Performance of steel bridges during the 1995 Hyogo-ken Nanbu (Kobe, Japan) earthquake

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Abstract: A large number of steel bridges were damaged by the January 17, 1995, Hyogo-ken Nanbu (Kobe, Japan) earthquake. The concentration of steel bridges in the area of severe shaking was considerably larger than for any previous earthquake this century. As a result, this earthquake has provided a unique opportunity to examine how steel bridges of various designs and configurations behave when subjected to severe ground shaking. In this paper, a description of the Japanese past and current bridge design requirements is first presented, followed by an in-depth overview of the observed damage to steel bridges. The relevance of these observations to the Canadian bridge design practice is also reviewed.

Key words: earthquake, seismic, steel, bridges, steel columns, buckling, brittle fractures, bearing failures, seismic restrainers, design codes.

Résumé : De nombreux ponts en acier ont été endommagés par le tremblement de terre Hyogo-ken Nanbu (Kobe) survenu le 17 janvier 1995. La concentration des ponts d'acier dans la zone de vibration sévère était plus élevée que lors de n'importe quel autre tremblement de terre précédent. Par conséquent, ce tremblement de terre a offert une occasion unique pour examiner le comportement des ponts d'acier ayant divers designs et configurations lorsque soumis à de sévères tremblements de terre. Dans cet article, une description des exigences présentes et passées du code Japonais sur les ponts est présentée, suivie par une profonde vue d'ensemble de l'endommagement observé des ponts d'acier. La pertinence de ces observations pour le code Canadien sur les ponts est aussi discutée.

Mots clés : tremblement de terre, séismique, acier, ponts, poteaux d'acier, flambement, ruptures fragiles, rupture d'appui, retenus séismiques, codes de design.
[Traduit par la rédaction]

1. Introduction

The January 17, 1995, Hyogo-ken Nanbu earthquake struck Kobe, a highly developed and congested modern city in a country well known for its leading activities in earthquake engineering. Still, in spite of Japan's high level of earthquake awareness, extensive damage was suffered by numerous reinforced concrete and steel bridges in the area of severe shaking. As a result, all major roads and railways crossing Kobe were closed because of damaged or collapsed bridges. This disturbing outcome has nonetheless provided a unique opportunity for the Japanese, as well as worldwide observers, to review their state-of-practice in earthquake-resistant design of bridges. This is particularly true for steel bridges, as the concentration of steel bridges in the area of severe shaking was considerably larger than for any previous earthquake in recorded history. Indeed, steel superstructures and steel box-section piers have been extensively used in the construction of the expressways which weave their way through the particularly dense Japanese urban landscape; for example, roughly 10% of all piers supporting elevated expressways in Kobe were made of steel. Damage was suffered by many of these steel piers, as well as by bearings, seismic restrainers, and superstructure components, and some spectacular collapses resulted from this damage. Many important lessons can be learned from this damage, and the inadequacy of numerous details to provide a reliable ductile seismic response has been exposed by this earthquake.

In this paper, a description of the Japanese past and current bridge design requirements is first presented, followed by an in-depth review of the observed damage to steel bridges along with technical descriptions of the causes for this damage. The relevance of these observations to Canadian bridge design practice is then examined. This paper is part of a concerted multipaper reporting effort by a reconnaissance team of the Canadian Association for Earthquake Engineering which visited the Kobe area. Hence, seismological and geotechnical considerations, which are addressed thoroughly by others, are beyond the scope of this paper.

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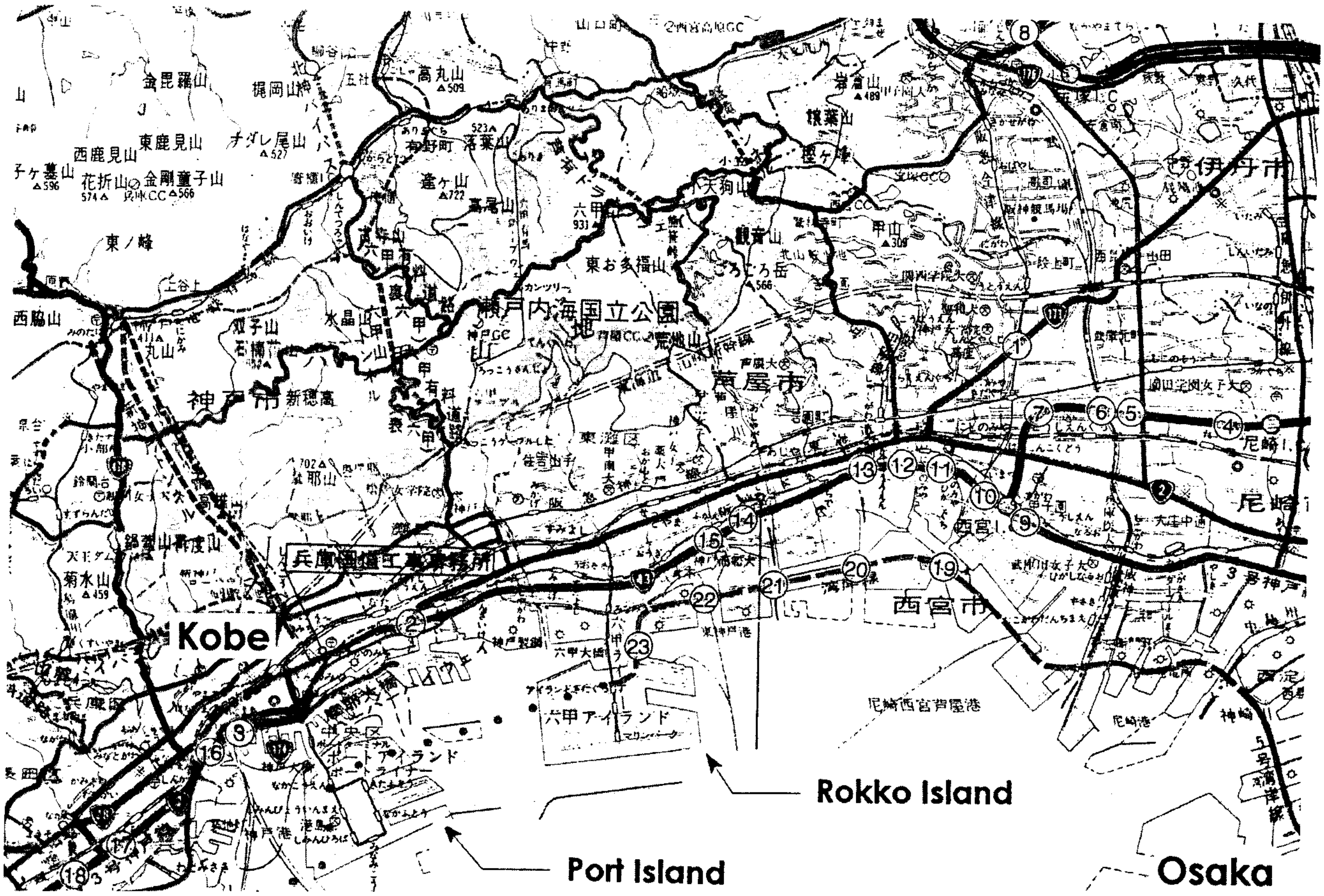
2. Description of transportation network

Kobe is a city constructed at the narrowest point along a band of land located between the Rokko mountains and Osaka

Fig. 1. Locations of severe bridge damage or collapses. Points 9 to 15, Hanshin Expressway; points 19 to 23, Wangan Route 5; point 19, Nishinomiya Port Bridge; point 22, Higashi–Kobe bridge; point 23, Rokko Island Bridge (from Hyogo-ken Nanbu Earthquake Committee 1995).



Bay. Nearly all major land transportation routes between western Japan and Osaka pass through Kobe. The extensive damage to that transportation network, and its lack of redundancy, severely affected the overall deployment of emergency and rescue vehicles in the affected area. As rail service was inoperative, and traffic was stalled over many kilometres, both the evacuation activities and the delivery of vital supplies were exceedingly difficult.

All three major highways of Kobe suffered a tremendous amount of structural damage from the earthquake, as will be described in later sections, and were closed to traffic.

The most severely damaged was the Hanshin Expressway (Route 3), an elevated highway built in the mid-1960s which crosses Kobe and the surrounding cities. For most of its length, the superstructure consists of concrete slabs and steel (I or box) girders, spanning approximately 30 m and supported by approximately 11 m tall single columns with cantilever header beams or portal frames. The authors' observation is that the 625 m long segment which spectacularly collapsed onto its side was the only portion having an entirely reinforced concrete superstructure. Steel or concrete columns seem to have been used interchangeably along the expressway, without any systematic pattern that can be understood by simple observation. All concrete columns had light transverse reinforcement. All steel columns were box-columns built up using slender stiffened welded plates. The highway is relatively near the seashore and located on soft soils. Thus, columns are founded on cast-in-place concrete piles, typically 16 piles, 30 m long with 11 m by 10 m pile caps.

The Harbour Highway is the other elevated expressway of similar vintage and construction which suffered significant damage. This highway is essentially a short trunk line that runs along Kobe Port, connecting the Hanshin Expressway (near Rokko Island) and the Kobe Bridge to Port Island.

The third highway is Wangan Route 5, a part of the Hanshin Expressway system and the newest toll route connecting Kobe and Osaka. Most of that route extends across a number of artificially created islands along the coast, and was opened to full operation in April 1994. However, it is mostly the long-span bridges of that highway that sustained damage during the earthquake, and the description of their damage will be presented in a separate section. A map of this highway, also identifying some of the long-span bridges that were damaged, is shown in Fig. 1.

Ironically, following the January 17, 1994, Californian Northridge earthquake which damaged many bridges, Japan's Ministry of Construction indicated that elevated expressways in Japan could withstand earthquakes as large as the magnitude 7.9 Great Kanto (Tokyo) earthquake of 1923 because they were constructed using more reinforcement steel, thicker piers, and stringent seismic-resistant bridge design codes and quality control (Asahi 1995). Exactly one year

Table 1. Edition of bridge design code used for the design of the bridges that suffered damage during the Hyogo-ken Nanbu earthquake.

Road number	Edition year of bridge design specification				
	1964 or older	1971	1980	1990	Total
National Highway 2	91 (16%)	58 (39%)			149 (100%)
National Highway 43	129 (97%)	4 (3%)			133 (100%)
National Highway 171	162 (100%)				162 (100%)
National Highway 176	24 (89%)	3 (11%)			27 (100%)
Hanshin Route 3	971 (83%)	204 (17%)			1175 (100%)
Hanshin Route 5			308 (84%)	58 (16%)	366 (100%)
JH Meishin	1099 (100%)				1099 (100%)
JH Chugoku	574 (100%)				574 (100%)
Total	3050 (83%)	269 (7%)	308 (8%)	58 (2%)	3685 (100%)

later, the Hyogo-ken Nanbu earthquake brutally deflated that illusion.

Finally, many railroad bridges experienced damage or collapsed during this earthquake. Most of these bridges in the Kobe area had their superstructures supported by concrete columns, but a few were found to have steel columns. While the damage to the concrete bridges was extensive, some steel railroad bridges also suffered noteworthy damage.

Following the earthquake, the Ministry of Construction of Japan surveyed 4449 highway bridge piers and nearly 5000 superstructure spans in Kobe and six surrounding cities in the earthquake stricken area; 3685 of those piers were rated as having sustained minor to severe damage, and the survey reported important damage to superstructure. Statistics compiled from that effort are presented in Tables 1–3 (translated from Hyogo-ken Nanbu Earthquake Committee ... 1995). Table 1 illustrates that although most of the damaged bridges were designed according to older editions of the bridge design code, numerous new bridges also suffered damage. Some evidence that the newer bridges performed better than the older ones is provided by comparing the severity of damage to the bridges along the older Hanshin Expressway Route 3 versus those along the newer Route 5 (Table 2). Although both expressways were likely not subjected to similar ground excitations, they were located at most within a few kilometres from each other and both located on relatively soft soils. Statistics on superstructure damage are presented in Table 3. For the purpose of Tables 2 and 3, some damage indices are subjectively defined as follows:

A_S: Collapse, extensive damage, lost bearing capacity of piers.

A: Extensive cracking, fracture, rupture, severe local buckling of steel piers; local buckling of main reinforcement in concrete piers; major damage of main superstructure members with regards to loss of bearing capacities, such as rupture of lower flange in steel girders and extensive drop-off of concrete girders.

B: Local buckling of web and flanges of steel piers; local buckling of main reinforcement and large cracks in reinforced concrete piers; moderate damage of main superstructure members with regards to loss of bearing capacities, such as deformation of lower flange in steel girders and large cracks in concrete girders.

C: Residual deformations of webs and flanges of steel piers; drop-off of cover concrete and slight cracks in reinforced concrete piers; damage of secondary superstructure members.

D: Minor damage.

As demonstrated quantitatively in Tables 2 and 3 for the Hanshin Expressway alone, damage to bridge structures was very extensive. To better understand the causes for this damage, it is worthwhile to first review the past and current Japanese seismic-resistant design practice. However, only information relevant to steel bridges is presented to remain within the scope of this paper.

3. Past and current design requirements

3.1. Evolution of design philosophy and force levels

In 1926, shortly following the devastating 1923 magnitude 7.9 Kanto (Tokyo) earthquake that damaged 1785 bridges, the Japanese Ministry of Internal Affairs issued *Details of Road Structures (Draft)*, the first Japanese design document that prescribed seismic-resistant design requirements. It specified that earthquake lateral forces of 15% to 40% of a bridge self-weight (depending on location and ground condition) be considered during design, introducing the concept of "seismic coefficient," k. Based on experience in 1923, a minimum k of 0.3 was recommended for Tokyo and Yokohama. Since that time the evolution of earthquake design requirements has been largely driven by observed damage in major earthquakes, with no fewer than eight major damaging earthquakes ($M > 7$) occurring in Japan since 1923. In several of these earthquakes, new types of damage emerged as the implementation of design standards successfully dealt with earlier problems. In 1939, the horizontal seismic coefficient was changed to $k_h = 0.2$, and a vertical coefficient $k_v = 0.1$ was introduced. In 1956, the horizontal coefficient was again changed to be dependent upon location and ground condition, with values ranging from 0.1 to 0.35 (but the vertical coefficient disappeared). Clearly, the magnitude of the seismic coefficient fluctuated somewhat over time and subsequent editions of bridge design codes, its maximum value in the most severe earthquake zones of Japan (which includes Kobe) varying between 20% and 35% before 1971 (this is particularly important given that construction of the network of Japanese expressways began in the 1960s).

While seismic-related lateral strength requirements for

Table 2. Damage indices to routes 3 and 5 of Hanshin Expressways.

Type of piers	Damage indices A_s	A	B	C	D	Total
(a) Hanshin Expressway Route 3 (Kobe)						
Steel piers						
Single column	2 (4%)	8 (15%)	3 (6%)	32 (60%)	8 (15%)	53 (100%)
Others	1 (1%)		9 (8%)	80 (73%)	20 (18%)	110 (100%)
Subtotal	3 (3%)	8 (15%)	12 (7%)	112 (69%)	28 (17%)	163 (100%)
Reinforced concrete piers						
Single column	60 (7%)	83 (10%)	94 (12%)	213 (27%)	352 (44%)	802 (100%)
Others	5 (2%)	1 (0%)	13 (6%)	33 (16%)	158 (75%)	210 (100%)
Subtotal	65 (6%)	84 (8%)	107 (11%)	246 (24%)	510 (50%)	1012 (100%)
(b) Hanshin Expressway Route 5 (Bayshore)						
Steel piers						
Single column					6 (100%)	6 (100%)
Others			13 (9%)	21 (15%)	103 (75%)	137 (100%)
Subtotal			13 (9%)	21 (15%)	109 (76%)	143 (100%)
Reinforced concrete piers						
Single column			1 (1%)	2 (2%)	99 (97%)	102 (100%)
Others				20 (17%)	101 (83%)	121 (100%)
Subtotal			1 (1%)	22 (10%)	200 (90%)	223 (100%)

Table 3. Superstructure damage to Hanshin Expressway.

Superstructure component	Damage indices A_s	A	B	C	D	Total
Bearings	0	112 (7%)	142 (8%)	62 (4%)	1364 (81%)	1680 (100%)
Seismic restrainers	0	4 (0%)	24 (2%)	38 (3%)	1317 (95%)	1382 (100%)
Deck	0	1 (0%)	5 (0%)	2 (0%)	1672 (100%)	1680 (100%)
Others	0	26 (14%)	29 (16%)	12 (6%)	120 (64%)	187 (100%)
Total	0	143 (3%)	200 (4%)	114 (2%)	4473 (91%)	4930 (100%)

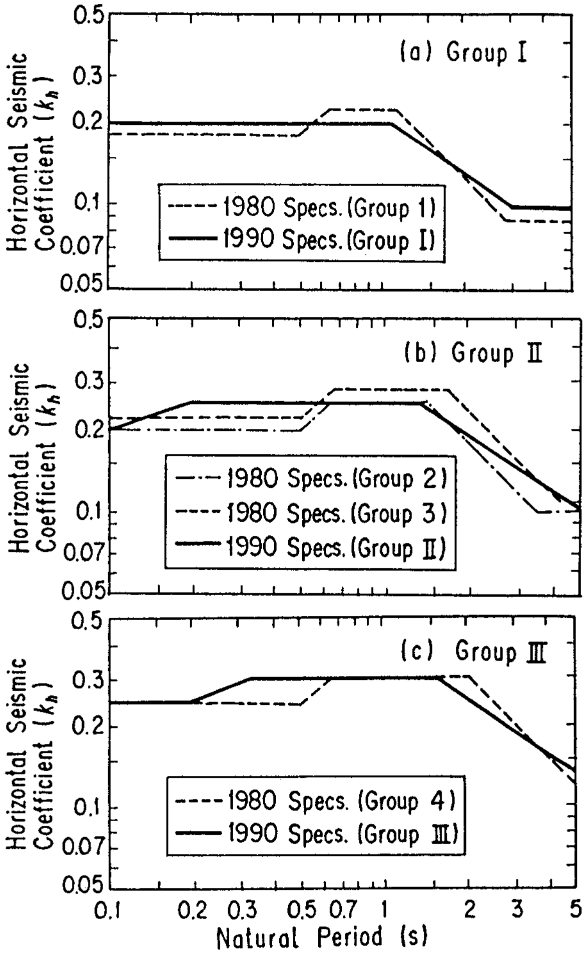
bridges have apparently existed for more than 70 years in Japan, no other specific requirements existed beyond the mandated seismic force until 1971 when the Japan Ministry of Construction published the first comprehensive earthquake-resistant design regulations for bridges known as *Guide Specifications for Seismic Design of Highway Bridges*. That nearly 400 bridges were damaged by the combined effect of the Miyagi-ken Hokubu (magnitude 6.5, April 30, 1962), Niigata (magnitude 7.5, June 16, 1964), and Tokachi-oki (magnitude 7.9, May 16, 1968) certainly provided some incentive for that major code overhaul. Among the relevant features of the new code were requirements to assess the impact of earthquakes on soil-liquefaction and other bridge foundation problems, and the introduction of seismic restrainers and other measures to prevent span failures. Also, in applying the coefficient method, a distinction was made between rigid and flexible bridges, with rigid bridges being defined as those having piers heights less than 15 m. The specifications were re-issued in 1980 as Part V — Seismic Design, in the *Design Specifications for Highway Bridges*, with, most notably, a completely revised method for evaluation of soil liquefaction, and permissible increases in allowable stresses for earthquake-resistant design, typically of 70% for steel superstructures and 50% for steel substructures (JSCE 1984).

The earthquake-resistant requirements for highway bridges were again extensively revised in 1990 (JRA 1990). An extensive description of the 1990 changes has been recently presented by Kawashima and Hasegawa (1994). In particular, the seismic coefficient method no longer makes a distinction between rigid and flexible structures, and, in all cases, the period of the structure must now be computed as part of the determination of the seismic coefficient. Ground conditions are also classified in three groups as a function of the site period T_G, computed using the standard simple multilayer shear beam formula. These three groups are (I) $T_G < 0.2$ (rock); (II) $0.2 \leq T_G < 0.6$ (alluvium/diluvium); and (III) $0.6 \leq T_G$ (soft alluvium).

According to this latest edition of the Specifications, the design horizontal seismic coefficient is determined as

$$[1] \quad k_h = c_Z c_G c_I c_T k_{h0}$$

Fig. 2. Japanese *Specifications for Highway Bridges*. Part V: seismic design: Comparison between 1980 and 1990 seismic coefficient levels for soil groups I (rock), II (alluvium/diluvium), and III (soft alluvium) (from Kawashima and Hasegawa 1994).



in which k_{h0} is the standard design horizontal seismic coefficient (equal to 0.2), c_Z is a modification factor to account for the earthquake risk in a given geographical region (equal to 1.0, 0.85, or 0.7, going from the most to least severe seismic zones; nearly all of Japan's major urban centres lie in the most severe zone), c_G is a modification factor to account for the ground condition (equal to 0.8, 1.0, or 1.2 for soil conditions I, II, or III, respectively), c_I is a modification factor for importance (equal to 1.0 for important bridges, 0.8 for all other bridges), and c_T is a modification factor for structural response that depends on the ground condition and the structural period. Therefore, the horizontal seismic coefficient lies in the range of 0.1 (specified minimum) to 0.3. A comparison of k_h in the 1980 and 1990 specifications is shown in Fig. 2. Note that soil groups 2 and 3 in the 1980 specifications were combined into group II in 1990. Incidentally, the seismic zoning map provided in the Specifications is a hybrid of probabilistically based calculations, engineering judgement, and other political factors. Hence, no specific return period is assigned to the zoning map.

A new subunit method has also been introduced in the Specifications to compute the inertia force for multispan bridges. Therefore, the horizontal static seismic design force is determined by multiplying k_h by the weight of the bridge subunit under consideration, and this force is applied at the centre of gravity of the subunit. The 1990 specifications also allow for dynamic analysis and provide a design acceleration response spectrum for each of the three soil groups, as well as three spectrum compatible acceleration records that may be used for time history analysis. However, it is important to remember that virtually all bridges in Kobe were designed to the much earlier editions of the Specifications, using static analysis.

Finally, although the 1990 edition of the Specifications also introduced major new requirements to check the ductility of reinforced concrete piers, no new requirements were introduced that affected the detailing practice for steel bridges. As of today, the Japanese bridge design specifications still do not include any requirements to ensure ductile response of steel piers; implicitly, reliance on the inherent ductility of the material was deemed to be likely sufficient.

It is noteworthy that the scope of the 1990 Specifications is limited to bridges having span lengths less than 200 m. Seismic specifications for bridges with spans longer than 200 m are developed on a case-by-case basis. Moreover, in the 1960s, when many large road construction projects were being initiated in Japan, other seismic design criteria were developed for application to special projects administered by the Japan Highway Public Corporation, the Metropolitan Expressway Public Corporation, the Hanshin Expressway Public Corporation (which operates the expressways in the Kobe – Osaka area), the Honshu – Shikoku Bridge Authority, and the Japanese National Railways. Some of these projects, such as Honshu – Shikoku, are ongoing and involve extensive seismic engineering studies using state-of-the-art techniques in seismology, geotechnical, and structural engineering. However, the *minimum* requirements adopted by these corporations have generally closely followed the various editions of the *Specifications for Highway Bridges*, being updated in an identical manner and nearly simultaneously to that basic document.

3.2. Evolution of steel piers detailing requirements

Since important damage was suffered by steel piers during the Hyogo-ken Nanbu earthquake, as will be demonstrated in a later section, a review of their past and current design/detailing requirements is relevant.

3.2.1. Stability of rectangular stiffened box-piers

Three types of buckling can typically occur in the stiffened box sections typically used as bridge piers in Japan. First, the plate-segments between the stiffeners may buckle, the stiffeners acting as nodal points; in this type of "panel buckling," buckled waves appear on the surface of the piers, but the stiffeners do not appreciably move perpendicularly to the plate during the buckling process. Second, the entire box stiffened wall can globally buckle; in this type of "wall buckling," the plate and stiffeners move together perpendicularly to the original plate plane. Third, the stiffeners themselves may buckle first, triggering in turn other buckling modes.

