$F_L = R/L \tag{44.39}$$

where, F_L = liquefaction resistant ratio, R = dynamic shear strength ratio, and L = shear–stress ratio during an earthquake. The dynamic shear strength ratio R may be expressed as

$$R = c_W R_L \tag{44.40}$$

where c_W = corrective coefficient for ground motion characteristics (1.0 for Type I ground motions, 1.0 to 2.0 for Type II ground motions), and R_L = cyclic triaxial strength ratio. The cyclic triaxial strength ratio was estimated by laboratory tests with undisturbed samples by the *in situ* freezing method.

The shear–stress ratio during an earthquake may be expressed as

$$L = r_d k_{hc} \sigma_v / \sigma_v' \tag{44.41}$$

where r_d = modification factor shear–stress ratio with depth, k_{hc} = design seismic coefficient for the evaluation of liquefaction potential, σ_v = total loading pressure, σ_v' = effective loading pressure.

It should be noted here that the design seismic coefficient for the evaluation of liquefaction potential k_{hc} ranges from 0.3 to 0.4 for Type I ground motions, and from 0.6 to 0.8 for Type II ground motions.

44.4.9.2 Design Treatment of Liquefaction for Bridge Foundations

When liquefaction occurs, the strength and the bearing capacity of a soil decreases. In the seismic design of highway bridges, soil constants of a sandy soil layer which is judged liable to liquefy are reduced according to the F_L value. The reduced soil constants are calculated by multiplying the coefficient D_E in Table 44.9 to the soil constants estimated on an assumption that the soil layer does not liquefy.

44.4.9.3 Design Treatment of Liquefaction-Induced Ground Flow for Bridge Foundations

The influence of liquefaction-induced ground flow was included in the revised Design Specifications in 1996. The case in which ground flow that may affect bridge seismicity is likely to occur is generally that the ground is judged to be liquefiable and is exposed to biased Earth pressure, for example, the ground behind a seaside protection wall. The effect of liquefaction-induced ground flow is

TABLE 44.9 Reduction Coefficient for Soil Constants Due to Soil Liquefaction

Range of F_L	Depth from the Present Ground Surface x (m)	Dynamic Shear Strength Ratio R	
		$R \leqq 0.3$	$0.3 < R$
$F_L \leqq 1/3$	$0 \leqq x \leqq 10$	0	1/6
	$10 < x \leqq 20$	1/3	1/3
$1/3 < F_L \leqq 2/3$	$0 \leqq x \leqq 10$	1/3	2/3
	$10 < x \leqq 20$	2/3	2/3
$2/3 < F_L \leqq 1$	$0 \leqq x \leqq 10$	2/3	1
	$10 < x \leqq 20$	1	1

considered as the static force acting on a structure. This method premises that the surface soil is of the nonliquefiable and liquefiable layers, and the forces equivalent to the passive Earth pressure and 30% of the overburden pressure are applied to the structure in the nonliquefiable layer and liquefiable layer, respectively.

The seismic safety of a foundation is checked by confirming that the displacement at the top of foundation caused by ground flow does not exceed an allowable value, in which a foundation and the ground are idealized as shown in Figure 44.12. The allowable displacement of a foundation may be taken as two times the yield displacement of a foundation. In this process, the inertia force of structure is not necessary to be considered simultaneously, because the liquefaction-induced ground flow may take place after the principal ground motion.

44.4.10 Bearing Supports

The bearings are classified into two groups: Type A bearings resisting the seismic force considered in the seismic coefficient method, and Type B bearings resisting the seismic force of Eq. (44.2). Seismic performance of Type B bearings is, of course, much higher than that of Type A bearings. In Type A bearings, a displacement-limiting device, which will be described later, has to be coinstalled in both longitudinal and transverse directions, while it is not required in Type B bearings. Because of the importance of bearings as one of the main structural components, Type B bearings should be used in Menshin bridges.

The uplift force applied to the bearing supports is specified as

$$R_U = R_D - \sqrt{R_{heq}^2 + R_{veq}^2} \tag{44.42}$$

in which R_U = design uplift force applied to the bearing support, R_D = dead load of superstructure, R_{heq} and R_{veq} are vertical reactions caused by the horizontal seismic force and vertical force, respectively. Figure 44.13 shows the design forces for the bearing supports.

44.4.11 Unseating Prevention Systems

Unseating prevention measures are required for highway bridges. Unseating prevention systems consist of enough seat length, a falling-down prevention device, a displacement-limiting device, and a settlement prevention device. The basic requirements are as follows:

1. The unseating prevention systems have to be so designed that unseating of a superstructure from its supports can be prevented even if unpredictable failures of structural members occur;
2. Enough seat length must be provided and a falling-down prevention device must be installed at the ends of a superstructure against longitudinal response. If Type A bearings are used, a displacement-limiting device has to be further installed at not only the ends of a superstructure but at each intermediate support in a continuous bridge; and

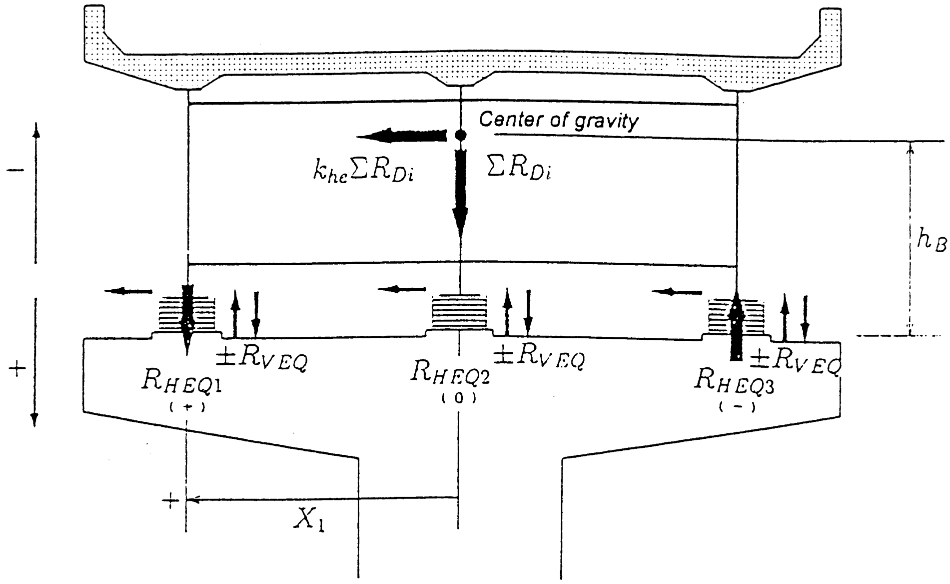


FIGURE 44.13 Design forces for bearing supports.

3. If Type A bearings are used, a displacement-limiting device is required at each support against transverse response. The displacement-limiting device is not generally required if Type B bearings are used. But, even if Type B bearings are adopted, it is required in skewed bridges, curved bridges, bridges supported by columns with narrow crests, bridges supported by few bearings per pier, and bridges constructed at sites vulnerable to lateral spreading associated with soil liquefaction.

The seat length S_E is evaluated as

$$S_E = u_R + u_G \geqq S_{EM} \tag{44.43}$$

$$S_{EM} = 70 + 0.5l \tag{44.44}$$

$$u_G = 100 \cdot \varepsilon_G \cdot L \tag{44.45}$$

in which u_R = relative displacement (cm) developed between a superstructure and a substructure subjected to a seismic force equivalent to the equivalent lateral force coefficient k_{hc} by Eq. (44.2); u_G = relative displacement of ground along the bridge axis; S_{EM} = minimum seat length (cm); ε_G = ground strain induced during an earthquake along the bridge axis, which is 0.0025, 0.00375, and 0.005 for Group I, II, and III sites, respectively; L = distance that contributes to the relative displacement of ground (m); and 1 = span length (m). If two adjacent decks are supported by a pier, the larger span length should be l in evaluating the seat length.

In the Menshin design, in addition to the above requirements, the following considerations have to be made.

1. To prevent collisions between a deck and an abutment or between two adjacent decks, enough clearance must be provided. The clearance between those structural components S_B should be evaluated as

$$S_B = \begin{cases} u_B + L_A & \text{between a deck and an abutment} \\ c_B \cdot u_B + L_A & \text{between two adjacent decks} \end{cases} \tag{44.46}$$

TABLE 44.10 Modification Coefficient for Clearance c_B

$\Delta T/T_1$	c_B
$0 \leqq \Delta T/T_1 < 0.1$	1
$0.1 \leqq \Delta T/T_1 < 0.8$	$\sqrt{2}$
$0.8 \leqq \Delta T/T_1 \leqq 1.0$	1

in which u_B = design displacement of Menshin devices (cm) by Eq. (44.39), L_A = redundancy of a clearance (generally ±1.5 cm), and c_B = modification coefficient for clearance (refer to Table 44.10). The modification coefficient c_B was determined based on an analysis of the relative displacement response spectra. It depends on a difference of natural periods $\Delta T = T_1 - T_2$ ($T_1 > T_2$), in which T_1 and T_2 represent the natural period of the two adjacent bridge systems.

2. The clearance at an expansion joint L_E is evaluated as

$$L_E = u_B + L_A \tag{44.47}$$

in which u_B = design displacement of Menshin devices (cm) by Eq. (44.39), and L_A = redundancy of a clearance (generally ±1.5 cm).