Prior to 1971, the design requirements for stiffened plates in the Japanese bridge design code were essentially identical to the German ones (DIN 1952). Although not codified in great detail, the approach used worldwide at the time relied on results from the elastic buckling theory, generally multiplying the buckling stress calculated according to this theory by some safety factor. However, following the failure during construction of four orthotropic deck bridges designed using the linear buckling theory (Ballio and Mazzolani 1983; ECCS 1976), it was recognized that this approach did not provide correct estimates of ultimate capacity, as the behaviour of stiffened plates is greatly influenced by geometric and mechanical imperfections (i.e., out-of-straightness and residual stresses) not taken into account by the linear elastic buckling theory. Eventually, more refined analytical methods capable of correctly predicting the ultimate capacity of stiffened plates were developed, but until these are amenable to a more practical format for design purposes, the practice worldwide has been to modify semi-empirically the elastic analysis results using complex factors which account for plate slenderness, stiffener rigidity, and construction imperfections.

In Japan, a design criterion was developed following an extensive program of testing of stiffened steel plates in the 1960s and 1970s (Watanabe et al. 1981); the results of this testing effort are shown in Fig. 3, along with a best-fit curve. The slenderness parameter, which defines the abscissa in that figure, deserves some explanation. Realizing that the critical buckling stress of plate panels between longitudinal stiffeners can be obtained by the well-known result from the theory of elastic plate buckling,

$$[2] \quad \sigma_{cr} = \frac{k_o \pi^2 E}{12(1 - \nu^2)(b/nt)^2}$$

a panel slenderness factor can be defined as

$$[3] \quad R_P = \bar{\lambda} = \sqrt{\frac{\sigma_Y}{\sigma_{cr}}} = \left(\frac{b}{nt}\right)\sqrt{\frac{12(1-\nu^2)\sigma_Y}{k_o \pi^2 E}} = \frac{b}{(425nt)}\sqrt{\frac{\sigma_Y}{k_o}}$$

In the above, b and t are the stiffened plate width and thickness respectively, n is the number of panel spaces in the plate (i.e., one more than the number of internal longitudinal stiffeners across the plate), E is Young's modulus, ν is Poisson's ratio (0.3 for steel), and k_o is a factor taking into account the boundary conditions (k_o = 4 in this case). The Japanese design requirement for stiffened plates in compression is based on a simplified and conservative curve based on the experimental data, and defined as follows:

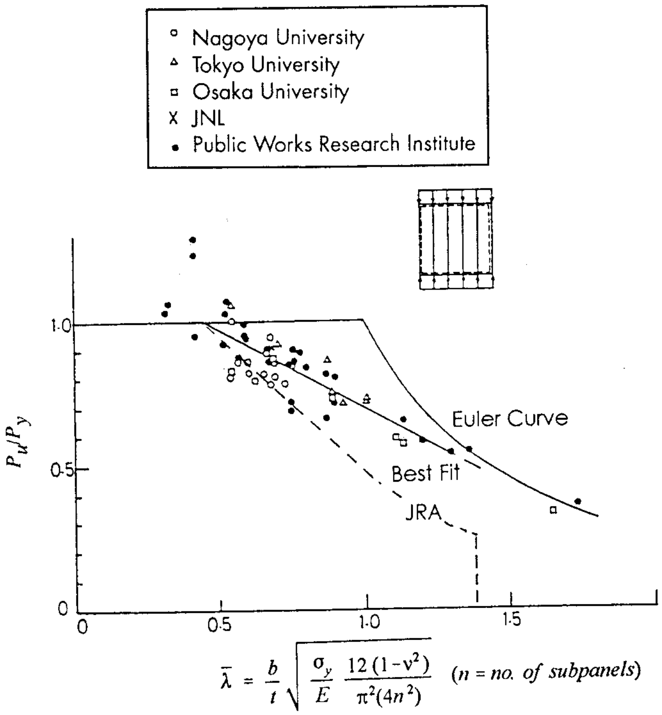
$$\frac{\sigma_U}{\sigma_Y} = 1.0 \qquad \text{for } R_P \le 0.5$$

$$[4] \quad \frac{\sigma_U}{\sigma_Y} = 1.5 - R_P \qquad \text{for } 0.5 < R_P \le 1.0$$

$$\frac{\sigma_U}{\sigma_Y} = \frac{0.5}{R_P^2} \qquad \text{for } R_P > 1.0$$

This curve is also shown in Fig. 3. Note that values of σ_U/σ_Y less than 0.25 are not permitted. This expression is then converted into the allowable stress format of the Japanese bridge code, using a safety factor of 1.7. However, as allowable stresses are magnified by a factor of 1.7 for load combinations which include earthquake effects, the above ultimate strength expressions are therefore effectively used.

Fig. 3. Results of Japanese tests of stiffened plates (data points) and curve-fitting of data (solid line) (from Watanabe et al. 1981), and design requirement for steel box-piers as per the Japanese *Specifications for Highway Bridges*. Part V: seismic design (dashed line).



The point R_P = 0.5 is important, as it defines the theoretical boundary between the region where the yield stress can be reached prior to local buckling (R_P below 0.5) and vice versa (R_P above 0.5). For a given steel grade (and corresponding yield stress), this will correspond to a given b/tn ratio; and for a given plate width b, a "critical thickness" t_o can be calculated. In other words, for a stiffened column of a given width and material, using plates thicker than t_o will ensure yielding prior to buckling, and vice versa.

To be able to design the longitudinal stiffeners, it is necessary to define two additional parameters: the relative flexural stiffness of a longitudinal stiffener to a plate panel, γ_l, and the corresponding relative extensional rigidity, δ_l. As the name implies,

$$[5] \quad \gamma_l = \frac{\text{stiffener flexural rigidity}}{\text{plate flexural rigidity}}$$

$$= \frac{EI_l}{bD} = \frac{12(1-\nu^2)I_l}{bt^3} = \frac{10.92 I_l}{bt^3}$$

where I_l is the moment of inertia of the T-section made up of a longitudinal stiffener and the effective width of the plate to which it connects (or more conservatively and expediently, the moment of inertia of a longitudinal stiffener taken about the axis located at the inside face of the stiffened plate). Similarly, the extensional rigidity is expressed as

$$[6] \quad \delta_l = \frac{A_l}{bt}$$

where A_l is the area of a stiffener.

Since the purpose of adding stiffeners to a box section is partly to eliminate the severity of the wall buckling problem, there must therefore exist an "optimum rigidity," γ_l^*, of the stiffeners, beyond which panel buckling between the stiffeners will develop before wall buckling. In principle, according to elastic buckling theory for ideal plates (i.e., plates without geometrical imperfections or residual stresses), further increases in rigidity beyond that optimum would not further enhance the box-pier's buckling capacity. Although more complex definitions of this parameter exist in the literature (Kristek and Skaloud 1991), the above description is generally sufficient for the box-piers of interest here. This optimum rigidity is mathematically defined as

$$[7] \quad \gamma_l^* = 4\alpha^2 n(1 + n\delta_l) - \frac{(\alpha^2 + 1)^2}{n} \qquad \text{for } \alpha \leq \alpha_o$$

and

$$[8] \quad \gamma_l^* = \frac{1}{n}\{[2n^2(1 + n\delta_l) - 1]^2 - 1\} \qquad \text{for } \alpha > \alpha_o$$

where α is the ratio of the lateral stiffeners (or diaphragms) spacing, a, over the clear distance between the webs of the box column, b; and α_o is defined as

$$[9] \quad \alpha_o = \sqrt[4]{1 + n\gamma_l}$$

All other variables have been defined earlier. These expressions can be obtained by recognizing that, for plates of thickness less than t_o, it is logical to design the longitudinal stiffeners such that the stiffened plate does not buckle (i.e., wall buckling) prior to the panels between stiffeners (i.e., panel buckling), and consequently, as a minimum, is able to reach the same ultimate stress as the latter. Therefore, following the same derivation as above, a stiffened plate slenderness factor, R_H, can be defined:

$$[10] \quad R_H = \sqrt{\frac{\sigma_Y}{\sigma_{cr}}} = \left(\frac{b}{t}\right)\sqrt{\frac{12(1 - \nu^2)\sigma_Y}{k_s\pi^2 E}}$$

However, in this case, based on elastic plate buckling theory (Ballio and Mazzolani 1983), k_s is found to be equal to

$$[11] \quad \begin{aligned} k_s &= \frac{(1 + \alpha^2)^2 + n\gamma_l}{\alpha^2(1 + n\delta_l)} && \text{for } \alpha \leq \alpha_o \\ k_s &= \frac{2(1 + \sqrt{1 + n\gamma_l})}{1 + n\delta_l} && \text{for } \alpha > \alpha_o \end{aligned}$$

Therefore, by making $R_H = R_P$ (to be able to develop the same ultimate stress), the above values for γ_l^* can be derived. Thus, whenever the stiffened plate thickness, t, is less than the descriptive parameter t_o, the Japanese bridge design code specifies that $\gamma_{l,req}$ be equal to γ_l^* defined above.

However, whenever a plate thicker than t_o is chosen, bigger stiffeners are unnecessary, since yielding will occur prior to buckling. Indeed, for extremely thick plates, there would be obviously no benefit in having any stiffeners. Physically, this means that the critical buckling stress for the stiffened plate does not need to exceed the yield stress, which is reached by the panels between stiffners when $t = t_o$. Mathematically, this implies that

$$[12] \quad \begin{aligned} \sigma_{cr(\text{stiffened plate})} &= k_s \frac{\pi^2 E}{12(1 - \nu^2)}\left(\frac{t}{b}\right)^2 \\ &= \sigma_Y = \sigma_{cr(\text{panels})\text{when } t=t_o} \\ &= k_o \frac{\pi^2 E}{12(1 - \nu^2)}\left(\frac{t_o}{b/n}\right)^2 \end{aligned}$$

or, alternatively, that

$$[13] \quad \begin{aligned} R_H &= \left(\frac{b}{t}\right)\sqrt{\frac{12(1 - \nu^2)\sigma_Y}{k_s\pi^2 E}} = 0.5 \\ &= R_{P(t=t_o)} = \left(\frac{b}{nt_o}\right)\sqrt{\frac{12(1 - \nu^2)\sigma_Y}{k_o\pi^2 E}} \end{aligned}$$

Therefore, directly,

$$[14] \quad k_s = k_o n^2\left(\frac{t_o}{t}\right)^2 = 4n^2\left(\frac{t_o}{t}\right)^2 \qquad \text{for } t \geq t_o$$

As a result, whenever a plate thicker than t_o is provided, the code requires

$$[15] \quad \gamma_{l,req}^* = 4\alpha^2 n\left(\frac{t_o}{t}\right)^2(1 + n\delta_l) - \frac{(\alpha^2 + 1)^2}{n} \qquad \text{for } \alpha \leq \alpha_o$$

and

$$[16] \quad \gamma_{l,req}^* = \frac{1}{n}\left\{\left[2n^2\left(\frac{t_o}{t}\right)^2(1 + n\delta_l) - 1\right]^2 - 1\right\} \qquad \text{for } \alpha > \alpha_o$$

In addition to the above requirements, conventional slenderness limits are imposed to prevent local buckling of the stiffeners prior to that of the main member. For example, for a rectangular stiffener expected to reach its yield strength of 235 MPa, the ratio of its length divided by its thickness should not exceed 13.1.

It is noteworthy that the above requirements will not ensure ductile behaviour of steel piers. Although a panel slenderness R_P of less than 0.5 will ensure that yield stresses are reached prior to the onset of buckling, it does not indicate whether a stable ductile hysteretic behaviour can develop. Indeed, the Japanese specifications do not require a check on whether such ductile behaviour can be sustained during the design-level earthquake, contrary to what is currently mandated for reinforced concrete piers.

3.2.2. Stability of hollow circular steel piers

The Japanese specifications' allowable stress equations for steel tubular piers are based on local buckling considerations, accounting for manufacturing tolerances, an unevenness factor, U, of 0.001, and Plantema's experimental results (Galambos 1988). Typically, for SM41, SS41, SMA41, or STK41 steels (having a nominal yield strength of 235 MPa), the allowable compressive stresses (in MPa) are

$$[17] \quad \begin{aligned} &140 && \text{for } \frac{R}{\alpha t} \leq 50 \\ &140 - 0.43\left(\frac{R}{\alpha t} - 50\right) && \text{for } 50 < \frac{R}{\alpha t} \leq 200 \end{aligned}$$

where R is the radius of the steel tube (centre to outer edge), t is its thickness, and

$$[18] \quad \alpha = 1 + \frac{\phi}{10}$$

where $\phi = (\sigma_1 - \sigma_2)/\sigma_1$; σ_1 and σ_2 are the maximum compression and tension stresses acting on the cross section due to bending (compression being defined as negative), and α is a factor that accounts for the 20% larger strength generally observed experimentally when comparing the behaviour of cylindrical tubes loaded in bending against those in compression. Again, for earthquake effects, these allowable stresses would be multiplied by an "overstress" factor of 1.7, accordingly scaling the allowable values up to ultimate stress levels (although only approximately in this case, as a variable safety factor has been introduced over the permissible R/t range). Allowable shear stresses are specified based on the elastic shear buckling equation, with a safety factor of 3. Typically, for the same grade 41 steels as above, these allowable shear stresses (in MPa) are

$$[19] \quad \begin{array}{ll} 80 - 0.0019\left(\dfrac{R}{t}\right)^2 & \text{for } \dfrac{R}{t} \leq 125 \\ \dfrac{7500}{R/t} - 9 & \text{for } 125 < \dfrac{R}{t} \leq 200 \end{array}$$

Shear and axial compressive stresses are to be combined as follows:

$$[20] \quad \frac{\sigma}{\sigma_a} + \left(\frac{\tau}{\tau_a}\right)^2 \leq 1.0$$

where the subscript a refers to the allowable stresses specified above. Steel tubes used for highway bridge piers in Japan generally have ring stiffeners (or diaphragms) spaced at $3d$ to prevent buckling or local deformations due to shear or twist, but no longitudinal stiffeners. These ring stiffeners are empirically designed according to the following empirical requirement.

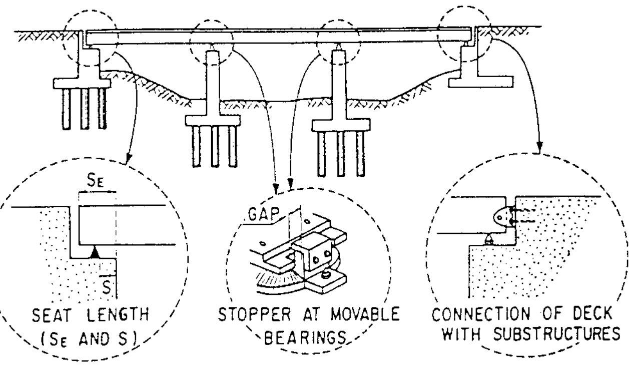
$$[21] \quad t \geq \frac{b}{17} \quad \text{and} \quad b \geq \frac{d}{20} + 70$$

where t and b are respectively the width and thickness of the ring stiffener, and d is the pier diameter. However, for practical reasons, these stiffeners may be omitted if R/t does not exceed 30, at the penalty of a lower allowable stress, for example, an allowable stress of 50 MPa irrespective of the R/t ratio for grade 41 steels.

3.3. Bearing resistance requirements

In Japan, the horizontal seismic force design requirement for bridge bearings has essentially always been identical to that specified for the bridge itself (Sect. 3.1). What has changed over time (besides the magnitude of the seismic coefficient) is the extent of the guidance given to the structural engineer regarding the distribution of this seismic force to the various supports. A description of the rather complex and detailed procedure now included in the Japanese *Specification for Highway Bridges* is beyond the scope of this paper, and has recently been summarized by other researchers (Kawashima and Hasegawa 1994).

Fig. 4. Seat-width requirements and example of stopper at moveable bearing (from Kawashima 1991).



Hence, for bearings, the Specifications explicitly but broadly require that "a fixed bearing portion shall be safe against inertia force of a superstructure." Only one special clause is added, requiring that an uplift force equal to the product of a design seismic vertical coefficient of 0.1 and the vertical reaction due to dead load be considered. This latter force is to be taken as a net uplift design force, taken alone without the simultaneous consideration of gravity loads or horizontal seismic forces. In many instances, engineering judgement is required to comply with the intent of the clause. As one example, the commentary to the Specifications suggests that webs of girders may need to be reinforced by stiffeners over the bearings to prevent localized damage.

However, what may appear at first as rather liberal bearing design requirements must be understood in the perspective that additional provisions are specified to prevent the collapse of superstructure in the event of bearing failures, as described in the next section.

3.4. Seismic restrainer design requirements

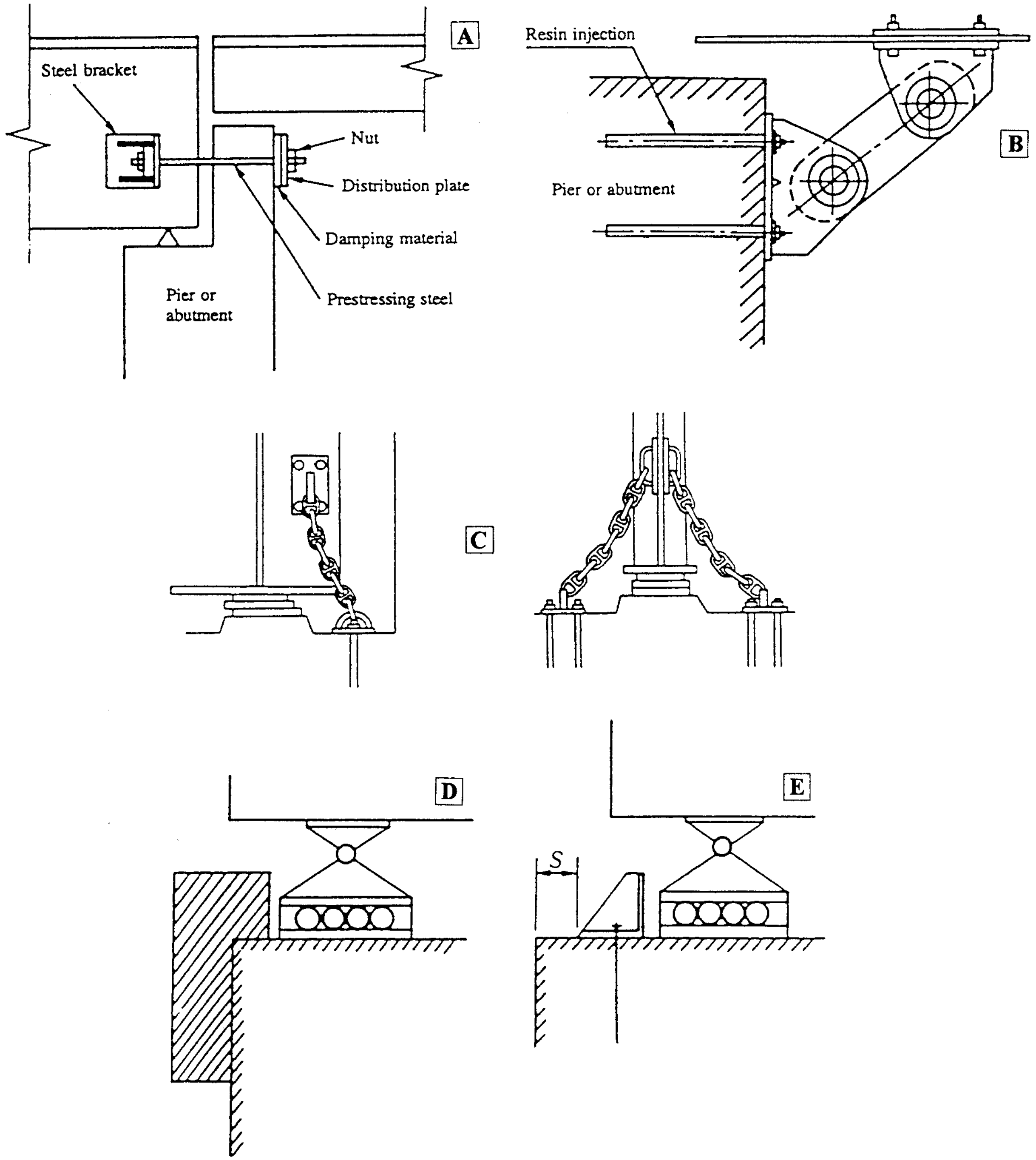
The 1971 edition of the *Specifications for Highway Bridges* introduced requirements "to prevent falling-off of superstructures," or, in other words, span collapses. The provision of a large seat-width, or, alternatively, seismic restrainers, is mandated at all girder ends, even if only fixed bearings are present. This recognizes that some minimum protection must exist against span collapses if fixed bearings rupture during unexpectedly severe earthquakes. It is even recommended that especially important bridges be provided with both minimum seat-width requirements and seismic restrainers.

Minimum seat-width requirements are prescribed for the total seat length, S_E, and the bearing seat length, S, as shown in Fig. 4, to be at least as follows:

$$[22] \quad \begin{array}{ll} S_E = 70 + 0.5L & \text{if } L \leq 100 \\ S_E = 80 + 0.4L & \text{if } L > 100 \\ S = 20 + 0.5L & \text{if } L \leq 100 \\ S = 30 + 0.4L & \text{if } L > 100 \end{array}$$

where S and L are the seat length and span length in centimetres and metres, respectively. Clearly, S is the distance to the nearest edge below the bearing, and S_E shall be taken

Fig. 5. Suggested details for seismic restrainers, and displacement restraining devices (from JRA 1990).



in the most critical direction, which, for example, could be measured perpendicularly to the expansion joint in the case of a skewed bridge.