44.5 Seismic Retrofit Practices for Highway Bridges

44.5.1 Past Seismic Retrofit Practices

The Ministry of Construction has conducted seismic evaluations of highway bridges throughout the country five times since 1971 as a part of the comprehensive earthquake disaster prevention measures for highway facilities. Seismic retrofit for vulnerable highway bridges had been successively made based on the seismic evaluations. Table 44.11 shows the history of past seismic evaluations [7,8].

The first seismic evaluation was made in 1971 to promote earthquake disaster prevention measures for highway facilities. The significant damage of highway bridges caused by the 1971 San Fernando earthquake in the United States triggered the seismic evaluation. Highway bridges with span lengths longer than or equal to 5 m on all systems of national expressways and highways were evaluated. Attention was paid to detect deterioration such as cracks of reinforced concrete structures, tilting, sliding, settlement, and scouring of foundations. Approximately 18,000 highway bridges in total were evaluated and approximately 3200 bridges were found to require retrofit.

Following the first, seismic evaluations had been subsequently made in 1976, 1979, 1986, and 1991 with gradually expanding highways and evaluation items. The seismic evaluation in 1986 was made with the increase of social needs to ensure seismic safety of highway traffic after the damage caused by the Urakawa-oki earthquake in 1982 and the Nihon-kai-chubu earthquake in 1983. The highway bridges with span lengths longer than or equal to 15 m on all systems of national expressways, national highways and principal local highways, and overpasses were evaluated. The evaluation items included deterioration, unseating prevention devices, strength of substructures, and stability of foundations. Approximately 40,000 bridges in total were evaluated and approximately 11,800 bridges were found to require retrofit. The latest seismic evaluation was made in 1991. The number of highways to be evaluated has increased from the number evaluated in 1986. Approximately 60,000 bridges in total were evaluated and approximately 18,000 bridges were found to require retrofit. Through a series of seismic retrofit works, approximately 32,000 bridges were retrofitted by the end of 1994.

TABLE 44.11 Past Seismic Evaluations of Highway Bridges

Year	Highways Inspected	Inspection Items	Number of Bridges		
			Inspected	Require Strengthening	Strenghtened
1971	All sections of national expressways and national highways, and sections of others (bridge length ≧ 5m)	1. Deterioration 2. Bearing seat length *S* for bridges supported by bent piles	18,000	3,200	1,500
1976	All sections of national expressways and national highways, and sections of others (Bridge Length ≧ 15m or Overpass Bridges)	1. Deterioration of substructures, bearing supports, and girders/slabs 2. Bearing seat length *S* and devices for preventing falling-off of superstructure	25,000	7,000	2,500
1979	All sections of national expressways, national highways, and principallocal highways, and sections of others (bridge length ≧ 15 m or overpass bridges)	1. Deterioration of substructures and bearing supports 2. Devices for preventing falling-off of superstructure 3. Effect of soil liquefaction 4. Bearing capacity of soils and piles 5. Strength of RC piers 6. Vulnerable foundations (bent pile and RC frame on two independent caisson founcations)	35,000	16,000	13,000
1986	All sections of national expressways, national highways and principal local highways, and sections of others (bridge length ≧ 15 m or overpass bridges)	1. Deterioration of substructures, bearing supports, and concrete girders 2. Devices for preventing falling-off of superstructure 3. Effect of soil liquefaction 4. Strength of RC piers (bottom of piers and termination zone of main reinforcement) 5. Bearing capacity of piles 6. Vulnerable foundations (bent piles and RC frame on two independent caisson foundations)	40,000	11,800	8,000
1991	All sections of national expressways, national highways and principal local highways, and sections of others (bridge length ≧ 15 m or overpass bridges)	1. Deterioration of substructures, bearing supports, and concrete girders 2. Devices for preventing falling-off of superstructure 3. Effect of soil liquifaction 4. Strength of RC piers (piers and termination zone of main reinforcement) 5. Vulnerable foundations (bent piles and RC frame on two independent caisson foundations)	60,000	18,000	7,000 (as of the end of 1994)

Note: Number of bridges inspected, number of bridges that required strengthening, and number of bridges strengthened are approximate numbers.

TABLE 44.12 Application of the Guide Specifications

Types of Roads and Bridges	Double Deckers, Overcrossings on Roads and Railways, Extremely Important Bridges from Disaster Prevention and Road Network	Others
Expressways, urban expressways, designated urban expressway, Honshu–Shikoku Bridges, designated national highways	Apply all items, in principle	Apply all items, in principle
Nondesignated national highways, prefectural roads, city, town, and village roads	Apply all items, in principle	Apply partially, in principle

The seismic evaluations in 1986 and 1991 were made based on a statistical analysis of bridges damaged and undamaged in the past earthquakes [9]. Because the collapse of bridges tends to develop because of excessive relative movement between the superstructure and the substructures and the failure of substructures associated with inadequate strength, the evaluation was made based on both the relative movement and the strength of the substructure.

Emphasis had been placed on installing unseating prevention devices in the past seismic retrofit. Because the installation of the unseating prevention devices was being completed, it had become important to promote strengthening of those substructures with inadequate strength and lateral stiffness.

44.5.2 Seismic Retrofit after the Hyogo-ken Nanbu Earthquake

44.5.2.1 Reference for Applying Guide Specifications to New Highway Bridges and Seismic Retrofit of Existing Highway Bridges

After the 1995 Hyogo-ken Nanbu earthquake, the "Part V: Seismic Design" of the "Design Specifications of Highway Bridges" (Japan Road Association) was completely revised in 1996 as discussed in the previous sections.

Because most of the substructures designed and constructed before 1971 do not meet the current seismic requirements, it is urgently needed to study the level of seismic vulnerability requiring retrofit. Upgrading the reliability of predictions of possible failure modes in future earthquakes is also very important. Since the seismic retrofit of substructures requires more cost, it is necessary to develop and implement effective and inexpensive retrofit measures and to design methods to provide for the next event.

For increasing seismic safety of the highway bridges that suffered damage by the Hyogo-ken Nanbu earthquake, various new drastic changes were tentatively introduced in the "Guide Specifications for Reconstruction and Repair of Highway Bridges Which Suffered Damage Due to the Hyogo-ken Nanbu Earthquake." Although intensified review of design could be made when it was applied to the bridges only in the Hanshin area, it may not be so easy for field design engineers to follow up the new Guide Specifications when the Guide Specifications is used for seismic design of all new highway bridges and seismic strengthening of existing highway bridges. Based on such demand, the "Reference for Applying the Guide Specifications to New Bridges and Seismic Strengthening of Existing Bridges" [10] was issued on June 30, 1995 by the Sub-Committee for Seismic Countermeasures for Highway Bridges, Japan Road Association.

The Reference classified the application of the Guide Specifications as shown in Table 44.12 based on the importance of the roads. All items of the Guide Specifications are applied for bridges on extremely important roads, while some items which prevent brittle failure of structural components are applied for bridges on important roads. For example, for bridges on important roads, the items for Menshin design, tie reinforcements, termination of longitudinal reinforcements, type of bearings, unseating prevention devices and countermeasures for soil liquefaction are applied, while the remaining items such as the design force, concrete-infilled steel bridges, and ductility check for foundations, are not applied.

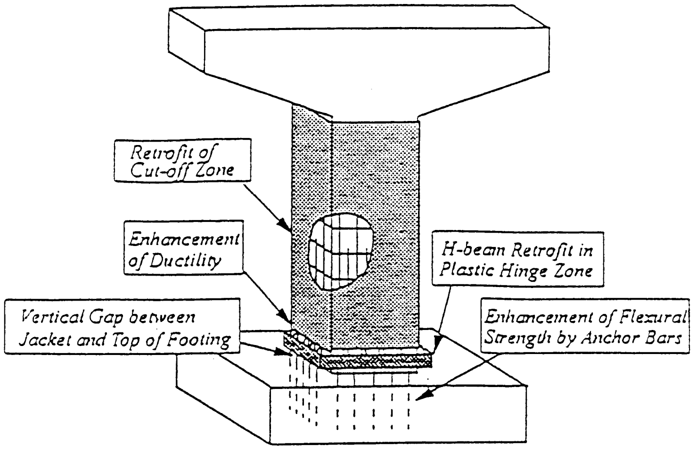


FIGURE 44.14 Seismic retrofit of reinforced concrete piers by steel jacket with controlled increase of flexural strength.

Because damage was concentrated in single reinforced concrete piers/columns with small concrete sections, a seismic retrofit program has been initiated for those columns that were designed according to the pre-1980 Design Specifications, at extremely important bridges such as bridges on expressways, urban expressways, and designated highway bridges, and also double-deckers and overcrossings, etc. which significantly affect highway functions once damaged. In the 3-year program, approximately 30,000 piers will be evaluated and retrofitted. Unseating devices also should be installed for these extremely important bridges.

The main purpose of the seismic retrofit of reinforced concrete columns is to increase their shear strength, in particular in piers with termination of longitudinal reinforcements without enough anchoring length. This increases the ductility of columns, because premature shear failure can be avoided.

However, if only ductility of piers is increased, residual displacement developed at piers after an earthquake may increase. Therefore, the flexural strength should also be increased. However, the increase of flexural strength of piers tends to increase the seismic force transferred from the piers to the foundations. It was found from an analysis of various types of foundations that failure of the foundations by increasing the seismic force may not be significant if the increasing rate of the flexural strength of piers is less than two. It is therefore suggested to increase the flexural strength of piers within this limit so that it does not cause serious damage to foundations.