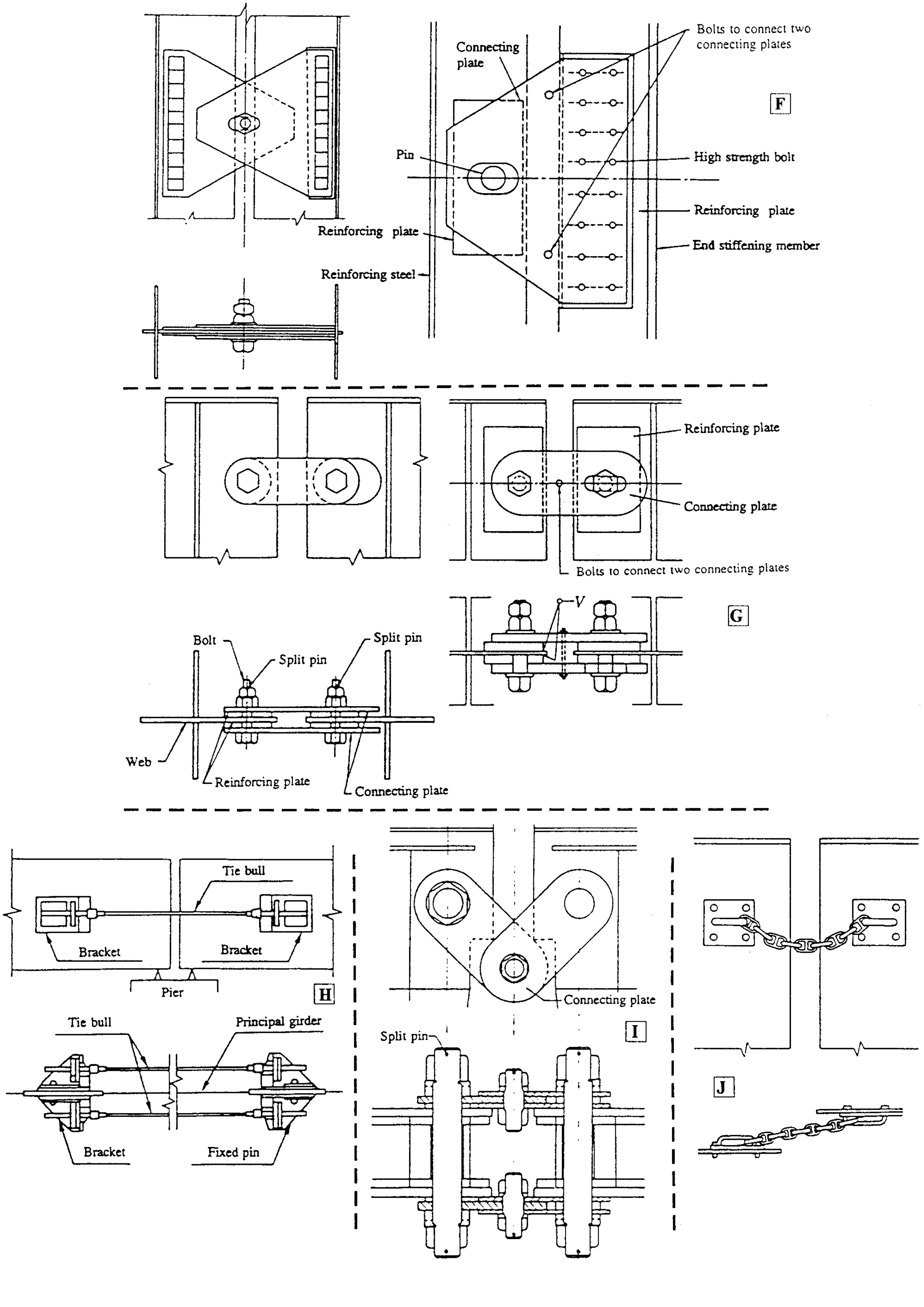
Seismic restrainers of various designs are proposed by the Specifications, to either connect a girder to a substructure (Figs. 5*a*–5*c*), two girders together (Figs. 5*f*–5*j*), or buttress against excessive displacements (Figs. 5*d* and 5*e*). All these devices to prevent span collapses shall be designed for twice the horizontal seismic coefficient considered in the design of the bridge. Therefore,

$$[23] \quad H_R \geq 2k_hR_d$$

where H_R is the design force requirement for the restraining device, k_h is the seismic coefficient defined earlier, and R_d is the dead-load vertical reaction carried by the bearing at the girder's end, and taken as the largest reaction for restrainers connecting adjacent girders. Moreover, restrainers connecting two girders together shall be capable of resisting the weight of the heaviest girder (i.e., $1.0R_d$) applied vertically, such that one girder may be able to support the adjacent one should it ever fall from its support.

In addition to the above requirements, the Specifications mandate that stoppers must be provided at roller-bearings to limit the relative movement of the upper and lower portions

Fig. 5 (*concluded*).



of those bearings (Fig. 4), thus reducing the risk that rollers will be dislodged from the bearing assembly. These should be capable of resisting a force of $1.5k_hR_d$. In some instances, the buttress designs described above could simultaneously satisfy this requirement.

3.5. Recent Japanese research results on ductility of steel bridge columns

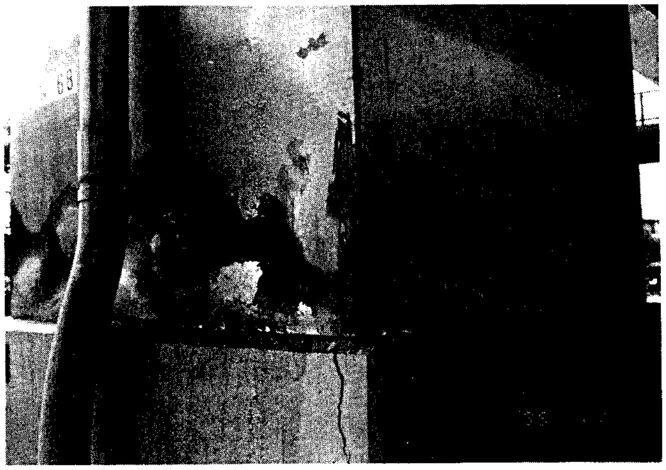
While large steel-box bridge piers have been used in the construction of Japanese expressways for at least 30 years, research on their seismic resistance only started in the early 1980s. The first inelastic cyclic tests of thin-walled boxes were conducted by Usami and Fukumoto (1982) as well as Fukumoto and Kusama (1985). They observed that even bridge piers that locally buckled could develop the stable cyclic hysteretic curves typical of steel. However, important strength degradation developed as local instability progressed, more notably for sections having larger plate slenderness.

Other tests have been conducted since by the Public Works Research Institute of the Ministry of Construction (e.g., Kawashima et al. 1992*a*, 1992*b*; MacRae and Kawashima 1992) and civil engineering research groups at various universities (e.g., Watanabe et al. 1992; Usami et al. 1992; Nishimura et al. 1992). (Note: Although much earlier references to that work can be found, most are in Japanese, whereas those cited here have been written in English.)

The Public Works Research Institute tests considered 22 stiffened box-piers of configuration representative of those used in some major Japanese expressways. These experiments demonstrated that these box-piers could develop reliable ductilities of at least 4, or even more, depending on the testing regime applied. Since a buckling mechanism was concurrent with the development of this ductility, strength degradation was significant and a function of loading history. Specimens with a gamma ratio, γ_l/γ_l^*, less than 1, or slightly greater than 1, behaved in a wall buckling mode with severe strength degradation and failed, after a few cycles, by fracture of the corner welds. Interestingly, as a consequence of plate buckling, specimens tested using a shake-table tended to progressively drift in a given direction without experiencing yielding in the reversed direction. This raised important questions on what should constitute an appropriate quasi-static testing regime to verify the seismic adequacy of steel piers. Also, based on this series of tests, Kawashima et al. (1992*a*) proposed a method to calculate the available ductility of steel piers, as a counterpart to the method currently required by the Japanese bridge code for concrete columns.

A large portion of the research effort in recent years investigated the effectiveness of many different strategies to improve the seismic performance, ductility, and energy dissipation of those steel piers. Among the factors observed to have a beneficiary influence were the use of (*i*) relative stiffeners rigidity, γ_l/γ_l^*, above a value of 3 as a minimum (Usami et al. 1992), or preferably 5 (Kitada et al. 1995; Watanabe et al. 1995); (*ii*) longitudinal stiffeners having a higher grade of steel than the box-plates (Usami et al. 1992); (*iii*) minimal amount of stiffeners (Watanabe et al. 1995; Kitada et al. 1995; Sugiura, Department of Civil Engineering, University of Kyoto, personal communication); (*iv*) concrete filling of steel piers (Usami et al. 1992); (*v*) box columns having round corners, built from bend plates, and having weld seams away from the corners, thus avoiding the typically problem-prone sharp welded corners (Watanabe et al. 1992); and (*vi*) to some extent, smaller width to thickness ratios (Usami et al. 1992; Fukumoto and Itoh 1992).

Fig. 6. Local buckling of the panel-type at the base of a rectangular steel column along Hanshin Expressway.



Interestingly, all the above research was concerned with the inelastic cyclic response of rectangular steel columns (or rectangular with rounder corners in some cases). To the authors' knowledge, tests of round columns have also been recently conducted by the Public Works Research Institute (Kawashima, Department of Civil Engineering, Tokyo Institute of Technology, personal communication), but results from this experimental research have not been published at the time of this writing.

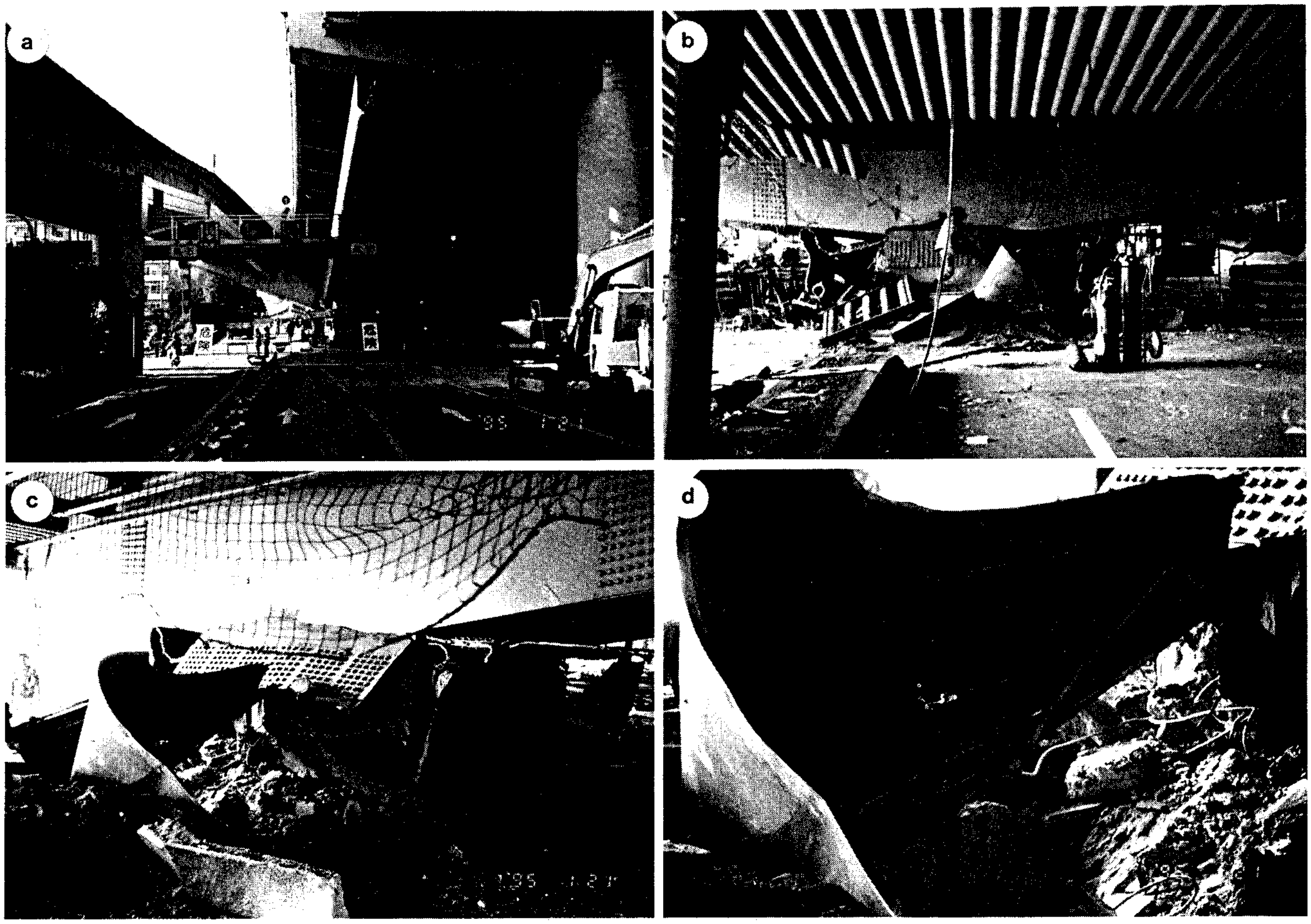
3.6. Seismic design of railroad bridges

The earthquake-resistant design standard for railway structures (Japanese National Railways 1979) has generally not included special provisions for the ductile detailing of steel structures. Only strength requirements are addressed by the consideration of a horizontal seismic coefficient equal to 16% to 25% of the structure's dead load, depending on the soil conditions and the period of the structure; note that allowable stresses magnified by a factor of 1.7 over those specified for gravity-type designs have been used, as typically done for highway bridges.

An interesting feature of these earthquake-resistant design requirements is that no less than 50% of the tracks (equivalent to 50% of the bridge length on single-track bridges) must be considered loaded by trains simultaneously to the application of the earthquake forces during design. It is also recommended that railroad bridges be designed such that their lateral vibration period avoids the occurrence of resonance with the natural "rolling" period of trains, typically in the 0.6–1.6 s range, in order to minimize the risk of derailment of trains located on bridges during earthquakes. The Standard also prescribes that bearings capable of resisting 28% of the bridge's self-weight in the transverse direction be provided, with side-blocks or shoes.

While numerous railroad steel bridges suffered damage in past Japanese earthquakes, that damage prior to the Kobe

Fig. 7. Failed rectangular steel column (Pier 55) along Hanshin Expressway: (*a*) collapsed span; (*b*) global view of the failed column; (*c*) close-up view, showing small concrete-filled portion; (*d*) plate of the built-up column.



earthquake was generally a consequence of foundation failure. However, it has been alleged by some railway bridge engineers that since rather conservative foundation designs are now adopted in that industry, failures in the newer steel bridges are likely to appear at the next weakest structural point in future earthquakes. However, newer Japanese railroad bridges tend to be of reinforced concrete, out of concern about the noise level produced by trains crossing bridges in crowded urban environments.

4. Damage to steel bridges

4.1. Elevated expressways and railroad bridges

4.1.1. Column failures — local buckling
Between Nishinomiya (east of Kobe) and Kobe, almost all concrete columns along the Hanshin Expressway suffered damage, triggering, in some instances, span collapses as a consequence of their severe shear failures or inelastic bending. Over the same length of expressway, a few of the steel box-columns buckled, some rather severely (panel buckling visible in Fig. 6), and collapse occurred at one location (Pier P55) as a result of a steel column failure (Fig. 7). The steel box-column that failed appeared to have been squashed vertically, almost as if the plates on each of its four sides were "peeled-off." Although little information can be obtained from that completely destroyed column, damage to the steel box-column on the adjacent span (i.e., Pier P56) provided a clue as to the triggering event: failure of a weld-seam at the bottom corner of the box-column (Fig. 8). The cross section of Pier P55 is shown in Fig. 9. Tests conducted by the Ministry of Construction of Japan on the steel plates of that pier indicate that the SM50 steel used had a yield strength of 369 MPa and an ultimate strength of 582 MPa. Using these data and the equations presented earlier, it is found that $\gamma_l \gamma_l^*$ equals 0.92; since this is less than unity, wall buckling would have been the ultimate failure mode expected, with eventual fracture of the corner weld, as demonstrated by the aforementioned research results (Kawashima et al. 1992*a*). A nearly identical failure occurred at Pier P9 of National Road 43 (Fig. 10); however, for that cross section (Fig. 9), the $\gamma_l/\gamma_l^*$is nearly equal to 3.0, and a more ductile performance would have been expected. Awaiting a more rational explanation, it has been alleged (Hyogo-ken Nanbu Earthquake Committee 1995) that failure initiated at the splice plates, since stiffeners are obviously discontinued at that location.

Local buckling of round columns used on part of the expressway west of Kobe was also particularly extensive, and sometimes very severe, with rupture of the buckled steel sometimes taking place due to excessive inelastic deformations (Fig. 11). Mild to severe local buckling also occurred

Fig. 8. Rectangular steel column in span adjacent to the failed column of Fig. 7, showing damage initiation mechanism: (*a*) global view; (*b*) close-up of seam-weld failure.

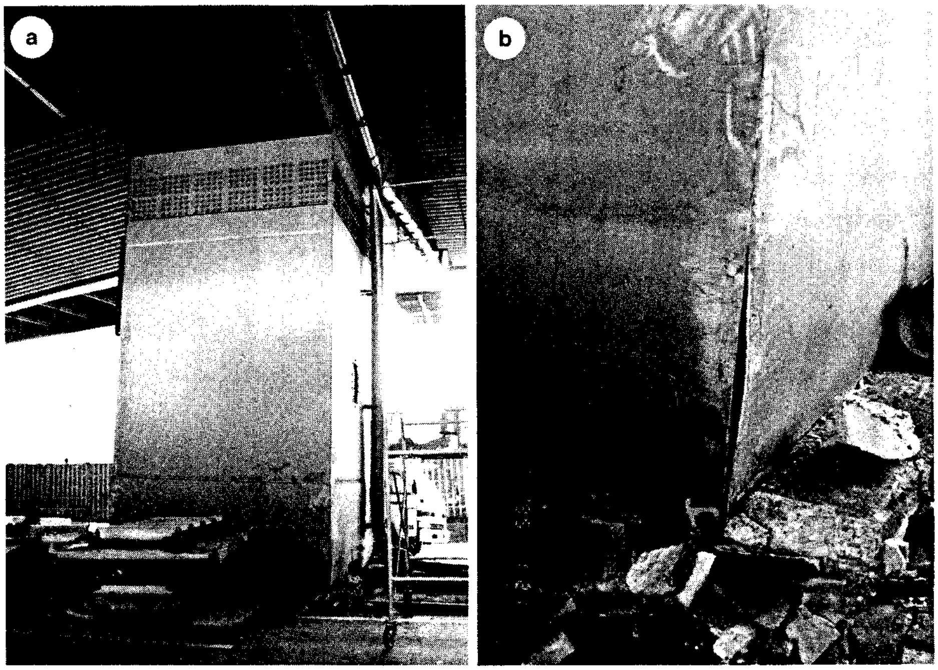
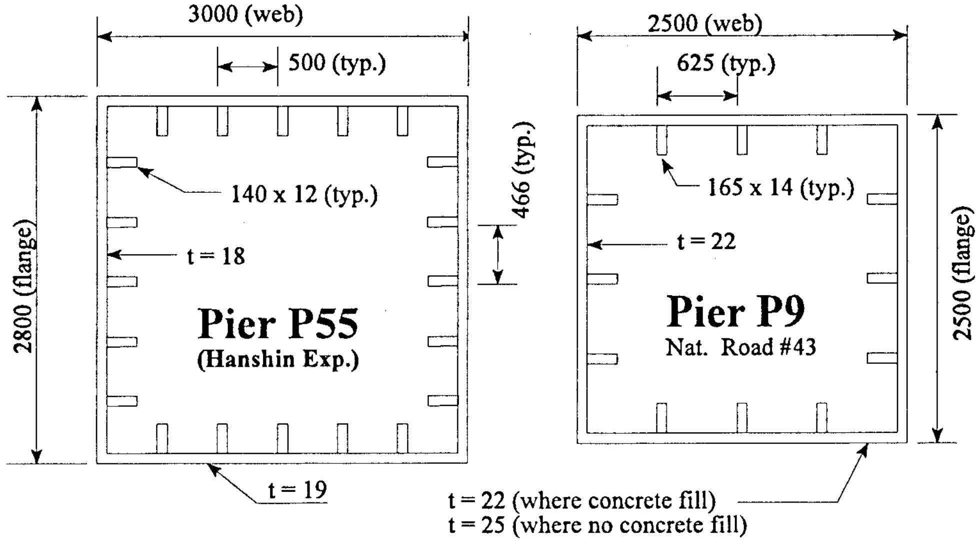


Fig. 9. Details of cross sections of Pier 55 of Hanshin Expressway and Pier 9 of National Road 43.



on a number of round and square built-up hollow steel columns of the Port-Harbour Highway. Many of these columns supported double-deck highway structures. Examples of such buckling are shown in Figs. 12–14. A typical local buckle at the base of a circular column, reminiscent of the so-called elephant-foot buckling often observed in large cylindrical tanks following earthquakes, can be seen in Fig. 13. Buckling at the third point along the height of circular and square columns are shown respectively in Figs. 12 and 14, across a door-hatch in the latter case. Frequently, above-base column damage as shown in some of the above figures has occurred at or near a structural discontinuity, such as (*i*) at the location of a door-hatch; (*ii*) where thinner steel plates were used as permitted by the moment diagrams considered during design, or (*iii*) at the top of the concrete fill. Indeed, on that latter point, it has been the Japanese practice to sometimes fill steel columns with concrete for a few metres above the base to prevent their damage in the event of a vehicle collision. It appears that this practice has been used irregularly, with no consistent height for the concrete filling.

4.1.2. Column failures — brittle fractures

Brittle failures were sporadically discovered in columns which otherwise showed no signs of local buckling. For example, in the rectangular column of Fig. 15, cracking occurred at a

Fig. 10. Failed rectangular steel column (Pier 9) along National Road 43: (*a*) global view of the failed column; (*b*) close-up of the damaged stiffened steel plate.

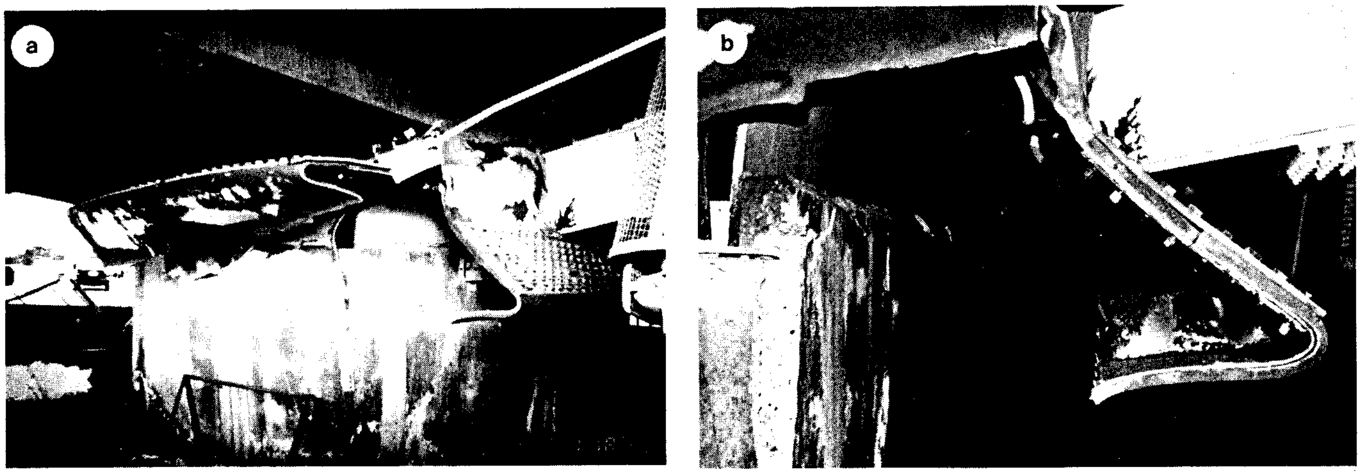
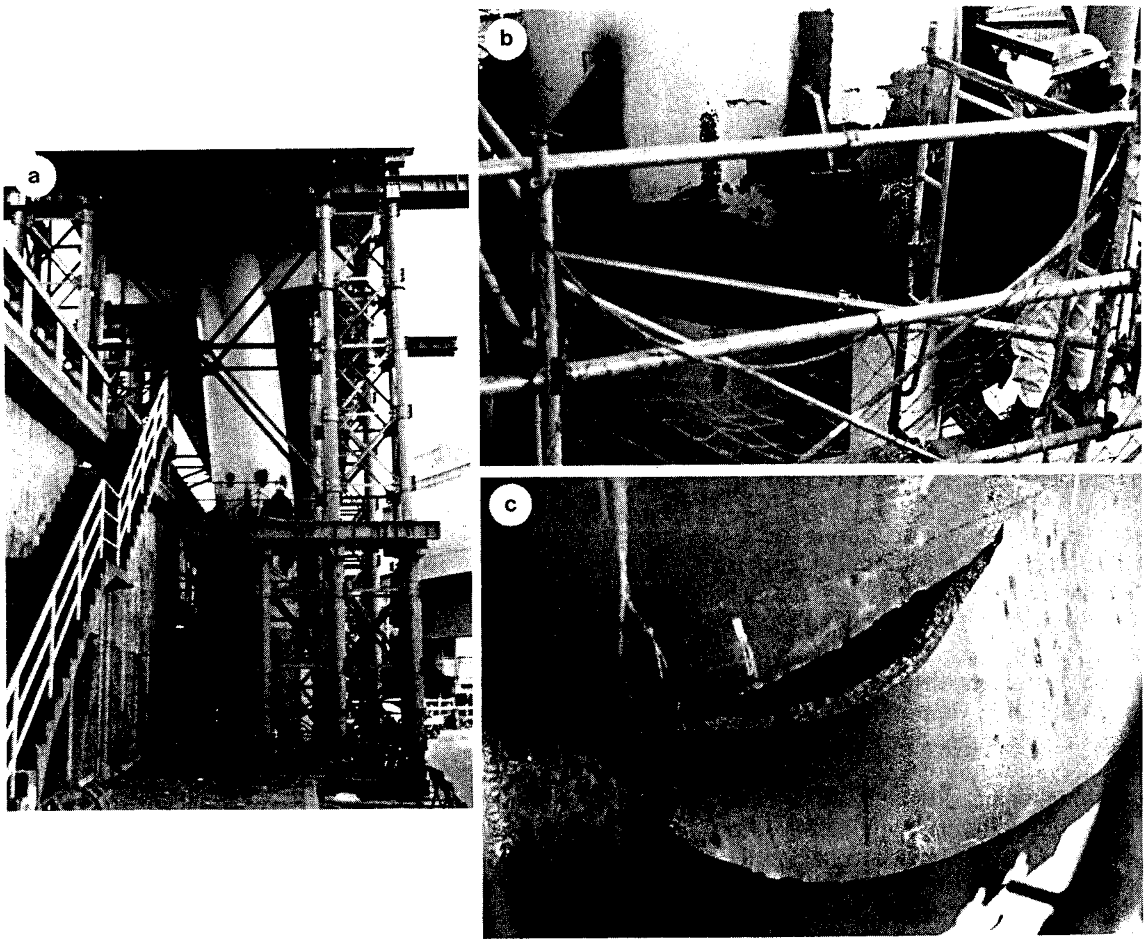