For such requirements, seismic strengthening by steel jackets with controlled increase of flexural strength was suggested [10, 11]. This uses a steel jacket surrounding the existing columns as shown in Figure 44.14. Epoxy resin or nonshrinkage concrete mortar are injected between the concrete surface and the steel jacket. A small gap is provided at the bottom of piers between the steel jacket and the top of the footing. This prevents excessive increase in the flexural strength.

To increase the flexural strength of columns in a controlled manner, anchor bolts are provided at the bottom of the steel jacket. They are drilled into the footing. By selecting an appropriate number and size of the anchor bolts, the degree of increase of the flexural strength of piers may be controlled. The gap is required to trigger the flexural failure at the bottom of columns. A series of loading tests are being conducted at the Public Works Research Institute to check the appropriate gap and number of anchor bolts. Table 44.13 shows a tentatively suggested thickness of steel jackets and size and number of anchor bolts. They are for reinforced concrete columns with *a/b* less than 3, in which *a* and *b* represent the width of a column in transverse and longitudinal direction, respectively. The size and number of anchor bolts were evaluated so that the increasing rate of flexural strength of columns is less than about 2.

TABLE 44.13 Tentative Retrofit Method by Steel Jacketing

Column/Piers	Steel Jackets	Anchor Bolts
$a/b \leqq 2$	SM400, $t = 9$ mm	SD295, D35 ctc 250 mm
$2 < a/b \leqq 3$ Column supporting lateral force of a continuous girder through fixed bearing and with $a/b \leqq 3$	SM400, $t = 12$ mm	

Conventional reinforced concrete jacketing methods are also applied for the retrofit of reinforced concrete piers, especially for piers that require an increase of strength. It should be noted here that the increase of the strength of the pier should be carefully designed in consideration with the strength of foundations and footings.

44.5.2.2 Research and Development on Seismic Evaluation and Retrofit of Highway Bridges

Prioritization Concept for Seismic Evaluation

The 3-year retrofit program was completed in the 1997 fiscal year. In the program, the single reinforced concrete piers/columns with small concrete section which were designed by the pre-1980 Design Specifications on important highways have been evaluated and retrofitted and other bridges with wall-type piers, steel piers, and frame piers, and so on, as well as the bridges on the other highways, should be evaluated and retrofitted if required in the next retrofit program. Since there are approximately 200,000 piers, it is required to develop prioritization methods and methods to evaluate vulnerability for the intentional retrofit program.

Figure 44.15 shows the simple flowchart to prioritize the retrofit work to bridges. The importance of the highway, structural factors, member vulnerability (reinforced concrete piers, steel piers, unseating prevention devices, foundations) are the factors to be considered for prioritization.

Priority R of each bridge may be evaluated by Eq. (44.48).

$$R = I \cdot S \cdot V_T \cdot w_v \cdot \left[f\left(V_{RP1}, V_{RP2}, V_{RP3}\right), V_{MP}, V_{FS}, V_F \right] \times 100 \tag{44.48}$$

$$f\left(V_{RP1}, V_{RP2}, V_{RP3}\right) = V_{RP1} \cdot V_{RP2} \cdot V_{RP3} \tag{44.49}$$

in which R = priority, I = importance factor, S = earthquake force, V_T = structural factor, w_v = weighting factor on structural members, V_{RP1} = design specification, V_{RP2} pier structural factor, V_{RP3} = aspect ratio, V_{MP} = steel pier factor, V_{FS} = unseating device factor, and V_F = foundation factor. Each item and category with a weighting number is tentatively shown in Table 44.14. If this prioritization method is to applied the bridges damaged during the Hyogo-ken Nanbu earthquake, the categorization number is given as shown in Table 44.14.

Seismic Retrofit of Wall-Type Piers

The steel-jacketing method as described in the above was applied for reinforced concrete with circular section or rectangular section of $a/b < 3$. It is required to develop the seismic retrofit method for a wall-type pier. The confinement of concrete was provided by a confinement beam such as the H-shaped steel beam for rectangular piers. However, since the size of the confinement beam becomes very large, the confinement may be provided by other measures, such as intermediate anchors for a wall-type pier.

The seismic retrofit concept for a wall-type pier is the same as that for rectangular piers. It is important to increase the flexural strength and ductility capacity with the appropriate balance. Generally, the longitudinal reinforcement ratio is smaller than that for rectangular piers; therefore, the flexural strength is smaller. Thus, it is essential to increase the flexural strength appropriately. Since the longitudinal