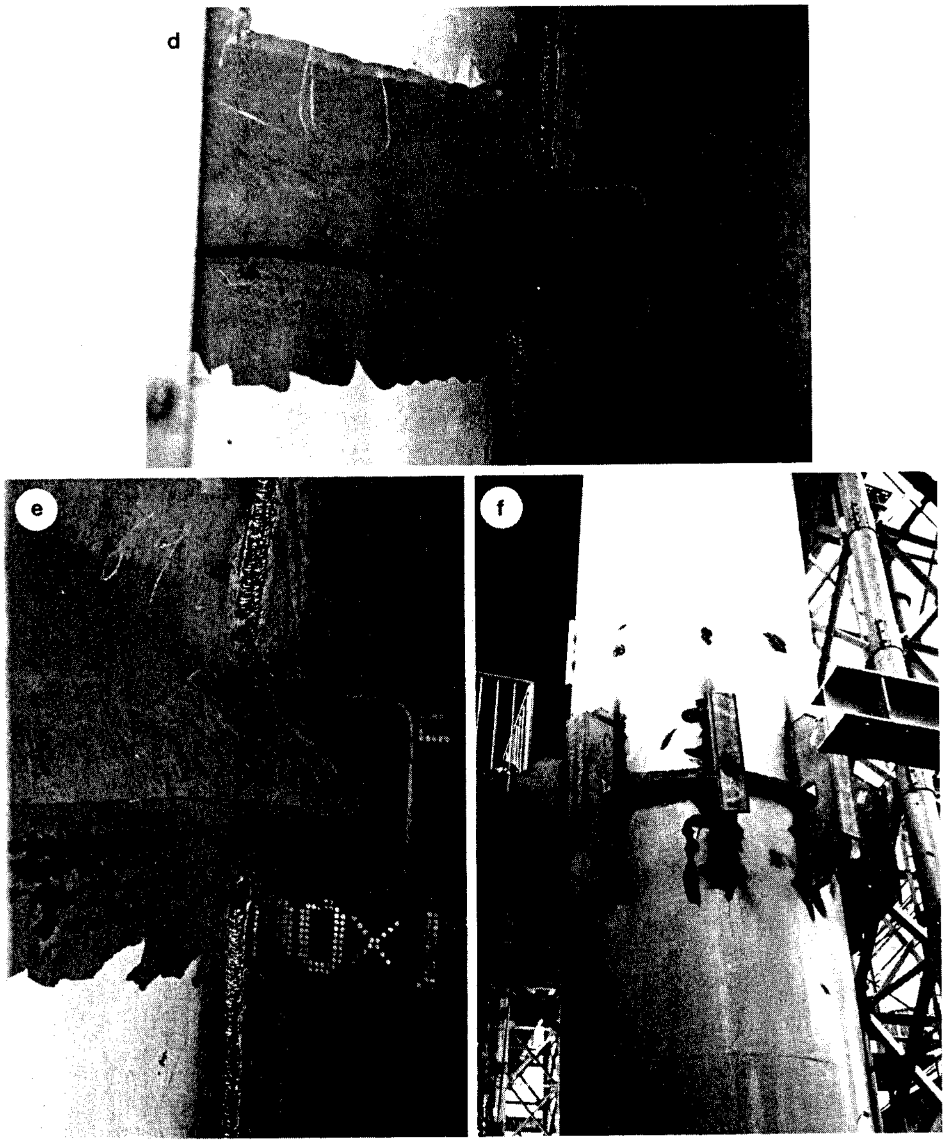
Fig. 11. Examples of severe local buckling of circular steel columns: (*a*) elevation of a damaged column, also showing temporary supports; (*b*) location of local buckling and fractured steel; (*c*) close-up of fractured steel; (*d*) close-up showing severity of buckling; (*e*) close-up showing fracture along buckled steel; (*f*) elevation of another damaged circular steel column, also showing temporary repair across a fractured locally buckled area.



level where the column was filled with concrete.

In at least one instance, brittle failure of the columns of a railroad-supporting steel portal-frame was also observed (Fig. 16). The bearings of that structure can also be seen to have failed. These columns were apparently of cast-steel, formed using a centrifugal procedure developed in Japan.

Fig. 11 (*concluded*).



Whether or not column failure was the triggering failure event, the sight of a brittle steel failure without evidence of prior yielding is disconcerting.

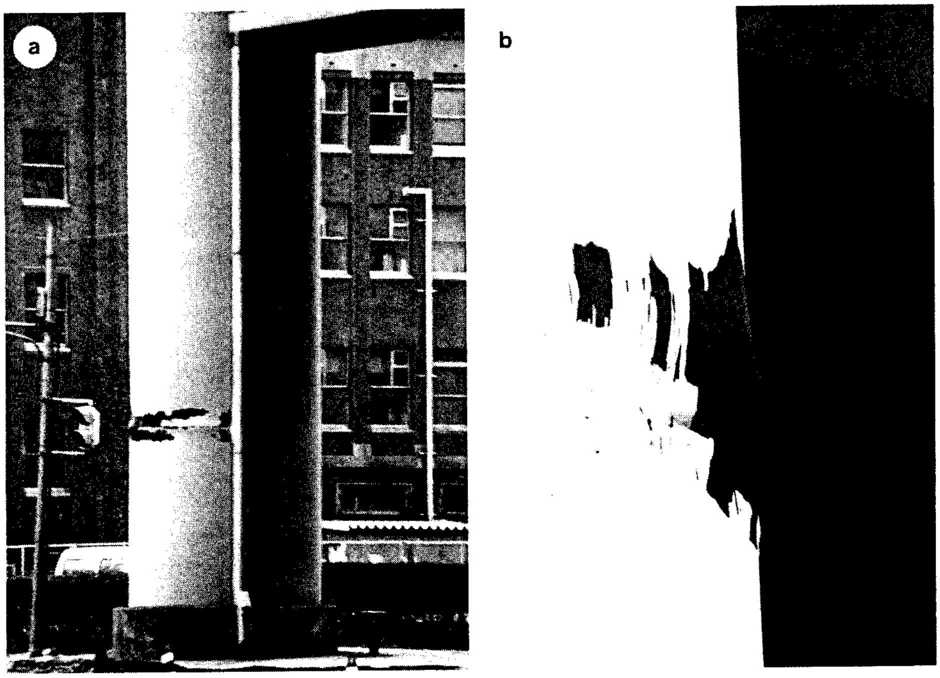
4.1.3. Seismic restrainers

Although Japanese seismic restrainers differ significantly in appearance from those developed in California, they essentially serve the same purpose. A number of different designs have apparently been used (as shown in Fig. 5), all intended to provide restraint in the longitudinal direction. In the Kobe region, a very frequently used restrainer type consisted of rectangular plates with slotted holes connected to each girder by a single jumbo-bolt (Fig. 17). Another commonly used restrainer type consisted of plates connected to one beam using multiple high strength bolts, and to the other beam using a slotted hole and jumbo-bolt (Fig. 18). Variations on the same principle were also used (Fig. 19). Many restrainers were observed to have worked effectively during this earthquake, preventing simply supported spans from falling from their supports. Numerous seismic restrainers showed signs of plastic yielding and (or) buckling (Fig. 19). Others were strained to their limit, often due to excessive substructure displacements, and failed (Figs. 17 and 18).

4.1.4. Bearing failures

Bearings suffered a considerable amount of damage during this earthquake. They frequently were the second structural element to fail following major substructure damage, but in some cases, they were observed to have failed in spite of the substructure remaining intact. Roller supports proved particularly vulnerable (Figs. 20 and 21), as their design provided a limited resistance against seismic forces applied laterally. Fixed supports at end-spans also frequently suffered damage (Figs. 22–24). In many of those instances (e.g., in Fig. 22), the bearing anchorage to the concrete base was significantly stronger (in bolt numbers and size) than the steel-to-bearing anchorage; as a result, the bolts connecting the girders ruptured, and girders slipped-off their bearings. The lateral

Fig. 12. Mild local buckling of a circular steel column: (a) global view; (b) close-up view.

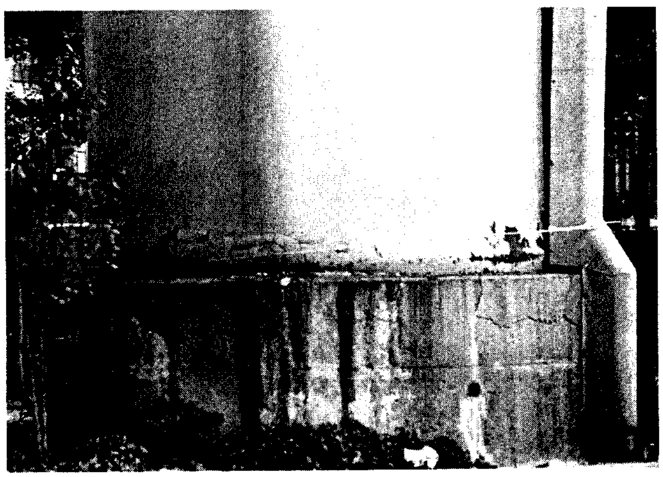


displacement observed for bridge spans that fell off their bearings was sometimes impressively large (Fig. 25), sometimes even producing localized severe lateral-bending of the steel girders and even rupture of the end-diaphragms. Failure of the bearing anchorage sometimes occurred in the concrete (Fig. 26), although in some cases this may have been precipitated by pounding from the adjacent span. Finally, the failure of stoppers whose sole purpose is to prevent displacement and unseating of moveable bearings was also observed (Fig. 27).

4.1.5. Indirect damage

Damage to steel bridges was sometimes not a consequence of structural deficiencies, but rather of peripheral issues. For example, although severe buckling can be observed at the end of the bridge of Fig. 28, it is a consequence of abutment failure. Lateral spreading of the soil–abutment system exerted a downward pressure on the steel superstructure, "crushing" it on its bearings.

Fig. 13. Mild local buckling of a circular steel column, resulting in "elephant-foot" type damage.



4.2. Long-span bridges

Reconnaissance visits were made to four long-span steel bridges that provide water crossings along the Wangan Route 5 Expressway (three bridges) and to artificially created Rokko Island (one bridge). Two are cable-stayed bridges (the Higashi–Kobe Bridge and the Rokko Island Connection Bridge (which will be referred to here as the Rokkoliner Bridge to avoid confusion with an arch bridge of a similar name)), and two are arch bridges (Rokko Island Bridge and Nishinomiya Port Bridge). Their locations are shown on the map in Fig. 1.

4.2.1. Higashi–Kobe Bridge

The Higashi–Kobe Bridge is a landmark structure of the east Kobe waterfront and the Wangan Route 5. This cable-stayed bridge has a continuous steel double-deck Warren truss with spans of 200–485–200 m. The deck width is 13.5 m. The H-shaped steel towers have straight legs that are 146.5 m in height. The use of steel towers for cable-stayed bridges is one of the distinctive features of cable-stayed bridge engineering in Japan. Cables are arranged in a harp pattern. The tower foundations are pneumatic caissons. The bridge, completed in 1992, is shown in Fig. 29. A unique feature of the seismic design is that the main girder can move longitudinally at all supports, resulting in a long sway period. Pendulum supports are provided at the end and intermediate sidespan piers to provide this longitudinal freedom (the top of one of the pendulums is shown in Fig. 29). Newly developed vane-type dampers are installed on the end piers to provide longitudinal displacement control during an earthquake.

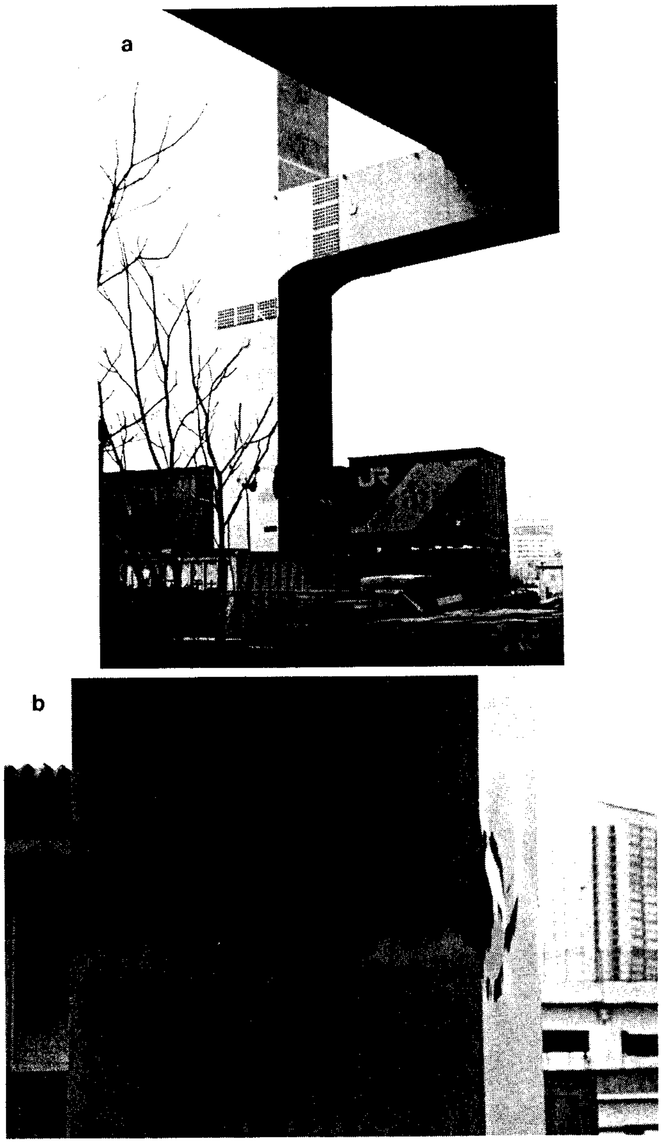


Fig. 14. Local buckling of a rectangular column at the level of the access hatch door: (*a*) global view; (*b*) close-up view.

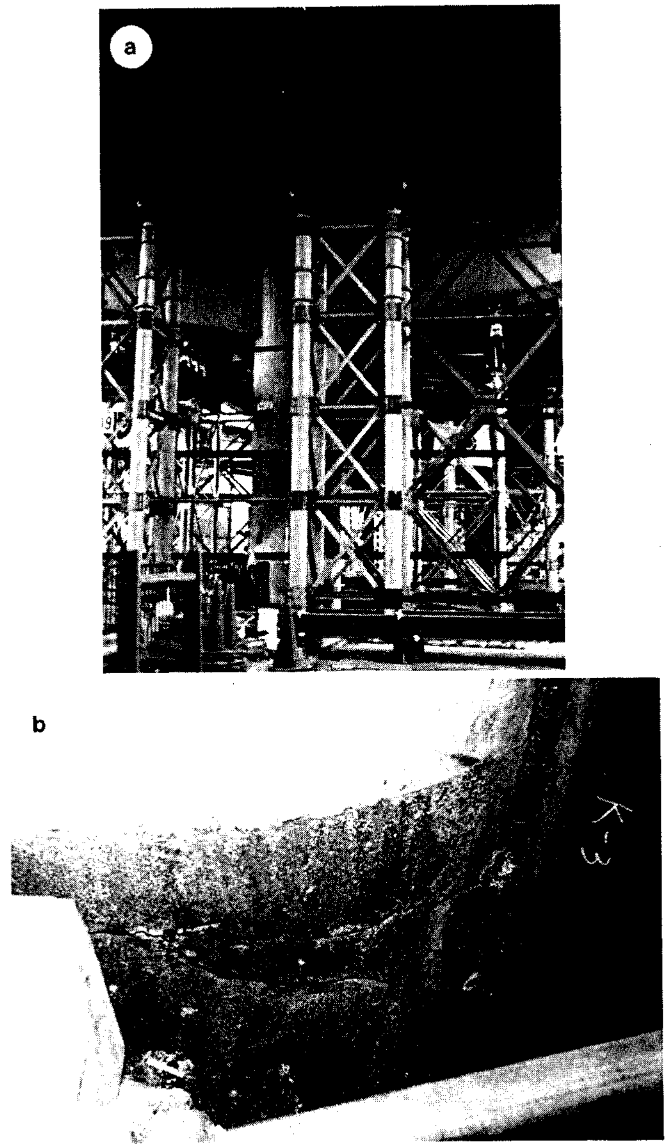


Fig. 15. Fracture of steel in a rectangular steel column at the level of concrete fill: (*a*) global view of the column and temporary supports; (*b*) close-up view of fracture.

The principal design reason for the intermediate sidespan piers is to control live-load deflections of the 485 m centre span. Wind shoes are used at all deck supports to transmit transverse loads directly to the towers and piers.

Viewed from a distance the bridge appeared to have survived the earthquake quite well. However, closer inspection of the bridge revealed damage that appeared to be confined to the piers and to the connection of the deck to the end pier. Figure 29*d* shows the west end pier. Damage to this steel pier included shear-induced buckling at the midpoint of the cross-beams, and compression buckling of one of the lower A-frame legs. Consistent with the compression buckling of the leg was a large crack (not shown in photos) in the concrete pier foundation on the adjacent tension leg. The intermediate sidespan pier also sustained shear-induced buckling in its lower cross beam. Shear-induced buckling was also visible in the approach spans to the bridge (Fig. 30).

At the west pier the connections to the deck of the two pendulum tension links, one wind shoe, and two oil dampers were all severed. Figure 31 (a close-up of Fig. 29) shows details of damage to the pendulum connection. The upper end of the pendulum (one pendulum on each side of the deck) was pin-connected to restrainer plates welded to the deck. It appears that transverse oscillations of the deck resulted in the plates bending outwards so that they no longer restrained the pins in the eyes of the pendulum, and the pendulums then fell free of the deck, as shown in Fig. 31.

Failures of the wind shoe and oil damper connections are shown in Figs. 32 and 33. The vertical plate in Fig. 32 was originally oriented 90° to the position shown in the photograph and was bolted to the underside of the deck. On the reconnaissance visit it was noted that the top of the pier was covered in oil that had escaped from the dampers.

Severance of the deck-to-end pier connections allowed the tension in the cables to lift the end of the deck by approxi-

Fig. 16. Fracture of a steel column supporting a railroad bridge: (*a*) global view; (*b*) close-up view of the fractured section.

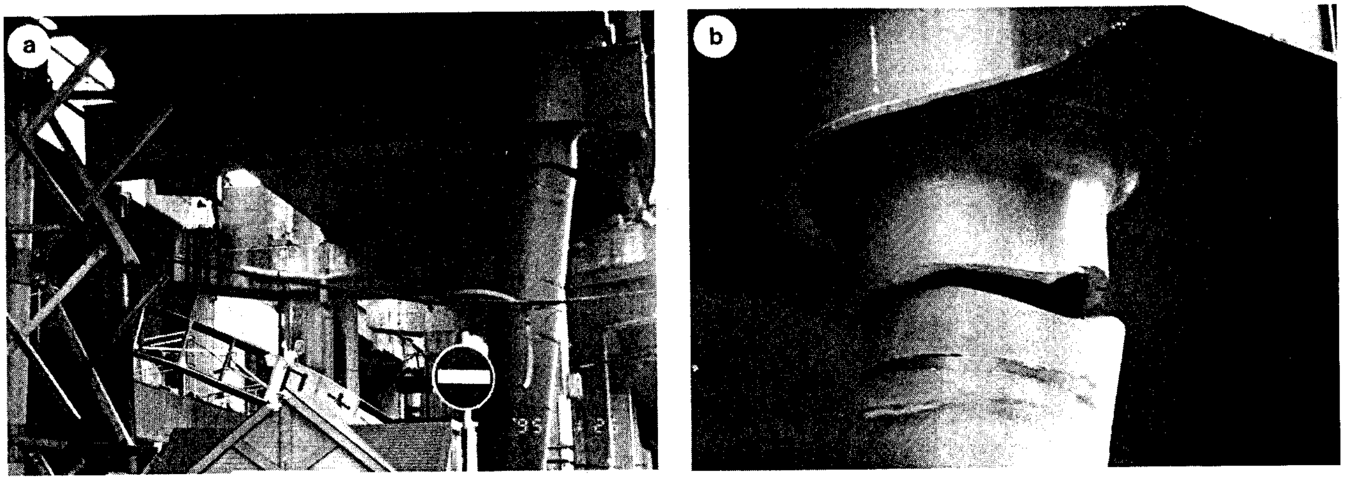
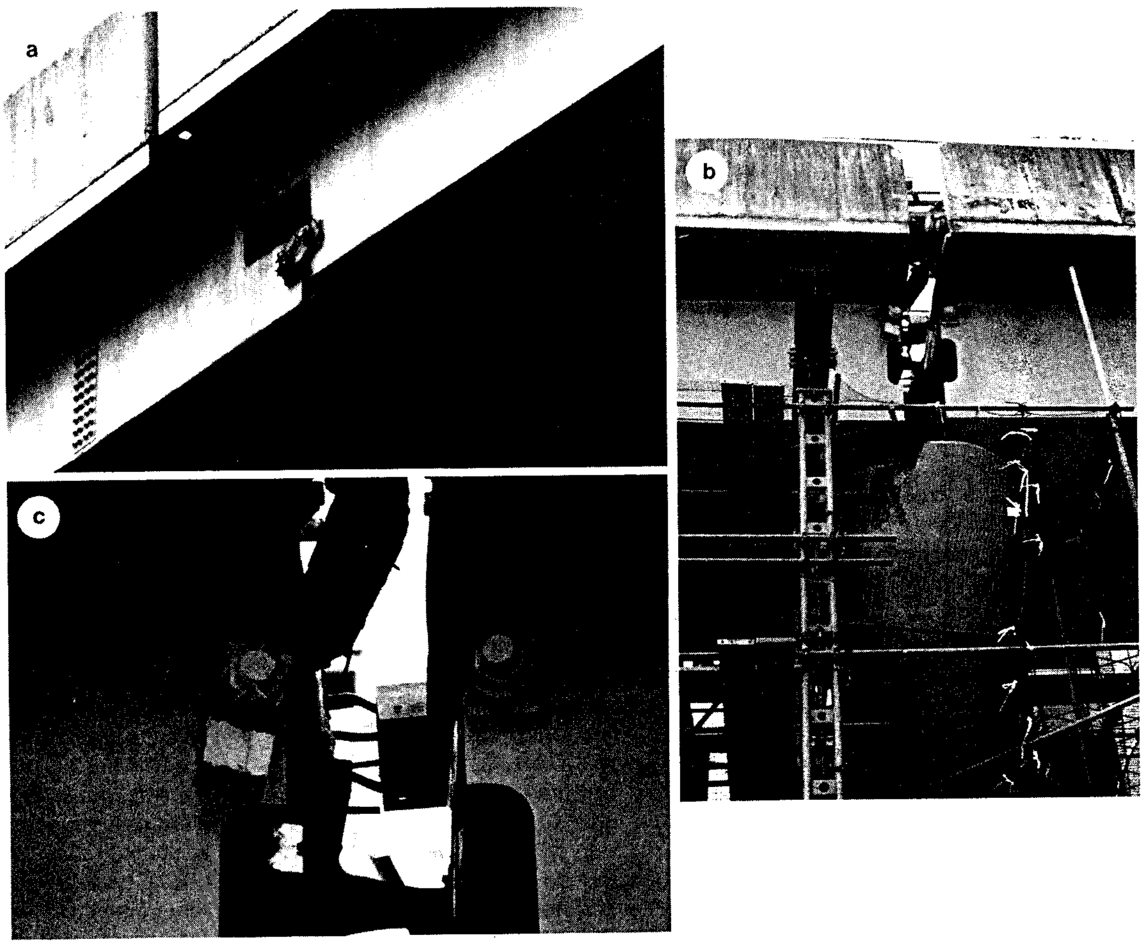


Fig. 17. Example of dual jumbo-bolts restrainers: (*a*) example of satisfactory performance; (*b*) example of ruptured restrainer; (*c*) close-up of ruptured restrainer.



mately 0.5 m. This is shown in Fig. 31.