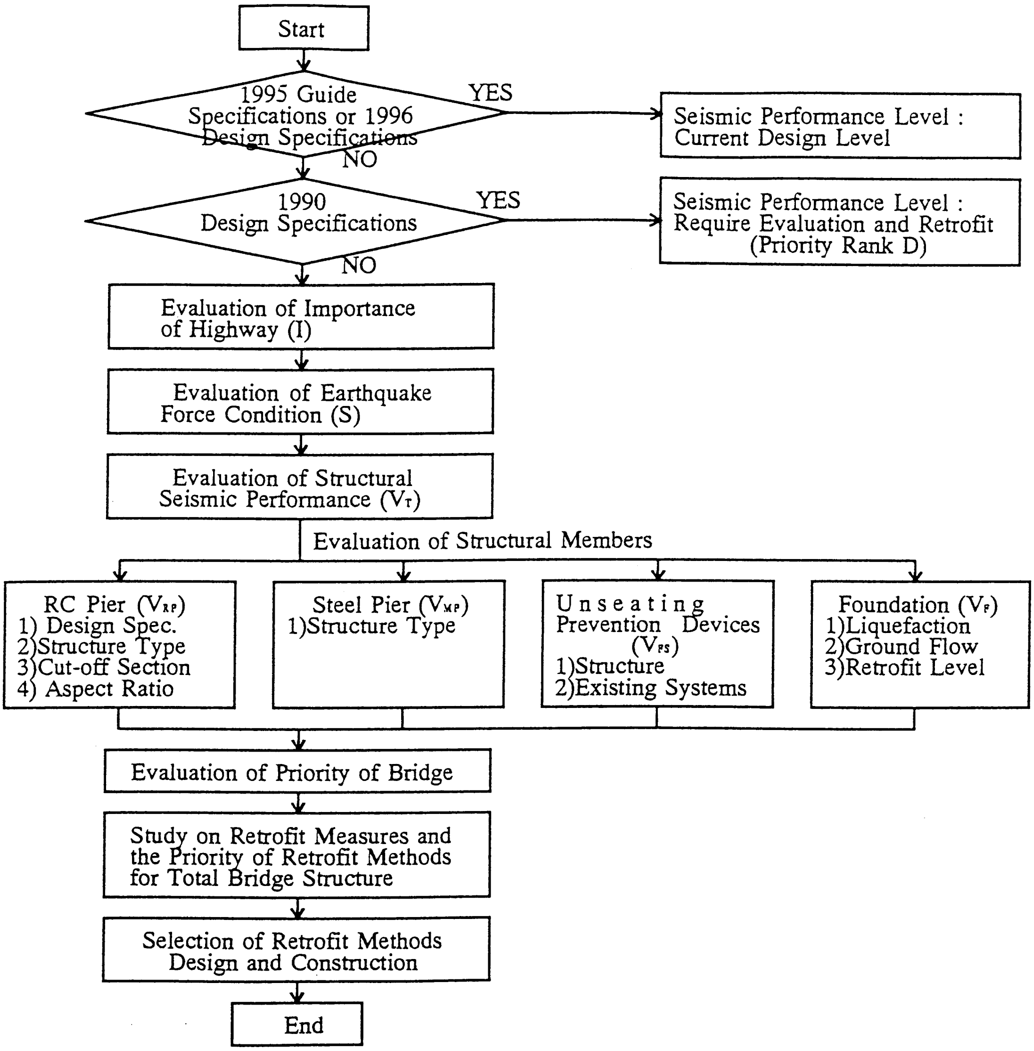


FIGURE 44.15 Prioritization concept of seismic retrofit of highway bridges.

reinforcement was generally terminated at midheight without appropriate anchorage length, it is also important to strengthen both the flexural and shear strength midheight section.

Figure 44.16 shows the possible seismic retrofit method for wall-type piers. To increase the flexural strength, the additional reinforcement by rebars or anchor bars are fixed to the footing. The number of reinforcements is designed to give the necessary flexural strength. It should be noted here that anchoring of additional longitudinal reinforcement is controlled to develop plastic hinge to the bottom of pier rather than the midheight section with termination of longitudinal reinforcement. And the increase of strength should be carefully designed considering the effect on the foundations and footings. The confinement in the plastic hinge zone is provided by steel bars for prestressed concrete or rebars which were installed inside of the column section.

Seismic Retrofit of Two-Column Bents

During the Hyogo-ken Nanbu earthquake, some two-column bents were damaged in the longitudinal and transverse directions. The strength and ductility characteristics of the two-column bents have been studied and the analysis and design method was introduced in the 1996 Design Specifications [12].

TABLE 44.14 Example of Prioritization Factors for Seismic Retrofit of Highway Bridges

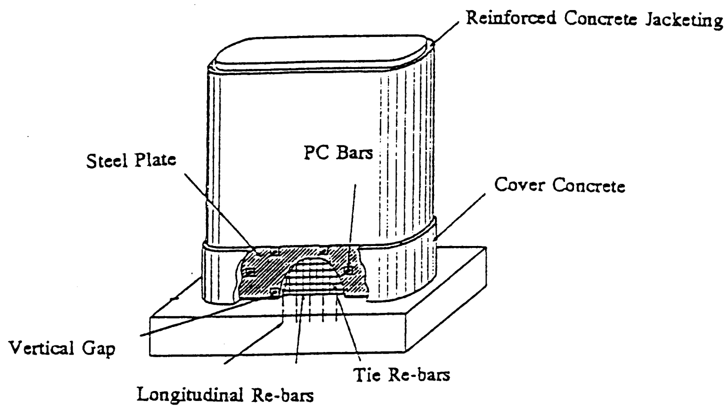
Item	Category	Evaluation Point
Importance of highway (I)	1. Emergency routes	1.0
	2. Overcrossing with emergency routes	0.9
	3. Others	0.6
Earthquake force (S)	1. Ground condition Type I	1.0
	2. Ground condition Type II	0.9
	3. Ground condition Type III	0.8
Structural factor (V_τ)	1. Viaducts	1.0
	2. Supported by abutments at both ends	0.5
Weighting factor on structural members (V_τ)	1. Reinforced concrete pier	1.0
	2. Steel pier	0.95
	3. Unseating prevention devices	0.9
	4. Foundation	0.8
Reinforced concrete pier 1. Design specification (V_{RP1})	1. Pre-1980 Design Specifications	1.0
	2. Post-1980 Design Specifications	0.7
2. Pier structure (V_{RP2})	1. Single column	1.0
	2. Wall-type column	0.8
	3. Two-column bent	0.7
3. Aspect ratio (V_{RP3})	1. h/D ≦ 3	1.0
	2. 3 < h/D < 4 with cutoff section	0.9
	3. H/D ≧ 4 with cutoff section	0.9
	4. $3 < h/D < 4$ without cutoff section	0.7
	5. $H/D \geqq 4$ without cutoff section	0.7
Steel pier (V_{MP})	1. Single column	1.0
	2. Frame structure	0.8
Unseating prevention devices (V_{FS})	1. Without unseating devices	1.0
	2. With one device	0.9
	3. With two devices	0.8
Foundations (V_F)	1. Vulnerable to Ground Flow (without unseating devices)	1.0
	2. Vulnerable to Ground Flow	0.9
	3. Vulnerable to Liquefaction (without unseating devices)	0.7
	4. Vulnerable to Liquefaction	0.6
Evaluation of the priority R	1. $R \geqq 0.8$	Priority Rank A
	2. $0.7 \leqq R < 0.8$	Priority Rank B
	3. $R < 0.7$	Priority Rank C

The strength and ductility of existing two-column bents were studied both in the longitudinal and transverse directions. In the longitudinal direction, the same as a single column, it is required to increase the flexural strength and ductility with appropriate balance. In the transverse direction, the shear strength of the columns or the cap beam is generally not enough in comparison with the flexural strength.

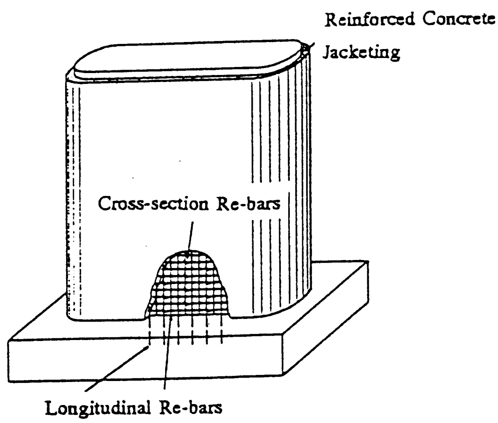
Figure 44.17 shows the possible seismic retrofit methods for two-column bents. The concept of the retrofit is to increase flexural strength and ductility as well as shear capacity for columns and cap beams. Since axial force in the cap beam is much smaller than that in the columns, increasing the shear capacity is essential for the retrofit of the cap beam. It should be noted that since the jacketing of cap beam is difficult because of the existing bearing supports and construction space, it is required to develop more effective retrofit measures for cap beams such as application of jacketing by new materials with high modulus of elasticity and high strength and out-cable pre-stressing, etc.

Seismic Retrofit Using New Materials

Retrofit work is often restricted because construction space is limited to open the structure for public traffic, particularly for the seismic retrofit of highway bridges in urban areas [13]. Therefore, there are sites where conventional steel jacketing and reinforced concrete jacketing methods are



(a)

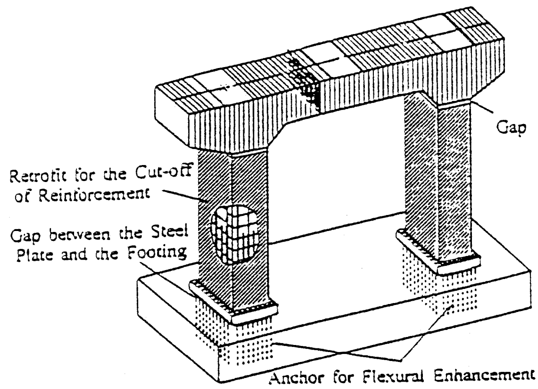


(b)

FIGURE 44.16 Seismic retrofit of wall-type piers. (a) Integrated seismic retrofit method with reinforced concrete and steel jacketing; (b) reinforced concrete jacketing.

difficult to apply. New materials such as carbon fiber sheets and aramid fiber sheets are attractive for application in the seismic retrofit of such bridges with construction restrictions as shown in Figure 44.18. The new materials such as fiber sheets are very light, do not need machines for use, and are easy to construct using glue bond as epoxy resin.