Figure 34 shows damage to one of the links connecting the approach span to the end pier (this link is behind the oil damper shown in Fig. 33). The link appears to have been damaged by tension between the end pier and the approach span. Although damaged, the link remained intact.

4.2.2. Rokkoliner Bridge (Rokko Island Connection Bridge)
This double-deck cable-stayed bridge (Fig. 35) provides both rail (hence the name Rokkoliner) and vehicular (truck and automobile) links to Rokko Island. Vehicular traffic lanes occupy the upper and lower decks, while rail traffic operates on separate spans outside the upper deck level. The cable-

Fig. 18. Example of single jumbo-bolts restrainers: (*a*) example of satisfactory performance; (*b*) example of ruptured restrainer.

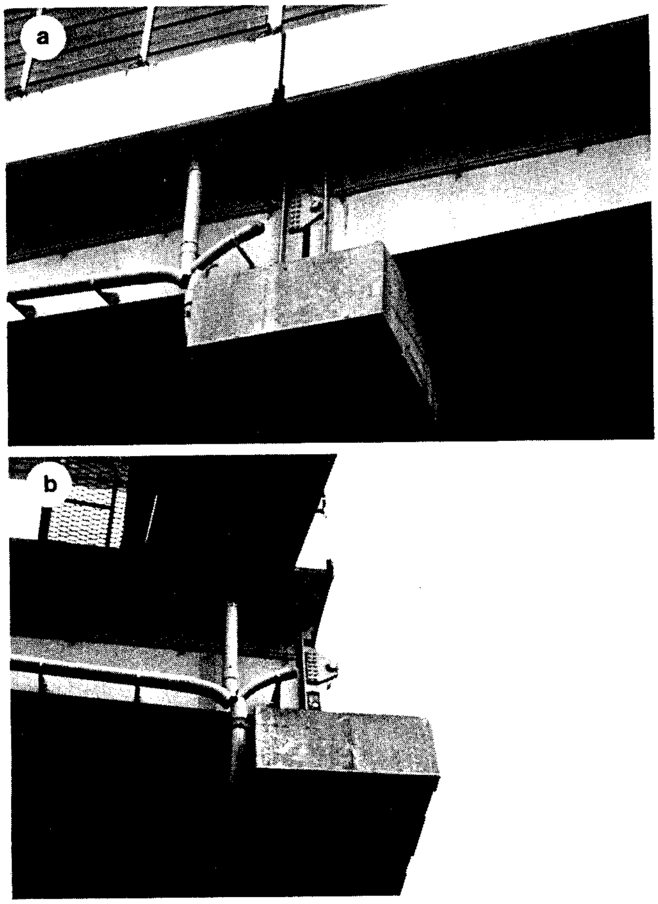
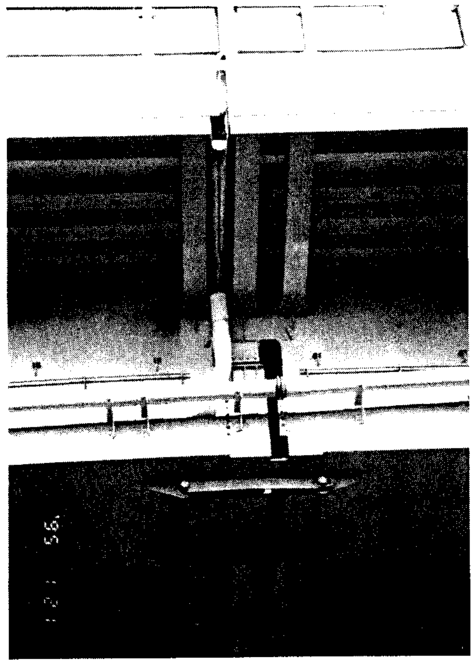


Fig. 19. Dual jumbo-bolts restrainers with long plates, showing evidence of plate yielding and buckling.



stayed segment has three spans of 90–220–90 m. Construction was completed in 1977, although the rail service appears to have been added at a later date.

The most significant damage to this structure was the collapse of one of the rail approach spans on the Rokko Island side. Figure 36 shows the approximately 1 m of ground settlement that occurred as a result of outward movement of the quay wall. This settlement caused one of the approach span piers to tilt, resulting in the collapse of the simply supported rail span, shown in Fig. 37. Figure 38 shows a close-up of the failed restrainer on the collapsed span.

An eastward offset of the deck of approximately 20 cm was observed at the Kobe (north) end of the bridge. There was also failure of the supports for the utility services carried on the bridge. After the earthquake the bridge was open to limited vehicular traffic.

4.2.3. Nishinomiya Port Bridge

This bridge is a 252 m single-deck cable-arch bridge spanning the entrance to Nishinomiya Port. The deck width varies from 27 to 31 m. An overall view of the bridge is shown in Fig. 39. The bridge was completed in 1993. The 52 m eastern approach span slipped off its bearings and collapsed. This was apparently a result of tilting of the arch pier, caused by large deformations in the soil at the base of the pier and immediately adjacent to the quay walls along the port entrance channel. Two single-bolt restrainers had been provided for each box-girder of the span at that location, but the connecting plates on the approach steel girders tore out. Views from the bridge deck and ground (Fig. 40) show the seating details for the collapsed span, and the fractured tension links. Bearing failure also took place further east along the approach spans, where bearing components could be observed on the ground, but girders did not collapse.

At the west end of the bridge, the cast bearings supporting the arch span were fractured, as shown in Fig. 41. The longitudinal motion of the arch causing fracture of the bearing is evident in this figure. One cable (Fig. 42) was slack, apparently as a result of a failure in the deck anchorage. All the other cables appeared to be intact.

4.2.4. Rokko Island Bridge

This 217 m double-deck steel arch bridge, completed in 1992, is shown in Fig. 43. The width of the arch is 16 m at the south (Rokko Island) end and 26.5 m at the north end to accommodate entry and exit ramps on the north side.

The south end of the arch was displaced eastward by approximately 2.5 m. This left the south end of the eastern arch unsupported by the pier. This dramatic offset is shown in Figs. 43 and 44. The approach spans were dragged eastward by the displacement of the arch and contacted the upper column on the approach pier, as shown in these two figures. Several members of the wind-bracing system connecting the arches also severely buckled (Fig. 45).

Shear-induced buckling was observed in the cross beams of several of the piers (although it is rather difficult to see in Fig. 44), similar to that observed on the approach piers of the Higashi–Kobe Bridge. However, for this bridge, buckling on the south approach piers is one way, consistent with a large force applied in an easterly direction at the top of the pier, the direction in which the arch displaced.

Large ground settlements occurred on the south approach

Fig. 20. Damaged roller bearings of steel girders: (*a*) damaged roller bearing; (*b*) fallen roller; (*c*) span fallen off the supports after failure of one bearing; (*d*) complete span failure after bearing rupture, with the column now directly supporting the deck.

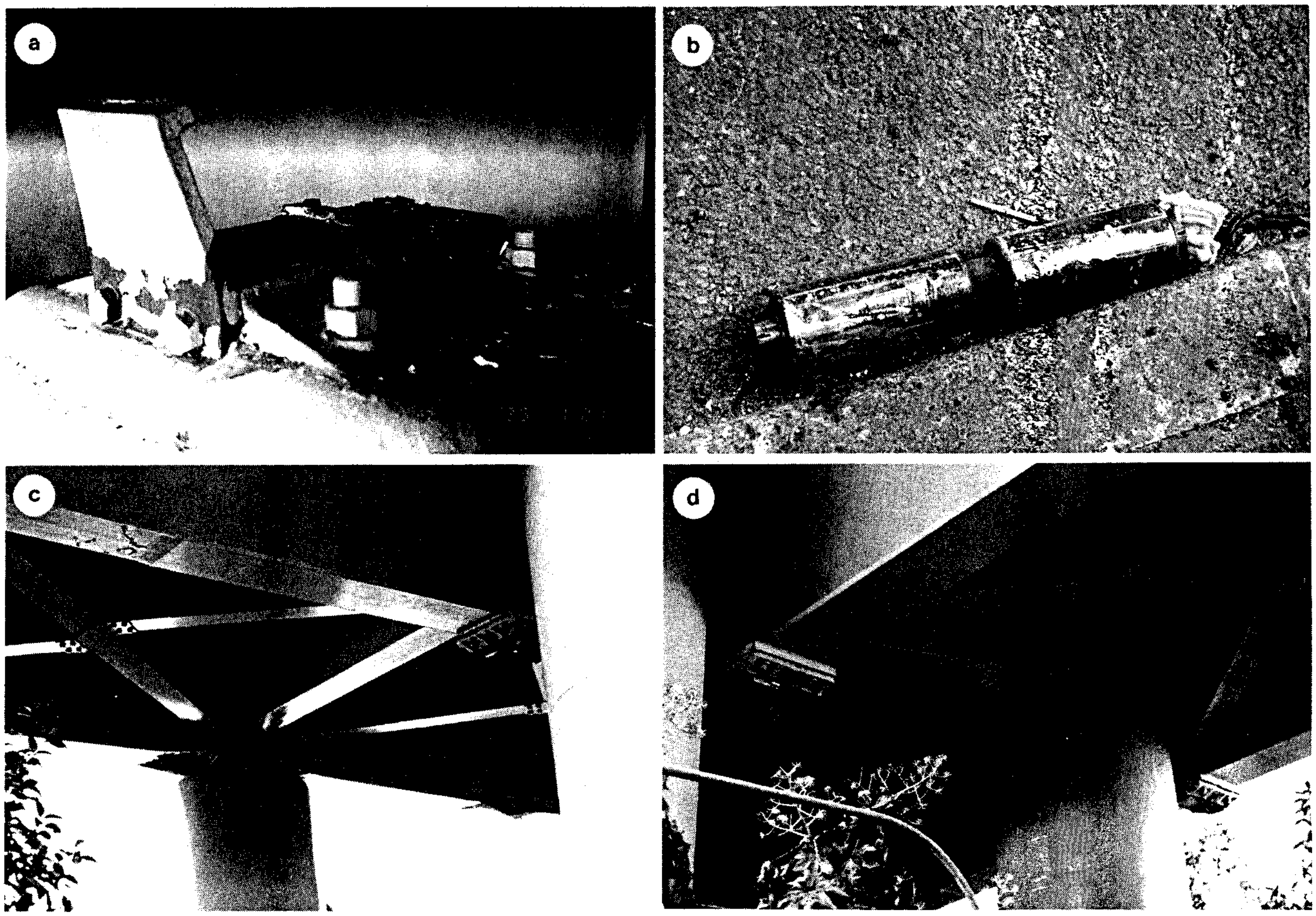
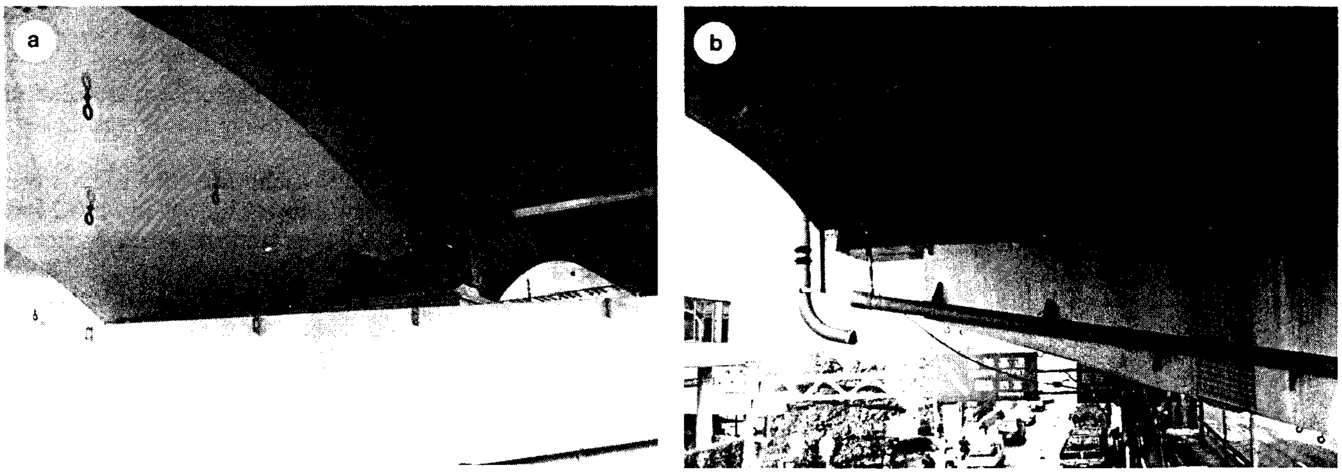


Fig. 21. Damaged roller bearing on a steel box-girder: (*a*) close-up view; (*b*) span offset.



due to outward movement of the quay walls. However, unlike the Rokkoliner Bridge there did not appear to be any appreciable rotation at the foundations of the approach piers. This is significant because the approach spans of both this bridge and the Rokkoliner Bridge are simply supported, and hence susceptible to loss-of-span failures, as happened to Rokkoliner.

4.2.5. Akashi Strait Bridge

This suspension bridge (part of the Honshu – Shikoku bridge network), still under construction at the time of the earthquake, will link the central Japan island of Honshu with Shikoku through Awaji Island located at the northeast tip of Shikoku. When it opens in spring 1998, it will be the world's longest suspension bridge (1990 m centre span, 3910 m

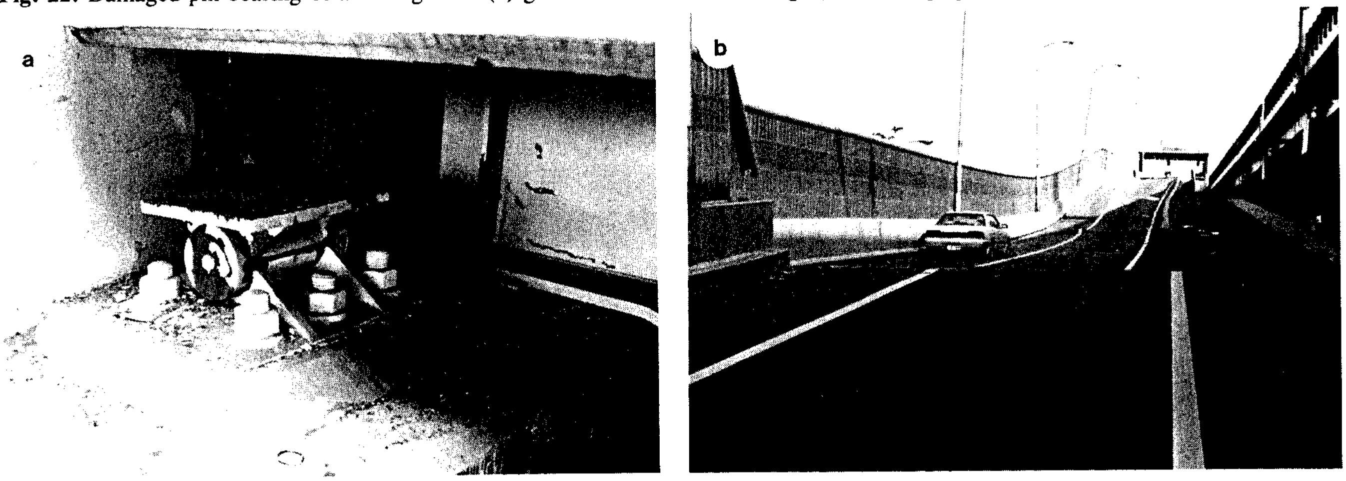


Fig. 22. Damaged pin bearing of a steel girder: (*a*) girder fallen off the bearing; (*b*) resulting span offset and damage.

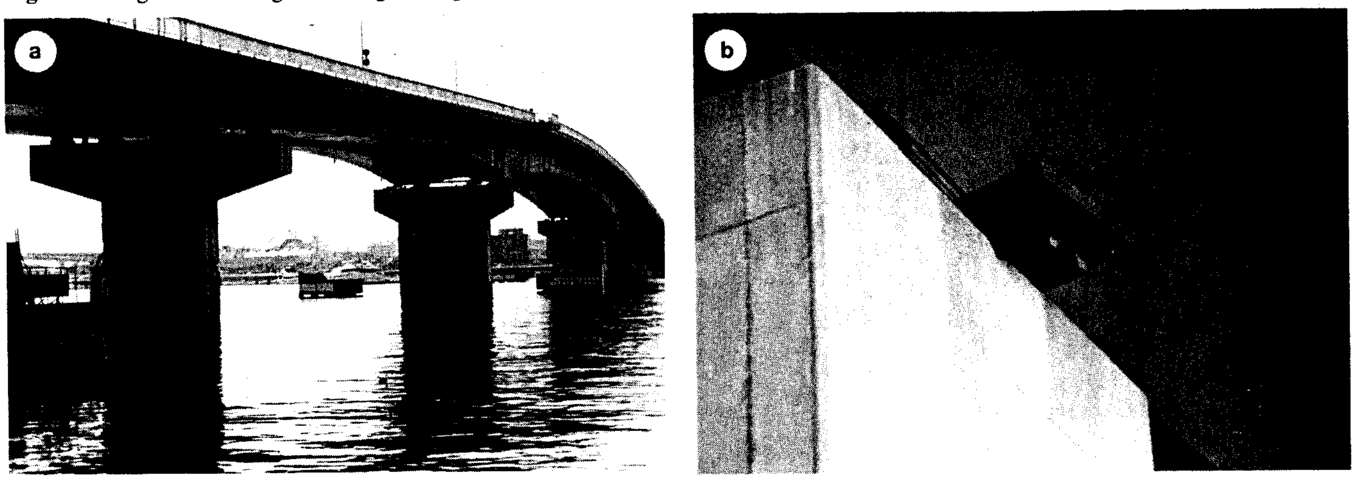


Fig. 23. Bridge with damaged bearings: (*a*) global view of the bridge; (*b*) close-up view of pin bearing damage.

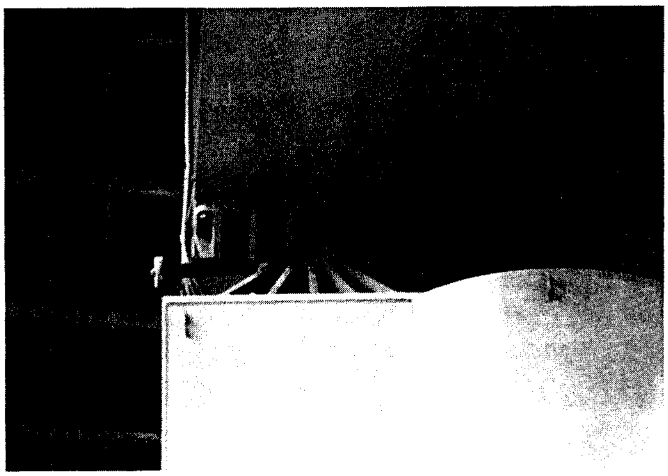
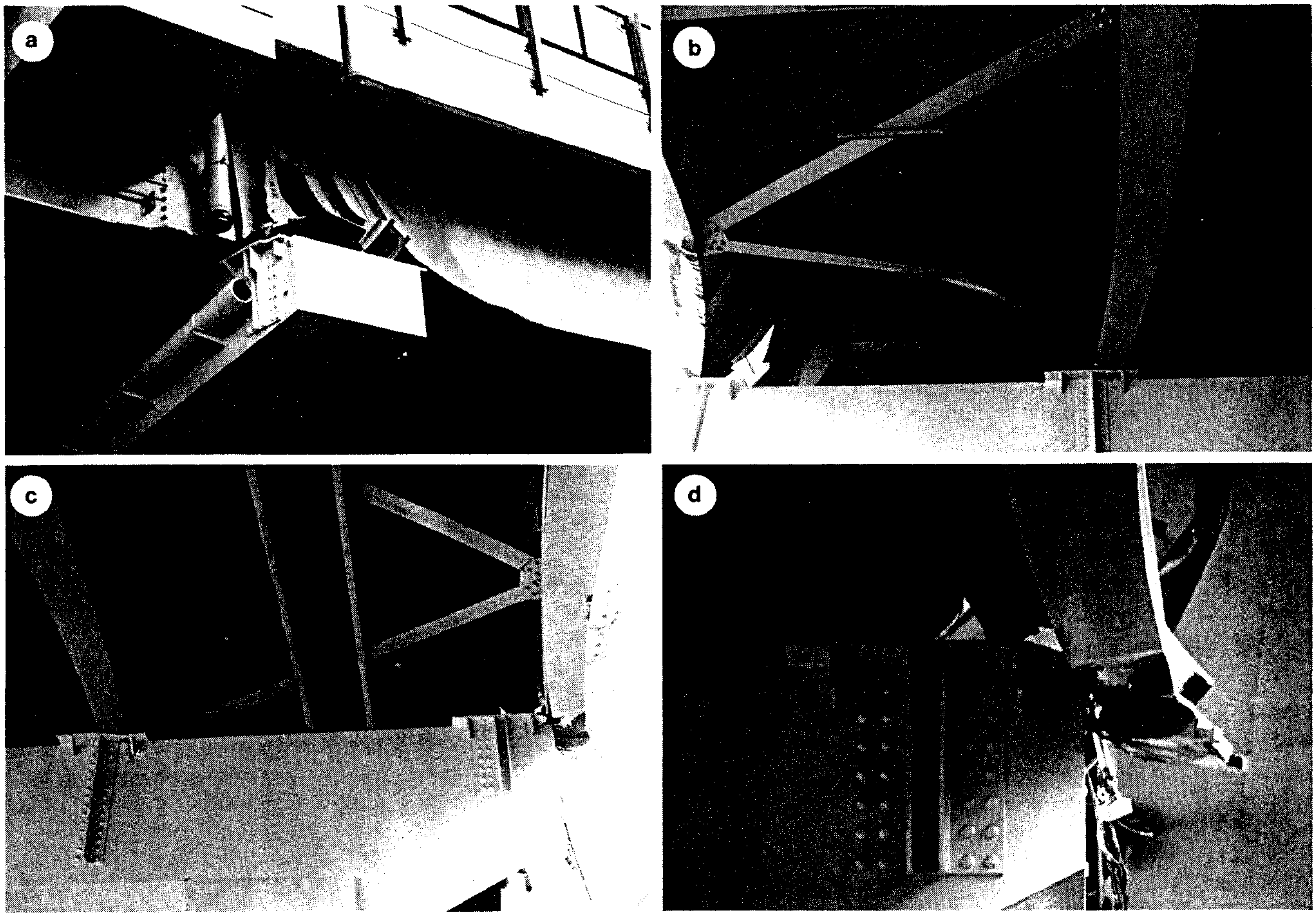


Fig. 24. Example of pin bearing damage, by failure of keeper plate.

total). Interestingly, the epicenter of the Hyogo-ken Nanbu earthquake was located approximately 2 km southwest of the southern end of this bridge. As only the towers and suspended cables had been erected prior to the earthquake, no structural damage was reported. However, the towers and end anchorages all moved from their original position. Submersible video cameras and sonic exploration revealed no traces of foundation sliding, suggesting that it was the supporting ground that displaced.