There are various studies on seismic retrofit methods using fiber sheets. Figure 44.19 shows the cooperative effect between fiber sheets and reinforcement for shear strengthening of a single reinforced concrete column. When carbon fiber sheets, which have almost the same elasticity and 10 times the failure strength as those of a reinforcing bar, are assumed to be applied, it is important to design the effects of carbon fiber sheets to achieve the required performance of seismic retrofit. In particular, strengthening of flexural, shear capacities, and ductility for reinforced concrete columns should be carefully evaluated. Based on experimental studies, it is essential to evaluate appropriately the effect of materials on the strengthening, carefully considering the material properties such as the modulus of elasticity and strength.



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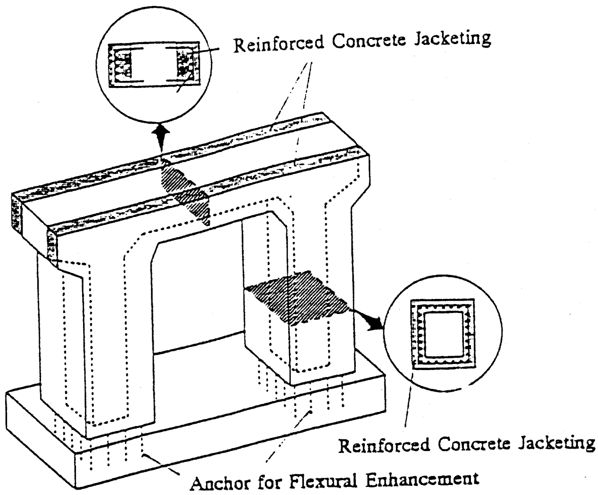

(b)

FIGURE 44.17 Seismic retrofit of two-column bents. (a) Steel jacketing; (b) reinforced concrete jacketing.

Acknowledgments

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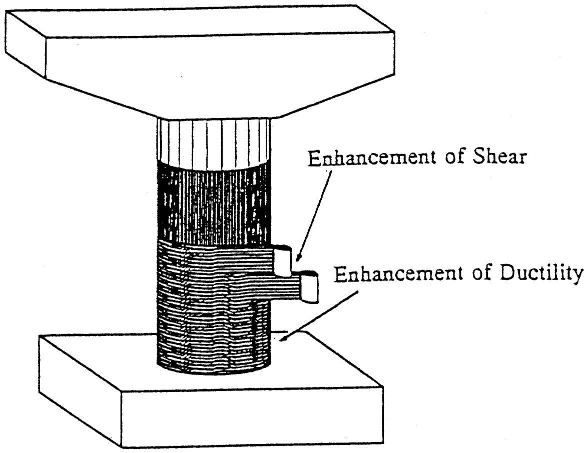

FIGURE 44.18 Application to new materials for seismic retrofit of reinforced column.

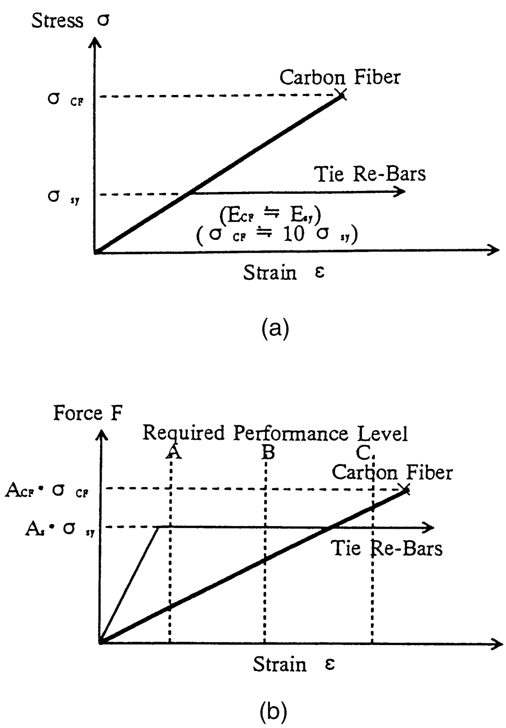

FIGURE 44.19 Cooperative effect between tie reinforcement and carbon fiber sheets. (a) Stress–strain relation; (b) force–strain relation.

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