With respect to the south anchorage, the south tower moved west 0.21 m, the north tower moved down 0.24 m, north 0.84 m, and east 0.34 m, and the north anchorage moved up 0.16 m and north 1.12 m. Moreover, small tower rotations produced additional northward displacements at the top of the south and north towers of the order of 0.11 and 0.07 m respectively (note that maximum top-of-tower deflection under traffic loads and wind and thermal effects is expected to reach 1.9 m). As a result of these displacements, midspan cable sag increased by 1.27 m, and the centre span lengthened to 1990.80 m (pushing the world record for the longest span by another 0.8 m). Fortunately, these slight changes in geometry and minor towers misalignment occurred before any piece of the stiffening deck had been constructed, and could all be accommodated by redesign. Beyond this inconvenience, construction resumed almost as planned; installation of the stiffening truss in 2700 t segments (84.6 m long, 35.5 m wide, and 14 m high), lifted by floating crane, began June 6, 1995.

Fig. 25. Large transverse displacements as a consequence of bearing failures: (*a*) severe damage to end of steel girders due to bearing failure (Hanshin Expressway); (*b*) underside view of same, showing tear-up of end-span diaphragm; (*c*) damage at the other end of the same pier; (*d*) close-up view showing the extent of lateral displacement; (*e*) nearly collapsed span of Port Liner rail bridge; (*f*) unseated continuous span at the second level of Harbour Highway; (*g*) view at the continuous support; (*h*) view at the simple support; (*i*) another unseated span of the same highway, also showing severe distortion damage to end of steel girders.



4.2.6. Moderate-span cable-stayed bridge
Minor damage also occurred to the main tower of a moderate-span bridge near downtown Kobe. The single tower of that unsymmetric bridge was damaged at the deck level, and can be seen leaning in Fig. 46.

4.3. Examples of satisfactory performance

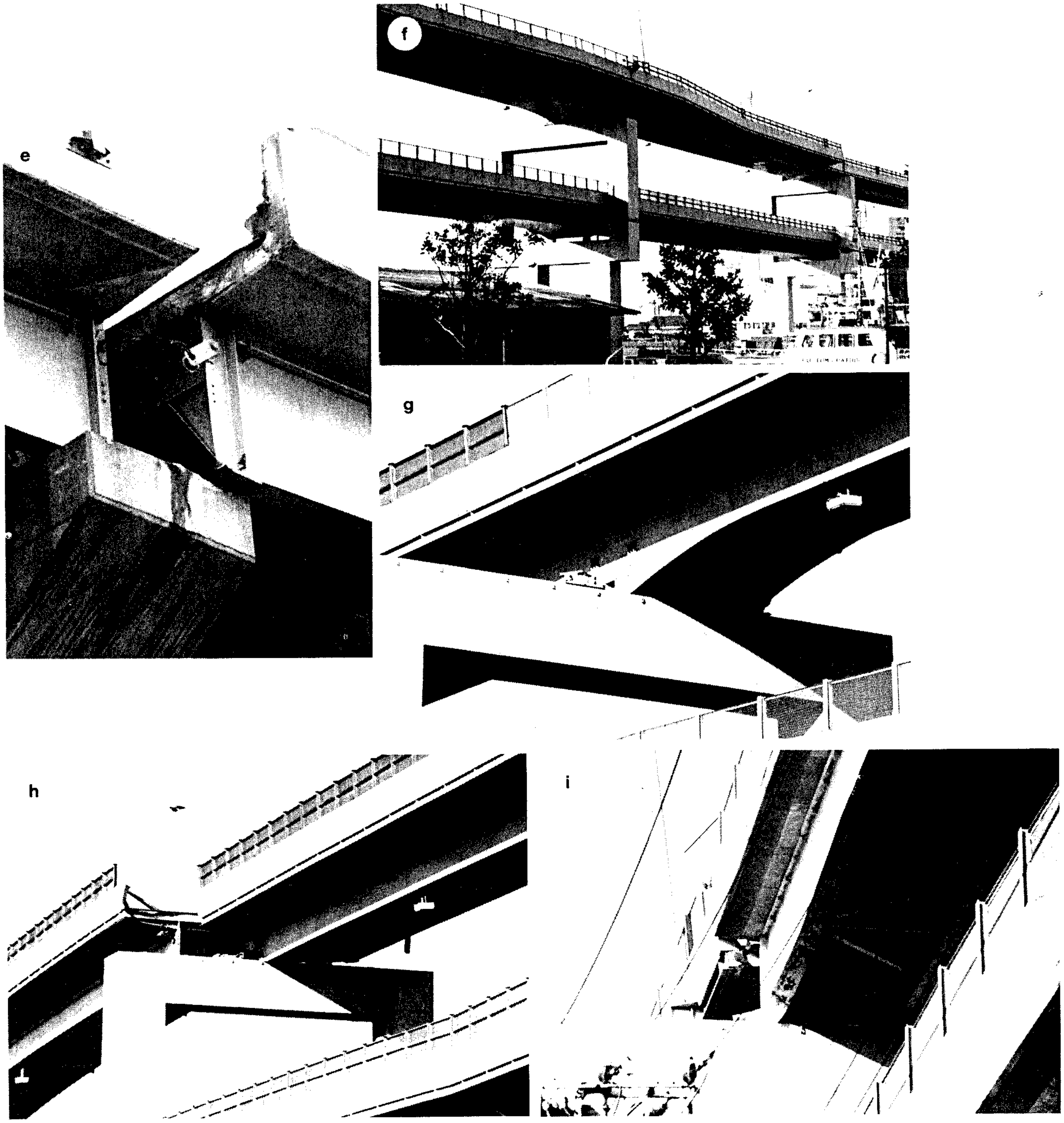
In many cases, steel columns have provided a quite satisfactory seismic performance, even in parts of Kobe where numerous buildings suffered severe damage. The elevated Harbour Monorail Line provides a good example of this excellent behaviour. Surrounded by damaged buildings, the steel columns of the monorail remained intact. It is noteworthy that in one particular segment of this elevated monorail, six concrete columns were used instead of steel columns; all these columns suffered severe damage or collapsed, triggering span collapses and rendering the line inoperable (Fig. 47).

Spans having rigid steel frames made of compact wide-flange sections oriented to provide strong axis bending against lateral excitations also appear to have provided an excellent seismic resistance, with no visible superstructure or bearing damage (Fig. 48*a*). By contrast, columns pinned at both ends evidently provided no contribution to the lateral load resistance, which sometimes proved critical for narrow railroad-type bridges (Fig. 48*b*).

No diaphragm failures of the type observed during the Northridge earthquake were observed by the authors following this earthquake. However, as is visible on some of the preceding figures, Japanese steel bridges seem to have been constructed with solid diaphragms, extensive underside lateral bracing, or both.

Finally, it must be said that although the Japanese had already initiated a bridge seismic retrofit program prior to the Kobe earthquake, no bridge had yet been retrofitted in the Kobe area (believed to be exposed to a lower seismic risk than other parts of Japan). However, in the "post-Kobe" climate, the Japanese bridge retrofitting activities are now considerably accelerated; for example, seismic retrofit of the elevated highways in the Tokyo area is currently scheduled to be entirely completed before the summer of 1996 (Kawashima, personal communication).

Fig. 25 (*concluded*).

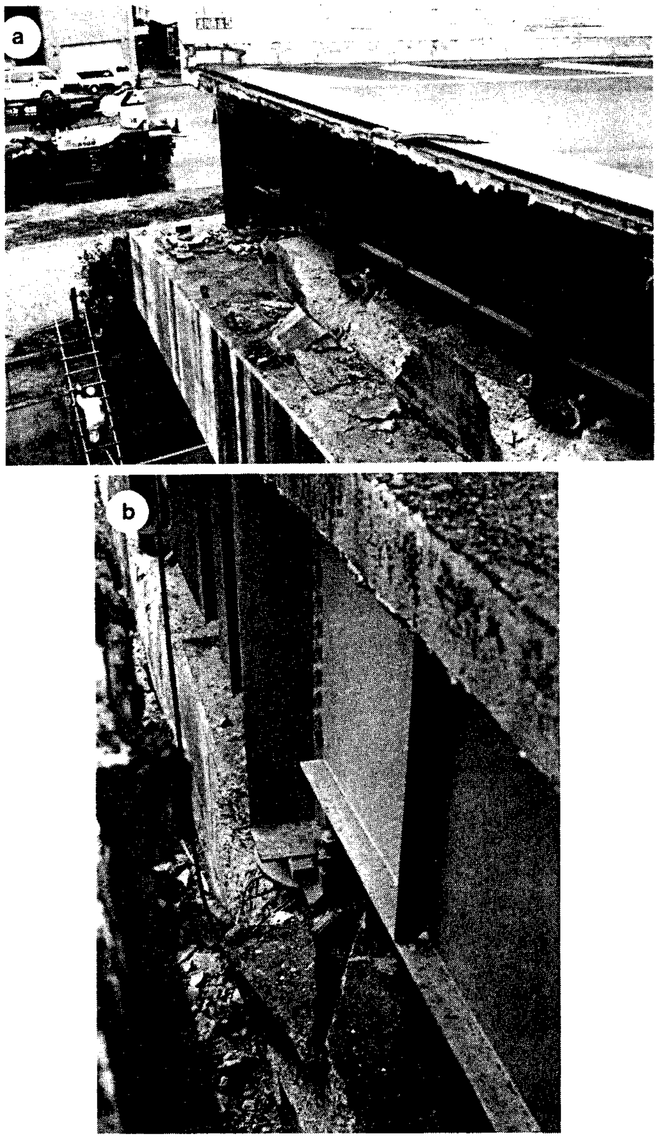


5. Relevance to Canadian practice

5. 1. Short- and medium-span highway bridges — current infrastructure and design practice

In Canada as well as in Japan, most short- and medium-span steel highway bridges have been built in the last 40 years, with the difference that bridges in Japan are, on average, more recent, since construction of the Japanese road infrastructure network in major urban centres has started somewhat later and is still in progress. In both countries, concrete slab on steel girders has been the most common superstructure type for steel bridges, and these spans have predominantly been supported on concrete abutments and piers, although steel columns and piers have also been used (more frequently so in Japan). Until the 1970s, designers mainly used simply supported I-shaped girders for both single-span and multiple-span construction. Longitudinal expansion joints were provided at each support, with a fixed bearing at one end of the span and a moveable bearing free to move longitudinally at the other end. In more recent structures, although I-shaped girders are still common, box girders have become more popular, and continuous girders and deck slab

Fig. 26. Damage concrete support of bearings: (*a*) view at end of span; (*b*) close-up view.

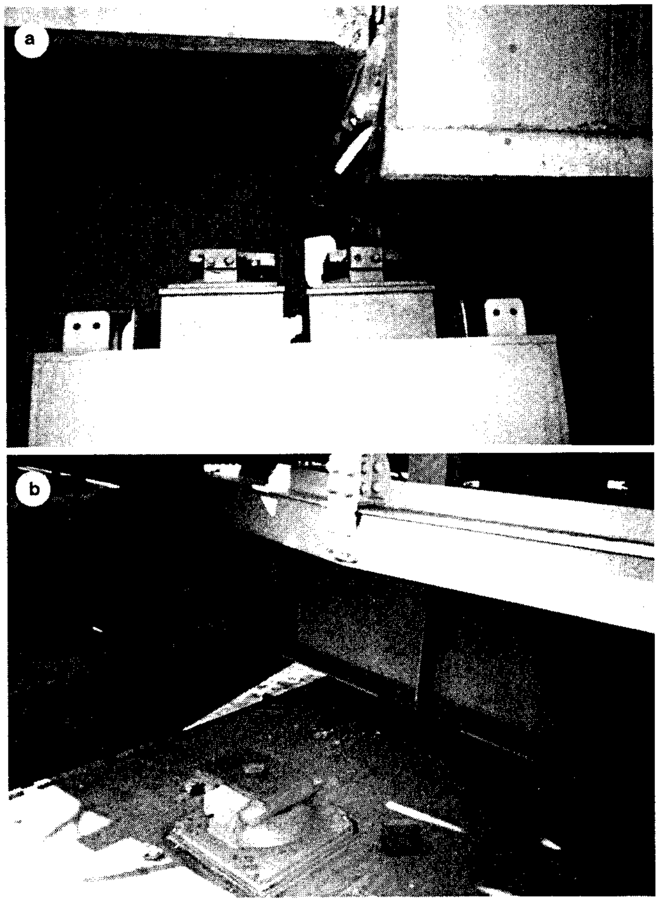


systems have nearly become the norm in multiple-span applications further to the developments of computer programs that have facilitated their analysis.

In those instances where the supporting structure is made of steel, the Canadian and Japanese practices differ greatly. In Canada, frames that include tapered wide-flange beams and columns, with curved flanges at joints, have been used for overpasses. These frames are sometimes braced in the transverse direction. In both countries, the steel substructure sometimes consists of concentrically braced frames, truss systems, or moment resisting frames made of standard structural shapes, although the braces and moment frame members designed to resist lateral loads are always heavier and more numerous (to provide redundancy) in the Japanese bridges. The use of large rectangular and circular tubular piers for short- and medium-span bridges, extremely popular in Japan, is uncommon in Canada.

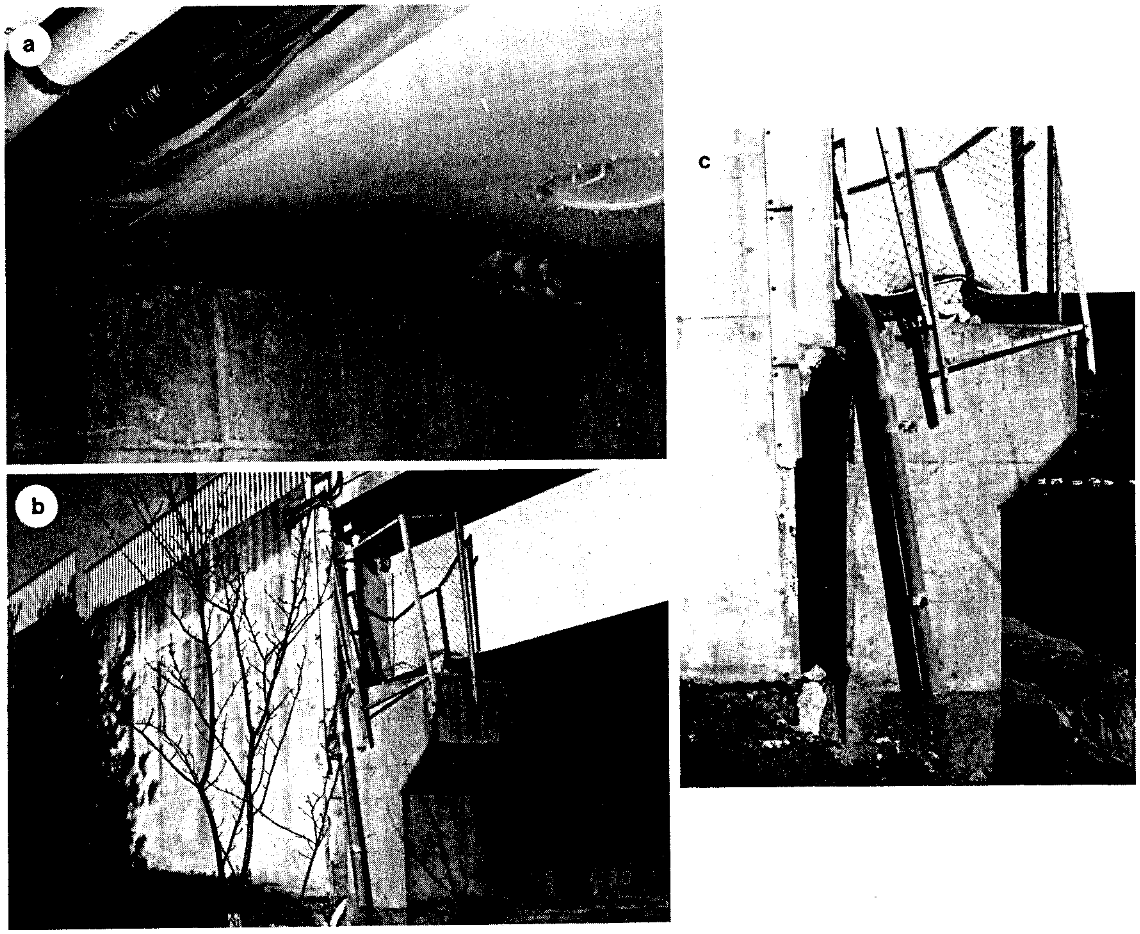
Similarities also exist between the design procedures

Fig. 27. Damage to stoppers at movable bearing: (*a*) sidesways displacement of spans as well as vertical offset between adjacent spans; (*b*) severe lateral displacement at end-span and ruptured connection to the stoppers.



adopted in the two countries. In Canada, the CSA-S6 *Canadian Standard for the Design of Highway Bridges* and the *Ontario Highway Bridge Design Code* (OHBDC) have been the primary reference code documents in the past decades (CSA 1966, 1978, 1988, 1994; OHBDC 1979, 1983, 1991). These codes have traditionally adopted a simplified equivalent static load method to provide resistance to earthquake ground motions. This approach is almost identical to the method followed in the Japanese specifications, but the level of seismic loads used in Canada has apparently always been smaller than in Japan for comparable seismic zones. For example, in Victoria, B.C., where the seismic risk approximately corresponds to that in Japan, the horizontal seismic coefficient specified in the 1966 edition of the CSA-S6 standard (CSA 1966) lay between 0.02 and 0.06, depending on the soil conditions and the type of foundation; that is considerably smaller than the values ranging between 0.20 and 0.35 that were used in the 1960s and 1970s in the Kobe area. In 1978 (CSA 1978), the Canadian seismic coefficients were slightly increased, up to 0.06 and 0.08 for Victoria, depending upon the type of substructure. In the current edition of the CSA-S6 standard (CSA 1988), the seismic coefficient, including the load factor of 1.3, now lies between 0.12 and 0.23 for the same city, depending on the importance of the

Fig. 28. Indirect damage to a steel box-girder bridge due to abutment movement: (*a*) damage to box-girder at supports; (*b*) global view of the displaced abutment; (*c*) abutment and support movements.



bridge, the type of substructure, and the soil conditions. The latter values are closer but still generally lower than those prescribed in the 1990 edition of the Japanese specifications (described in Sect. 3.1).

Nonetheless, all the above code-specified seismic loads are lower than those necessary to ensure elastic seismic response, and structural damage is an implicit consequence of those design criteria. However, as is also the case in the Japanese codes, no specific detailing requirements nor explicit capacity design rules have been included so far in the Canadian codes to ensure proper ductile behaviour of steel substructures. The first reference to ductile bridge response in Canada appeared in the 1983 edition of the OHBDC (OHBDC 1983), but was limited to concrete structural elements.

The design procedure for fixed bearings in Canada has been nearly the same as the design practice adopted in Japan. However, up to 1988, the force level prescribed in the CSA-S6 specifications corresponded to that used for the entire bridge. In the CSA-S6-88, the design force for the bearings was raised to twice the load used for the bridge (but not exceeding 25% of the weight of the connected superstructure), which coincides with the approach used in Japan.

Provisions to accommodate expected movements at expansion joints were included for the first time in the 1983 edition of the OHBDC and the 1988 edition of the CSA-S6 standard. In these documents, it is stated that the joint be designed to undergo, without span collapse, a displacement equal to 6 times the deformation produced by the prescribed seismic loads. Such amplified deformation was assumed to be representative of the total elastic and inelastic deformation experienced by the structure under the design earthquake. The first empirical expression for minimum seat-width was introduced only in the 1991 edition of the OHBDC (OHBDC 1991). Thus, for steel bridges built before the mid-1980s, which incidentally represents the majority of the steel bridges erected in Canada, no comprehensive design requirement was available to structural engineers to prevent the failure of spans due to unseating. As opposed to Japan where the installation of restraining devices has become mandatory since 1971 (most old bridges inspected in Kobe had restrainers when needed to overcome narrow seat-width problems), seismic retrofit efforts have been initiated only very recently in Canada and the problem still needs to be addressed in most of the older bridges. Currently, neither the OHBDC 1991 nor the CSA-S6 bridge design codes have provisions for restrainers. This shortcoming is somewhat accentuated by the fact that predictions of the actual movement of the superstructure in the longitudinal direction based on code-specified loads can be greatly underestimated.

Fig. 29. Higashi–Kobe Bridge: (*a*) elevation view (Hanshin Expressway Public Corporation); (*b*) global view; (*c*) west end pier of Higashi–Kobe Bridge showing damaged pendulum connection at the top of the pier, and compression buckling in the lower left leg on the far side; (*d*) west end pier of Higashi–Kobe Bridge showing panel buckling of the cross-beam and compression buckling of the lower left leg (concrete footing on the far left leg has a large tension crack that is not visible in this photograph).

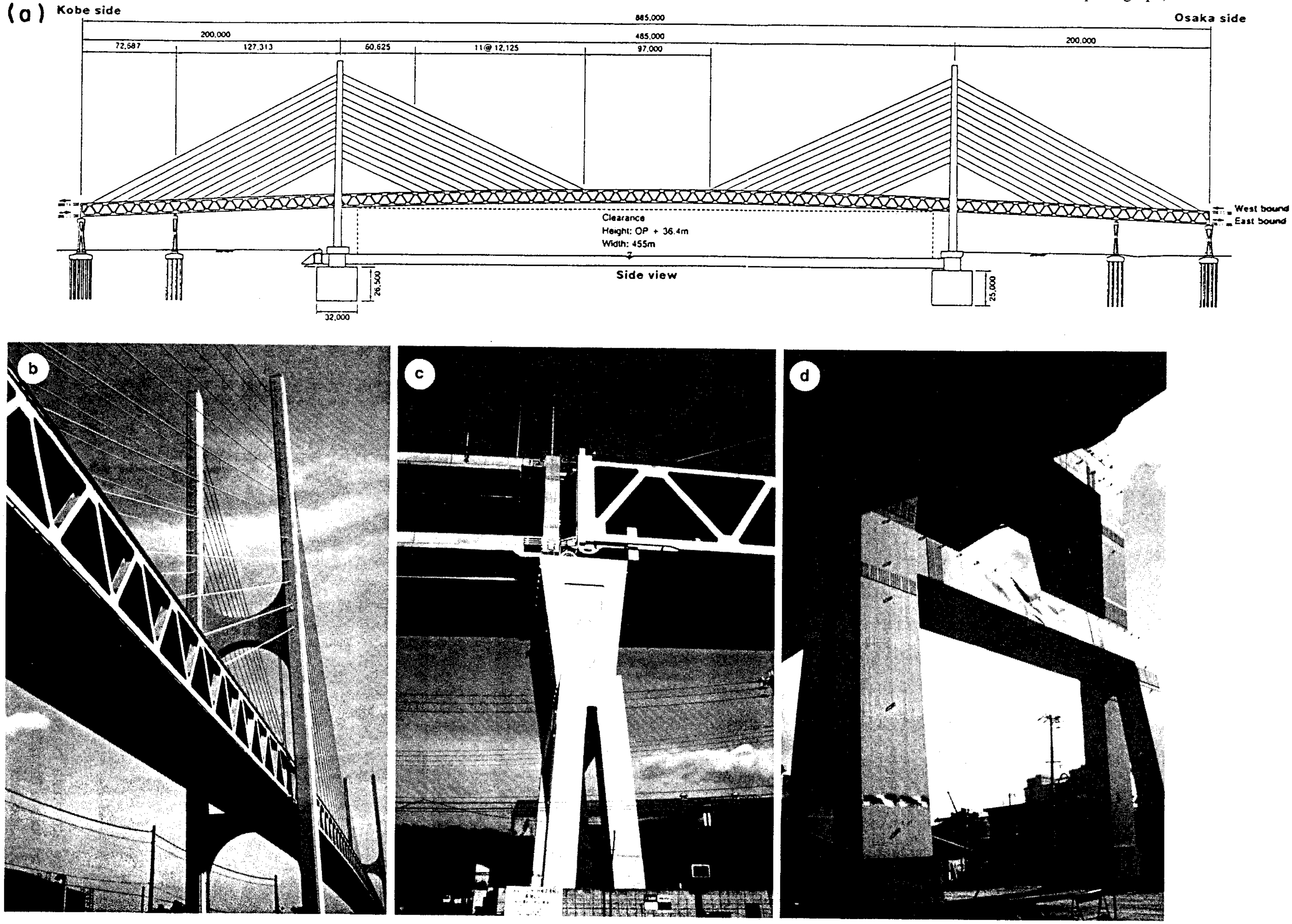


Fig. 30. Panel buckling in cross beams of the approach spans of Higashi–Kobe Bridge.

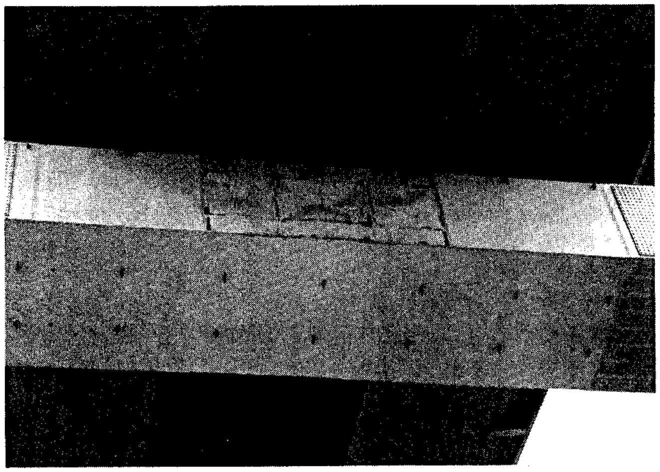


Fig. 31. Pendulum at the west end pier on the Higashi–Kobe Bridge: (*a*) damage to the pendulum connection viewed from the underside of the deck of the Higashi–Kobe Bridge; (*b*) close-up showing the top eye of the pendulum bearing and the outward bent pin restrainer plate.

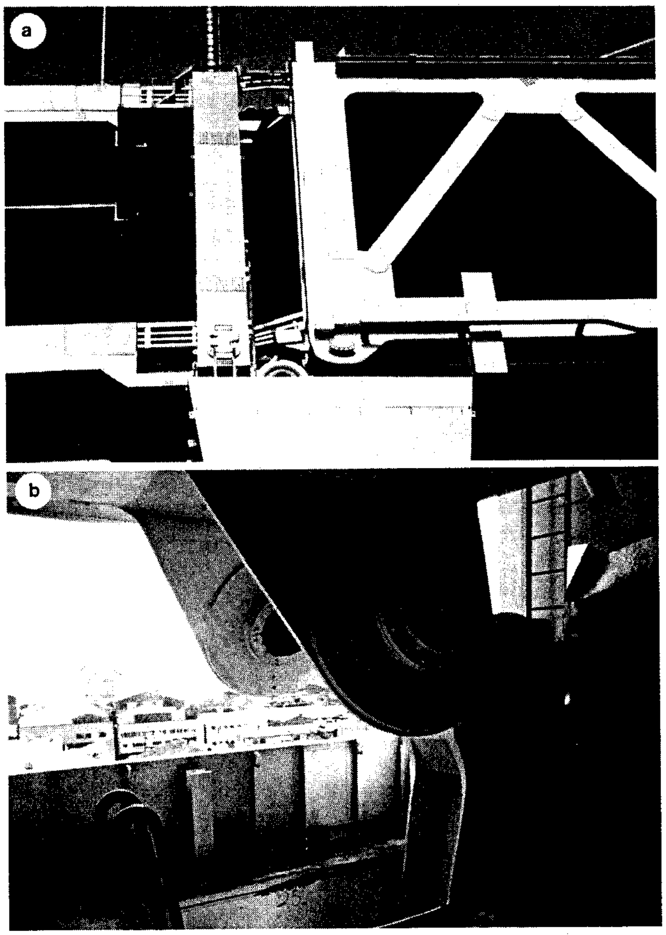
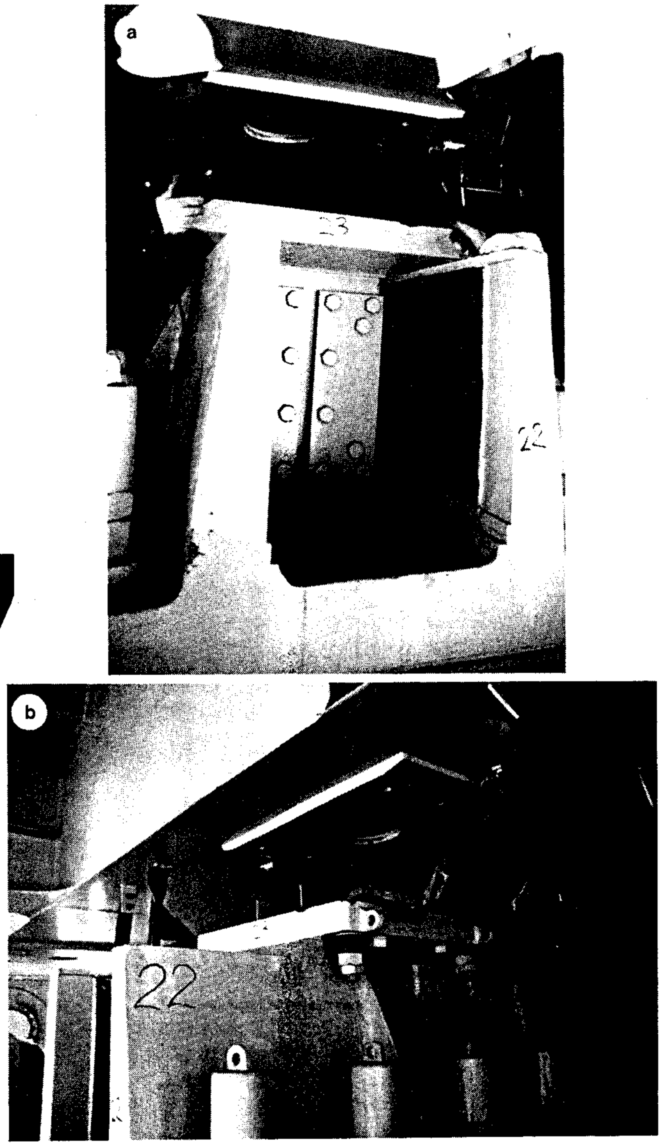


Fig. 32. Failure of the wind shoe connection on Higashi–Kobe Bridge (note that the top plate of the wind shoe was originally attached to the underside of the deck that is now approximately 0.5 m above the shoe plate): (*a*) front view; (*b*) side view.



In the aftermath of the Hyogo-ken Nanbu earthquake, having witnessed the extensive damage to steel bridges in the Kobe area, the above observations suggest that the current Canadian construction and design procedures need to be evaluated and improved in order to retrofit and build bridge structures that will exhibit a more desirable seismic performance. Indeed, without such changes, since earthquakes of the same magnitude as the Hyogo-ken Nanbu earthquake are anticipated in many regions of the country, and because the past and current Canadian earthquake-resistant design and construction practices have been generally less stringent than the Japanese ones, comparable or more severe structural damage would likely be experienced in Canada in case of severe ground shaking. Incidentally, a comprehensive revised set of seismic design provisions have recently been proposed for possible inclusion in the future edition of the *Canadian Highway Bridge Design Code* (CHBDC 1995). This comprehensive design procedure includes ductile detailing provisions, capacity check requirements, and specifications for preventing bearing failures. In the following, some of these seismic provisions are examined in light of the Kobe experi-

Fig. 33. Oil damper on Higashi–Kobe Bridge, detached from the underside of the deck.

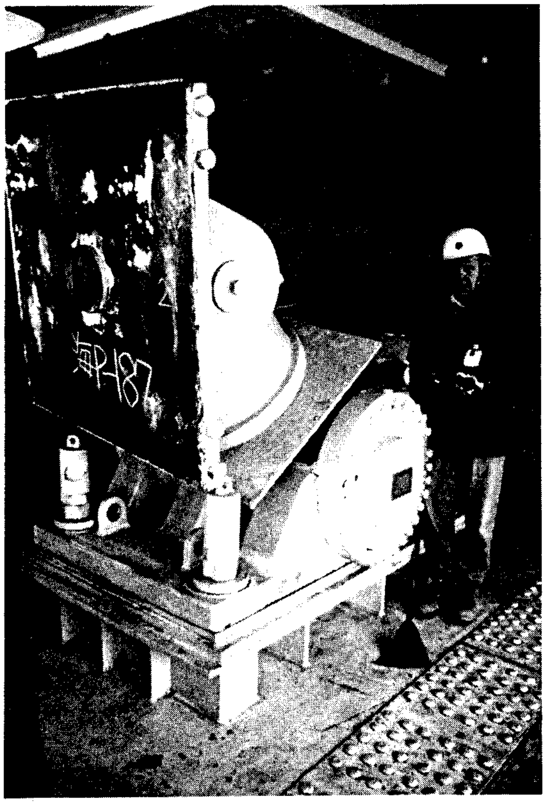
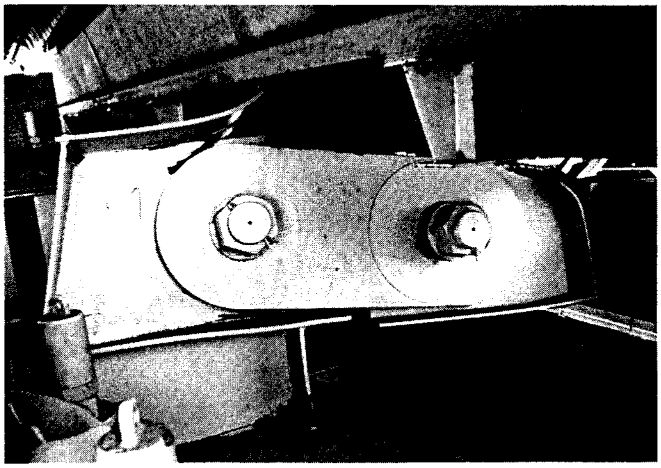


Fig. 34. Link connecting the approach span to the end pier on the Higashi–Kobe Bridge.



ence. However, the reader is cautioned that, at the time of writing, only a draft of those proposed seismic requirements was available (completed prior to the Hyogo-ken Nanbu earthquake), and that some minor modifications may result from the ongoing review process.

5.2. Relevance of the proposed CHBDC seismic design requirements

5.2.1. Modification factors

In the current bridge design codes used in Canada (OHBDC

Fig. 35. Overall view of the Rokkoliner Bridge (looking north towards Kobe).

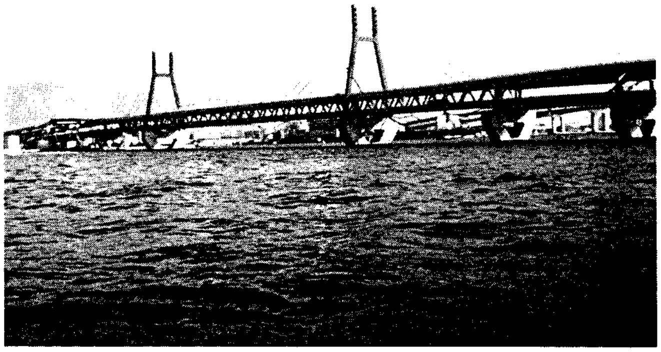
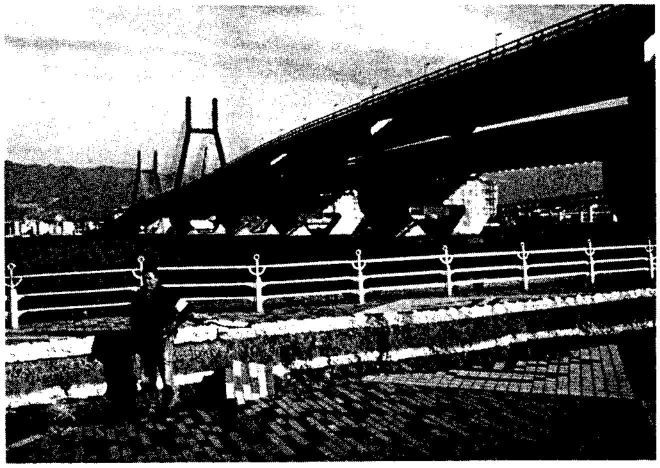


Fig. 36. Ground settlement behind quay wall on the Rokko Island side adjacent to Rokkoliner Bridge.



1991; CSA-S6 1988), design force levels are specified without any explicit reference to the fact that the structure has to respond in the inelastic range to sustain the design earthquake. The Kobe earthquake clearly demonstrated that bridge structures designed to specified code force levels must indeed rely on their ability to sustain loads in the inelastic range to survive.

In the proposed CHBDC provisions, the expected elastic forces are first determined in the calculations. They are then divided by a force modification factor R, conceptually similar to the one used in the *National Building Code of Canada* (NBCC 1995), to account for the capacity of structures to resist ground motions through inelastic response. Although such a modification may appear editorial, it provides the design engineer with a meaningful reference load level, and the approach clearly warns the designer that yielding will likely take place if he adopts a R factor larger than unity.

5.2.2. Bridge classification and design procedure

The tremendous impact of the closure of three major expressways in Kobe, together with the failure of main rail lines, dramatically demonstrated the need for critical facilities that

Fig. 37. Tilted pier and collapsed rail span on approach to bridge.

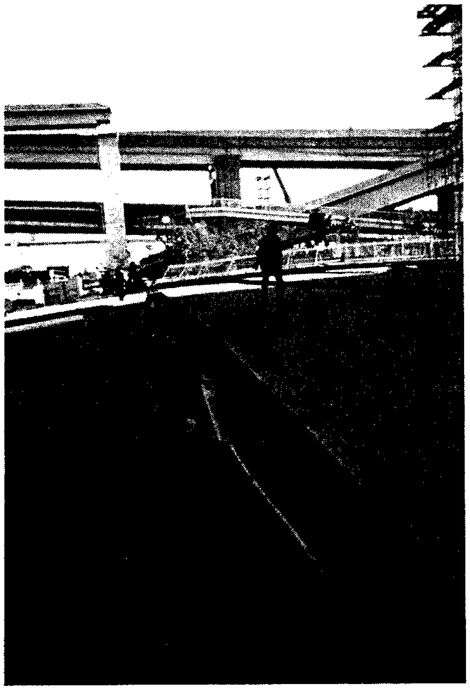


Fig. 38. Failed tension link on a collapsed rail span.

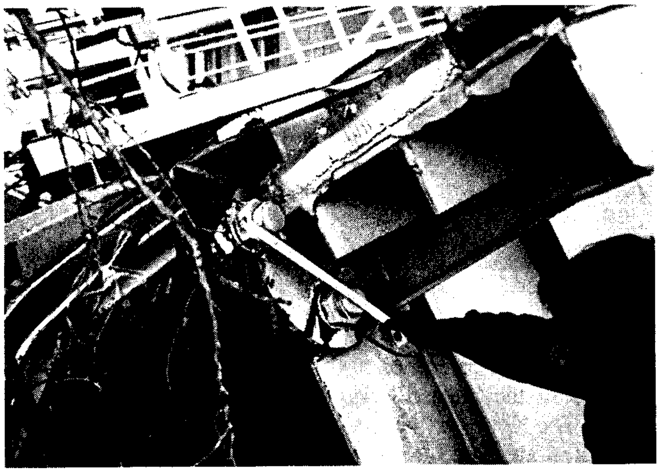


Fig. 39. The Nishinomiya Port Bridge: collapsed eastern approach span adjacent to arch pier.

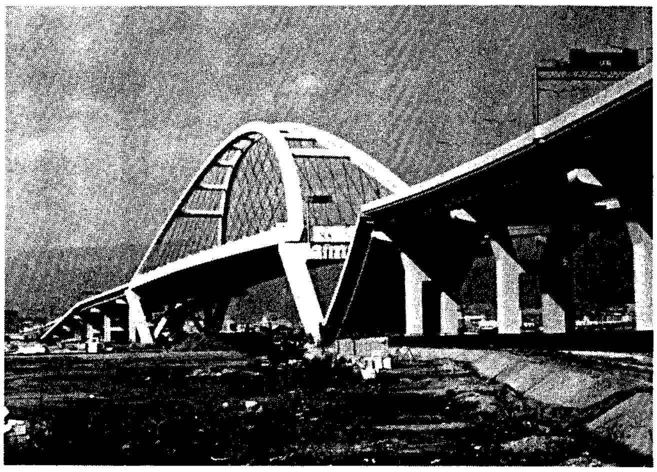
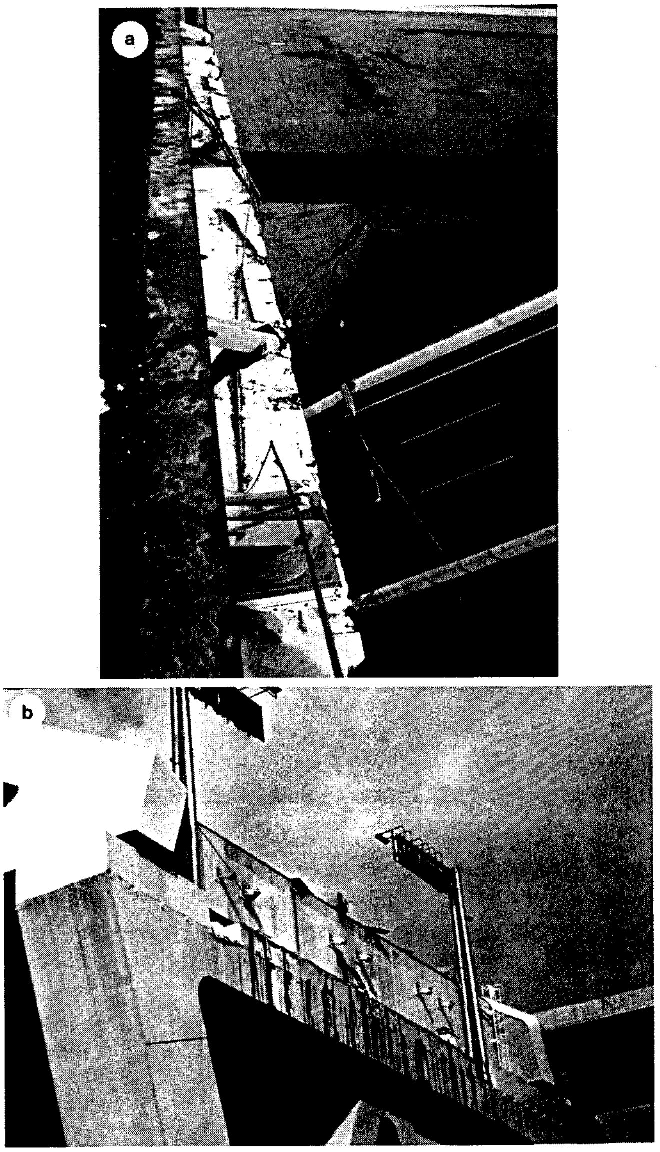


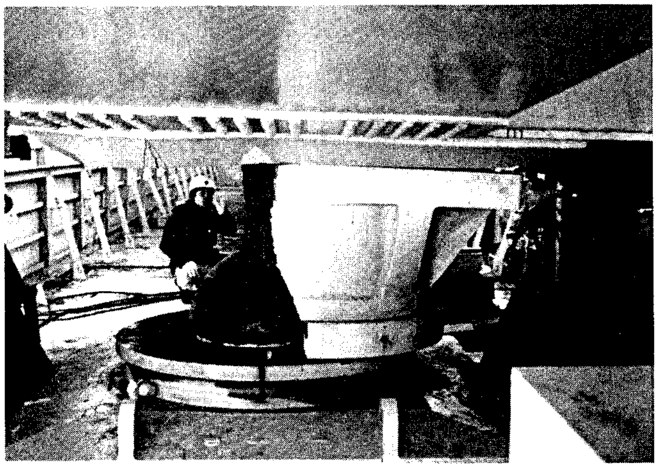
Fig. 40. Seating details on an eastern arch pier as seen from (*a*) bridge deck and (*b*) ground.



can be operational after an important seismic event. In addition, the extent of damage experienced by bridge structures in Kobe clearly indicates that the current factors for achieving this objective are totally inappropriate and must be revised.

In the proposed CHBDC, three importance categories are defined: critical bridges, essential bridges, and other bridges. The critical structures can be opened immediately to all traffic after the design earthquake (a return period of 475 years, corresponding to a 10% probability of exceedence in 50 years, or 15% in 75 years) and be usable by emergency vehicles after a very large earthquake (e.g., a 1000-year return period event). Essential facilities are those that must remain operational for emergency vehicles immediately after the design earthquake.

Fig. 41. Fracture of cast bearings supporting the west end of the arch (arch span is to the left (east)).



For critical bridges, it is suggested that the structure remains essentially elastic under the design earthquake. Therefore, these bridges would be assigned a *R* factor equal to 1.0 or 1.5, depending on the structural system used. For the essential bridges category, the *R* factor would vary between 1.0 and 3.5, depending upon the type of substructure and, in some cases, the type of foundation. For other bridges, the *R* factor lies between 1.0 and 5.0, depending on the ductility of the structural system.

In addition to specifying higher load levels, the importance of the bridge is also considered when selecting the structural analysis scheme to be used in the design. Four different analysis methods are defined: the uniform load method, the single mode method, the multi-mode method, and the elastic time-history analysis. Alternatively, a static push-over analysis can be performed in lieu of the elastic time-history analysis. The uniform load method essentially is the equivalent static load method used in current codes. In addition to the importance category of the bridge, the seismic risk at the site (four seismic performance zones are defined for the country) and the uniformity of the structure are also considered when determining which of the analysis methods has to be employed.

Current editions of the Canadian codes (CSA 1988; OHBDC 1991) recommended that more refined analysis be performed for nonuniform or irregular structures, but provided no quantitative guidelines to establish whether or not a structure could be considered as a regular one. In the draft of the CHBDC, the following criteria are proposed for this purpose: maximum number of spans, maximum curvature of the bridge, maximum span length ratio from span to span and maximum bent or pier ratio from span to span. The distinction between regular and irregular structures is obviously most important in seismic design, as severe damage was frequently observed in Kobe where nonuniformity existed in bridges, and the quantitative criteria introduced in the proposed CHBDC are helpful in this regard.

5.2.3. Inelastic response of steel piers

Many large hollow steel piers suffered severe local buckling during the Hyogo-ken Nanbu earthquake. As a result, some box sections completely lost their ability to support the applied gravity loads and collapsed in a spectacular way. Such performance could be predicted based on the results of Japanese research performed in the last few years. However, although progress has been made, reliable design or construction methods are still awaited to ensure a proper inelastic performance without axial strength degradation.

Fig. 42. Slack cable and cable anchorage.

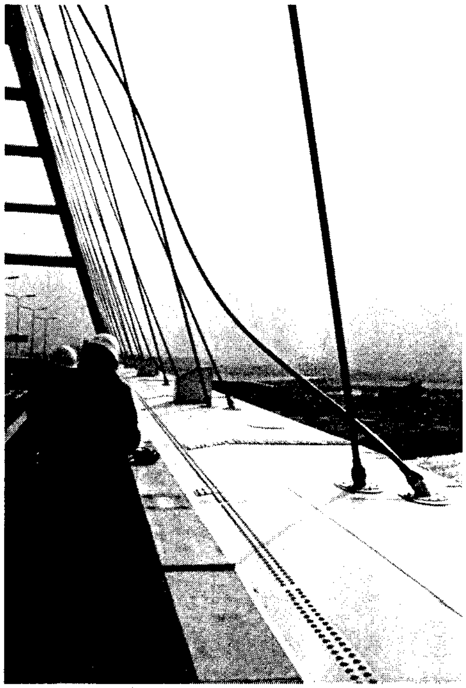
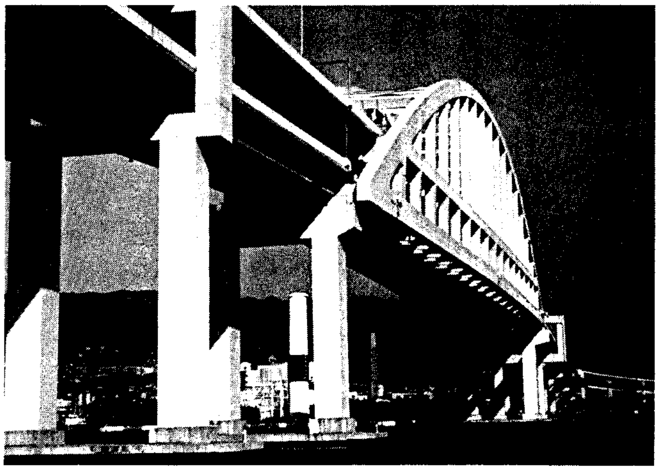
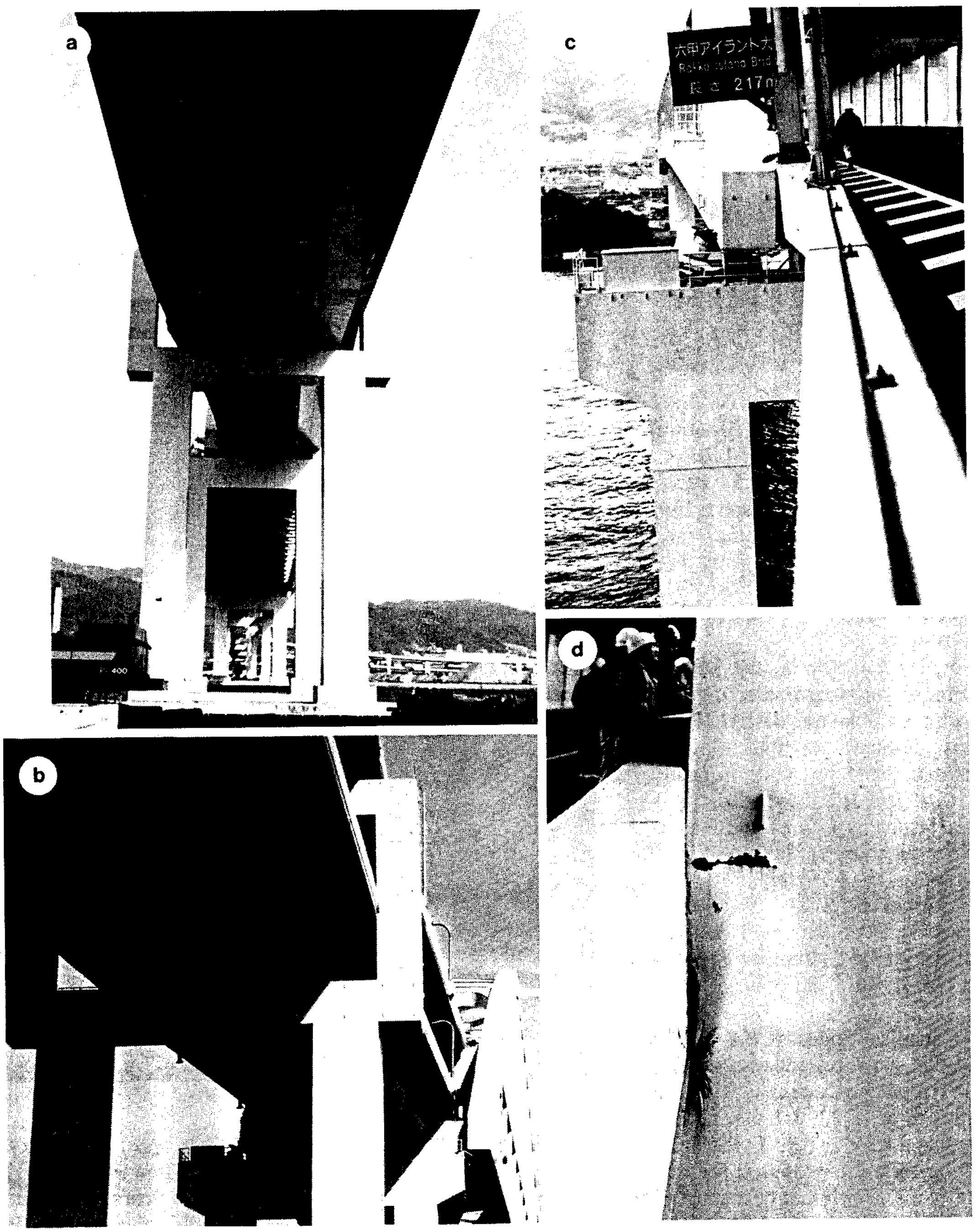


Fig. 43. Displaced arch span on Rokko Island Bridge, looking north (Rokko Island is south).



The proposed CHBDC does not include any provisions for the design of very large steel piers such as those used in Japan. It provides, however, limits for the width-to-thickness ratio (*b*/*t*) of rectangular tubular members and the diameter-to-thickness ratio (*D*/*t*) of circular ones expected to undergo cyclic inelasticity during earthquakes. The proposed *b*/*t* ratio of rectangular tubular sections subjected to such cyclic loading is based on tests performed using standard rolled shapes, and the *D*/*t* limit for circular tubular sections is identical to

Fig. 44. Displaced arch span and approach piers on Rokko Island Bridge, looking north: (*a*) underside view, showing panel buckling in the header cross-beam; (*b*) side view, showing the arch fallen from bearings, and the column impacted by displaced approach span; (*c*) closer view of the toppled bearing; (*d*) closer view of the column impacted by displaced approach span.

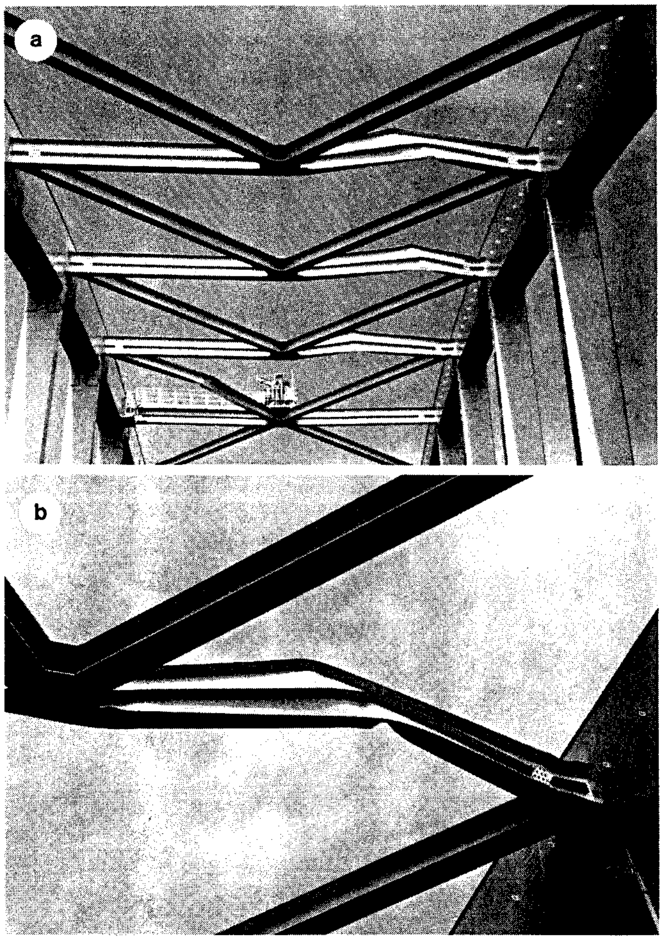


that in the AISC plastic design specifications. Wide-flange shapes that classify for plastic design (Class 1) are also suitable bridge column members which can sustain flexural yielding in a stable manner during an earthquake. Also, recognizing that limiting the stress due to axial loading in columns helps improve their inelastic seismic performance, the proposed CHBDC limits this stress to 60% of the yield stress.

5.2.4. *Fixed bearings*

Numerous fixed bearings failed during the Hyogo-ken Nanbu earthquake, either in the bridges' transversal or in the longitudinal directions. In many instances, failures can be attributed to excessive inelastic deformations (and even collapse) of nearby substructure elements, which pulled the superstructure away from its supports. In many other cases, however, the supporting elements showed only limited signs of

Fig. 45. Buckled horizontal wind-bracing between arches: (*a*) global view; (*b*) close-up of one such buckled horizontal wind-bracing member between arches.

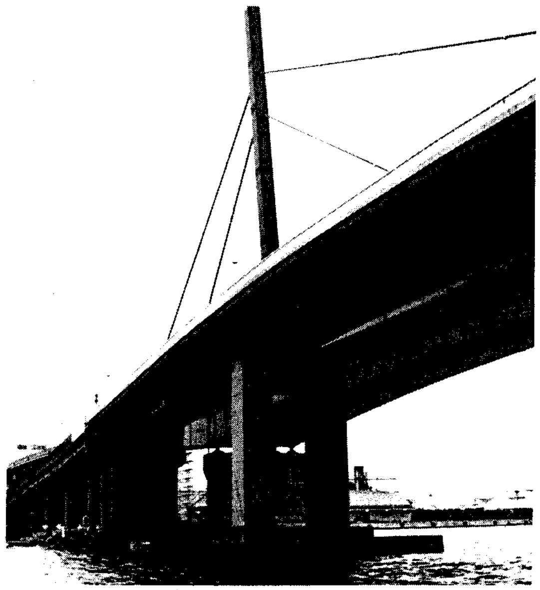


inelastic action in the vicinity of failed bearing units, which strongly suggests that the bearings' design loads were too low.

In the proposed CHBDC, the design loads for fixed bearings are obtained with a R factor equal to 0.80, which means that the loads considered for the design of the bearings are 25% higher than those that will produce an elastic response of the bridge superstructure. Such an amplification in the loads is intended to ensure the integrity of the structure. As a result, in Victoria, B.C., for example, the seismic coefficient for fixed bearings can be as high as 0.94 for short-period structures. The magnitude of this factor obviously depends on the period of the bridges and the soil conditions.

In Kobe, the design seismic coefficient that has been used in the last 30 years ranges between 0.10 and 0.35, depending on the code specifications, the period of the structure, the importance of the bridge, and the soil conditions. Hence, the CHBDC provisions should result in a better seismic performance of the bearings than that exhibited by the bridges during the Hyogo-ken Nanbu earthquake. Although the CHBDC procedure appears to be conservative, it will be interesting to see how Japanese bridge engineers will address this issue following their investigations of the damage from this earthquake.

Fig. 46. Damage to a medium-span cable-stayed bridge: leaning central steel column.



5.2.5. Seismic restrainers and seat length

For expansion bearings, the proposed CHBDC specifications explicitly require that restrainers be provided at the joint unless the support length is sufficient to accommodate the expected deflections. The latter is computed at the elastic force level (R factor equal to 1.0) which, according to the equal displacement principle, is believed to represent a realistic estimate of the inelastic deformations of a structure. The minimum length of the support is given by the following empirical equation:

$$[24] \quad N = K\left(200 + \frac{L}{600} + \frac{H}{150}\right)\left(1 + \frac{\psi^2}{8000}\right)$$

where L is the length of the bridge to the adjacent expansion joint (in mm), H is the average substructure height (in mm), Ψ is the skew angle in degrees, and K is a modification factor which varies between 0.5 and 1.5, depending upon the soil type and the seismic risk at the site. For most bridges, K equals 1.0, except in the highest seismic zone where a K of 1.5 must be used. This expression differs slightly from the Japanese equation (JRA 1990) presented earlier in that the proposed Canadian specifications recognize the influence of pier height and skew angle on the minimum needed support length.

For comparison, a typical straight simple span of the Hanshin Expressway structure (average span length of 30 m and pier height of 11 m) would have required a minimum bearing seat length of 485 mm according to the proposed Canadian specifications (assuming a K of 1.5), while the 1990 Japanese Specifications would only require a 350 mm long seat. Incidentally, most of the spans of the Hanshin Expressway had restrainers, as illustrated earlier, and most

Fig. 47. Elevated Harbour Monorail Line: (*a*) satisfactory seismic performance of steel columns (typical); (*b*) damaged concrete columns adjacent to a steel column (typical).

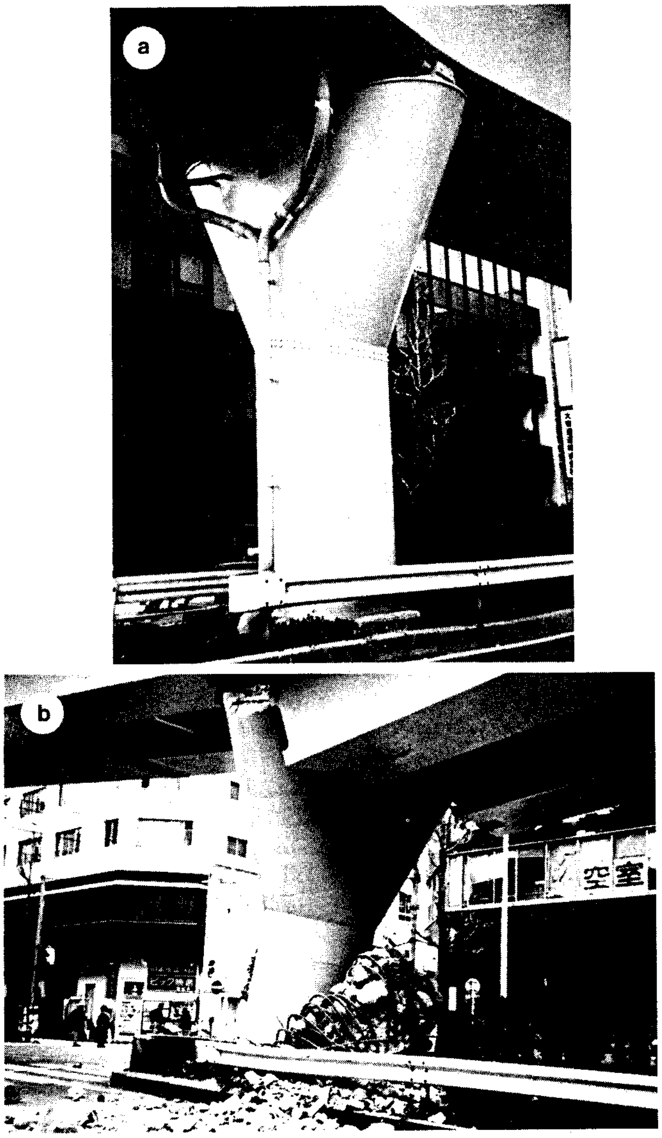
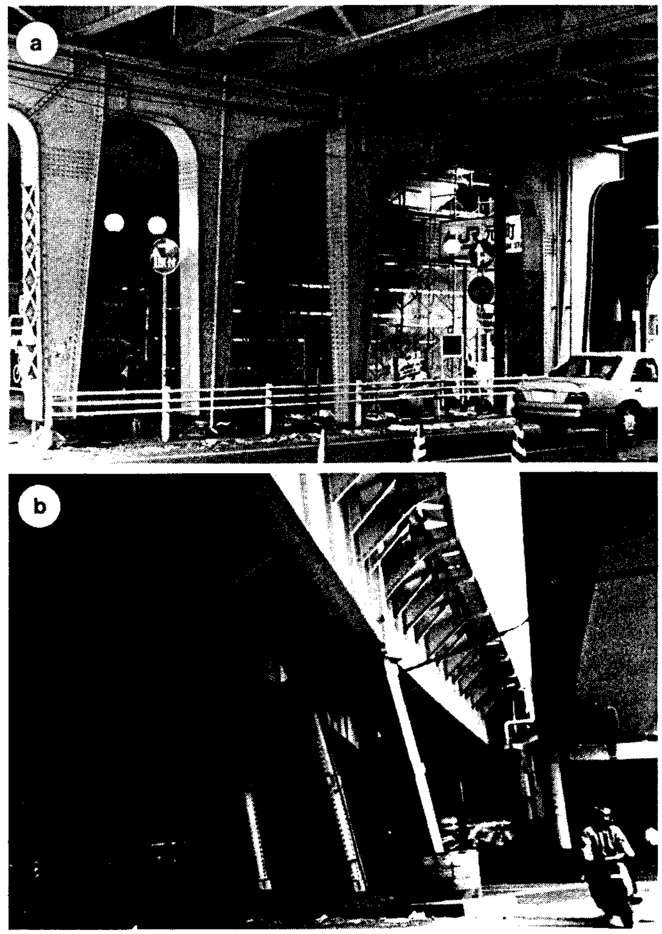


Fig. 48. (*a*) Undamaged steel frames bent with columns oriented to provide strong axis bending against lateral excitations; (*b*) steel columns pinned at both ends which provided no contribution to the lateral load resistance.



existing Japanese bridges have seat lengths considerably more generous than Canadian bridges of the same vintage. Nonetheless, span displacements of that magnitude have been observed at several locations in Kobe, some of them being the result of large yield deformations. It was also observed that in many cases of spans collapses, restrainers between adjacent spans were not connected to the underlying piers or columns. Typically, in most bridge codes, including the Japanese (JRA 1990) and the proposed CHBDC, connection of the girders to the piers and columns is strongly recommended, but not mandated. Hence, whenever adjacent spans over a pier are tied together, but not tied to the piers, it may be wise to consider designing the restrainers to be able to support the tributary weight of the unseated span.

Again, it will be interesting to see how the Japanese Specifications will be upgraded in light of the extensive restrainers failures, spans unseating, and other failures that occurred during the Hyogo-ken Nanbu earthquake. Likewise, while the proposed CHBDC provisions appear to introduce numerous improvements to the current Canadian bridge seismic-resistant design practice, by explicitly stating the expected desired level of seismic performance for a given structure at a given site, by recognizing the key parameters influencing the seismic behaviour of bridges, and by addressing those through a rational design procedure, it remains that some of the numerical values assigned to the different criteria and design parameters may require some future adjustments based on new data on the seismic performance of bridges.

5.3. Long-span bridges — current infrastructure and design practice

Canada has a significant stock of both older and newer long-span bridges. Although there are obviously fewer of these structures in the seismic regions of Canada compared to Japan, these bridges represent a large infrastructure investment, and constitute vital links in the Canadian transportation system. Vancouver can be considered as a case in point where there is a great reliance on several long-span bridges linking various parts of the city and the surrounding regions. Among those bridges are some older bridges, such as the

Lions Gate, Second Narrows, and Port Mann, as well as some fairly new bridges such as the Alex Fraser and Skytrain.

Design practice for the seismic engineering of long-span bridges in Canada tends to follow a case-by-case, non-code-formulated basis; an approach that is realistic only for large bridge projects (Taylor et al. 1985; Khalil and Bush 1987). Seismic design criteria are established using input from seismicity, ground motion, and geotechnical studies. Seismically effective structural design concepts are developed and detailed structural analyses are employed to verify the anticipated seismic performance. The analyses often make use of both response spectrum and time-history analyses. Similar detailed studies have been undertaken in recent years for the seismic evaluation and retrofit of some of the older long-span bridges in Vancouver. This approach is also similar to what has been followed in Japan for large bridge projects, such as for the Honshu–Shikoku links.

Some lessons learned from the seismic performance of long-span bridges in Kobe, and relevant to the design practice in Canada, can be summarized as follows:

- *Functionality*: Long-span bridges are critical links in urban transportation systems and should be designed to be fully functional after a major earthquake. Although damage to the long-span bridges in Kobe is repairable, the out-of-service time on most bridges is likely to be several months.
- *New designs*: Newer long-span bridges generally performed quite well, and there was no occurrence of major failures in the main span structures of any long-span bridges. However, it was also clearly demonstrated that even the newest designs had weaknesses, mainly in details, that resulted in the bridges being inoperational after the earthquake. It may be wise in new designs to identify the weakest links of seismic resistance, evaluate their threshold of damage, and assess whether their ultimate failure mode is acceptable.
- *Approach spans*: To ensure postearthquake functionality of a bridge, the seismic integrity of the approach spans is as important as the satisfactory performance of the main span.
- *Bearings*: Bearings are especially weak links that can be damaged by both longitudinal and transverse responses. This can cause expansion joint misalignments (in both transverse and vertical directions) that impair serviceability.
- *Connections details*: Large forces in connections, especially in superstructure-to-bearing and bearing-to-substructure connections, must be accommodated to maintain a continuous load path. Failure of bolted connections was a major initiating factor in the damage of several long-span Kobe bridges.
- *Restrainers*: Restrainers tying simple spans together are often weak links. They need to be designed for substantial seismic forces.
- *Soil movement*: Tilting of piers in soft soil due to liquefaction-induced lateral ground spreading or by movement of adjacent soil-supporting structures such as quay walls resulted in collapse of at least two bridge approach spans. A special attention must be paid to the foundation design of long-span bridges.
- *Span support*: Inadequate seat-widths were also a contributing factor to span collapses. Generous seat-widths should be provided.
- *Ground motion*: For major bridges we should anticipate the unexpected. In light of the Kobe experience (and also Northridge) it is reasonable to consider a scenario where the ground motion may be more severe than our best estimate of the maximum ground motion specified in the design criteria. In this situation one should ask what would be the response of the bridge. Would the weakest link still be strong enough to carry the larger seismic loads? Would the bridge still be serviceable?

5.4. Railway bridges

The design of steel railway bridges in North America is essentially based on the *Manual for Railway Engineering* published by the American Railway Engineering Association (AREA 1992). In this code document, and previous editions of it, there is no reference to seismic loading nor any design provisions to ensure proper behaviour under earthquake-induced ground motions. Recently, a new chapter entitled "Seismic design for railway structures" (AREA 1994) was drafted by the AREA, and has been undergoing review. However, this new chapter is largely concerned with providing postearthquake operation procedures restricting speed as a function of earthquake magnitude and distance to epicentre, and broad general informative design guidelines such as "utilize simple spans, keep the span lengths short, minimize weight, . . . use materials such as steel and timber that have performed well during past earthquakes, and ductile connections details that are easy to inspect and repair after earthquakes." Quantitative design criteria are not provided.

This situation is largely a result from the observation that railroad bridges have historically performed well in past North American seismic events. This satisfactory performance can be partly explained, once it is recognized that (*i*) train live-load models include an important lateral load component; (*ii*) most older railway bridges are unballasted light steel structures, and therefore have a low reactive mass; (*iii*) the continuous welded-rails used nowadays in track structures can provide an additional restraint against longitudinal and lateral seismic forces; (*iv*) a large number of existing railroad bridges are simply supported spans between abutments, a type of bridge already seismically advantaged (Dicleli and Bruneau 1995) compared to more complex structures; and (*v*) few of the railroad bridge types potentially more seismically vulnerable (e.g., ballasted bridges having concrete decks) have been exposed to severe earthquakes yet. However, observations in the Kobe area revealed that railroad bridge structures, even when designed with consideration to earthquake movements, could experience significant damage, such as unseating of girders at support, failure of bearings, and brittle fracture of substructures, that could lead to the closure of the lines. Also, research conducted at the University of Ottawa (unpublished yet) confirms the potential seismic vulnerability of some particular types of railroad bridges. This suggests that earthquake motions should be thoroughly accounted for in the design and analysis of railway bridges in Canada. Particularly, in view of the low (and reducing) redundancy of the Canadian railroad system, it can be argued that these structures should be considered essential bridges, and designed to a correspondingly high seismic load level.

6. Conclusion

The Hyogo-ken Nanbu earthquake struck an urban environment endowed with many modern steel bridges. The seismic performance of these steel bridges was generally good, particularly compared to concrete bridges of similar vintage. However, many older and some new steel bridges suffered damage, and numerous types of steel bridge failures were observed for the first time; these include severe to fatal buckling of steel columns, brittle column failures, and new types of bearing failure. Significant damage to large-span cable-stayed and arch bridges also occurred, providing the most extensive record of earthquake damage to long-span structures in the world. For long-span bridges, failures appeared to originate in the bearings systems, often leading to additional and more serious damage in the superstructure. Attention was also drawn to the vulnerability of approach spans, especially to conditions that would lead to collapse by loss-of-support at the seating. The observations of this reconnaissance visit provide valuable insights into the potential seismic-performance of comparable structures during future North American earthquakes.

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