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# Performance of steel bridges during the 1995 Hyogoken–Nanbu (Kobe, Japan) earthquake—a North American perspective

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A large number of steel bridges were damaged by the January 17, 1995, Hyogoken–Nanbu (Kobe, Japan) earthquake. This damage is particularly relevant to Eastern North America where considerably more steel bridges exist than in Western North America where bridges exposed to past earthquakes were mostly of reinforced concrete. Therefore, in light of the Kobe earthquake, a comparison of the steel design practice and design requirements in Japan and North America is instructive. In this paper, such a comparison is first presented, followed by a review of the observed damage to steel bridges and a review of the causes for this damage. Then, the relevance of these observations to North American bridge design practice is examined. 

**Keywords:** steel, bridges, steel columns, buckling, brittle fractures, bearing failures, seismic restrainers, design codes, Canadian practice, earthquake, seismic

## 1. Introduction

The January 17, 1995, Hyogoken–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 bridges in the area of severe shaking. As a result, all major roads and railways crossings in Kobe were closed due to 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. Damage was suffered by many steel piers, bearings, seismic restrainers, and superstructure components, and some spectacular collapses resulted from this damage[7,23]. 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. Moreover, this damage is particularly relevant to Eastern North America where considerably more steel bridges exist than in Western North America where bridges exposed to past earthquakes were mostly of reinforced concrete.

In this paper, a description of these past and current bridge design requirements is first presented, followed by a review of the observed damage to steel bridges. Then, the relevance of these observations to North American bridge design practice is examined. Only short and medium span bridges are considered within the scope of this paper.

## 2. Past and current design requirements

In Canada as well as in Japan, most short and medium span steel highway bridges have been built in the last 40 years, although construction of the road infrastructure network in major urban centres has started somewhat later in Japan 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/piers have also been used (more frequently so in Japan).

### *2.1. Evolution of design philosophy and force levels*

The first Japanese design document that prescribed seismic-resistant design requirements appeared in 1926, shortly following the devastating 1923 Magnitude 7.9 Kanto (Tokyo) earthquake that damaged 1785 bridges. It specified that earthquake lateral forces of 15–40% of a bridge self-weight (depending on location and ground condition) be considered during design, introducing the concept of 'seismic coefficient', *k*. Hence, from then on, although the magnitude of the seismic coefficient fluctuated somewhat over time (between 20% and 35% for the Kobe area), seismic-related lateral strength requirements for bridges have apparently existed for more than 70 years in Japan. Moreover, these earthquake design requirements also evolved as driven by observed damage in the numerous major earthquakes (M > 7) that occurred in Japan since. In several of these earthquakes, new types of damage emerged as the implementation of design standards successfully dealt with earlier problems. For example, requirements to assess the impact of earthquakes on soil-liquefaction and other bridge foundation problems as well as requirements to prevent span failures were introduced in the 1971 design specifications (the nearly 400 bridges that 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) earthquakes certainly provided some incentive for those changes). Extensive changes were also introduced in 1980 and 1990[19].

In Canada, bridge design codes have also traditionally adopted a simplified equivalent static load method to provide resistance to earthquake ground motions, but the level of seismic loads used in Canada has 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 Kobe, the horizontal seismic coefficient specified in 1966 lay between 0.02 and 0.06, depending on soil conditions and type of foundation. This value increased slowly over time, to between 0.06 and 0.08 in 1978, and 0.12 to 0.23 nowadays[13]; that is still considerably less than the values ranging between 0.20 and 0.35 that were used in the 1960s and 1970s in the Kobe area or those prescribed in the 1990 edition of the Japanese Specifications[21].

### *2.2. Evolution of steel detailing requirements*

In spite of the above history of comprehensive earthquake-resistant design requirements for bridges in Japan, prior to the 1995 Hyogoken–Nanbu earthquake, the Japanese bridge design specifications did not include any requirements to ensure ductile response of steel piers, even though ductile detailing was mandated for reinforced concrete piers; implicitly, reliance on the inherent ductility of the steel material was deemed to be likely sufficient. This partly explains why a number of steel piers behaved poorly during this earthquake. A comprehensive review of past and current design/detailing requirements for rectangular box-piers and hollow circular piers is presented in Bruneau *et al.*[7]. In summary, the Japanese specifications recognize that three types of buckling can typically occur in rectangular stiffened box sections (panel buckling, wall buckling, and stiffeners buckling), but does not preclude their development during earthquakes.

Likewise, in Canada, prior to 1998, no specific detailing requirements nor explicit capacity design rules were included in bridge design codes to ensure proper ductile behaviour of steel substructures. The first reference to ductile bridge response in Canada only appeared in the 1983 edition of the Ontario Highway Bridge Design Code[24], and was limited to concrete structural events.

### *2.3. 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. 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[19]. For bearing, the specifications explicitly but broadly require that 'fixed bearing portion shall be safe against inertia force of a superstructure'. An additional clause requires 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, as a net uplift design force, taken alone without the simultaneous consideration of gravity loads or horizontal seismic forces.

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 Canadian specifications corresponded to that used for the entire bridge. After 1988, 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).

### *2.4. 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 and the bearing seat length. For example, the distance to the nearest edge below the bearing shall be at least (in cm) $20 + 0.5\ L$ for spans less than 100 m, $30 + 0.4\ L$ otherwise, where $L$ is in metres. Seismic restrainers of various designs (and quite different that those used in North America) have been used in Japan to either connect a girder to a substructure, two girders together, or buttress against excessive displacements. All these devices to prevent span collapses are designed for twice the horizontal seismic coefficient considered in the design of the bridge. Furthermore, stoppers are generally provided at roller-bearings to limit the relative movement of the upper and lower portion of those bearings, thus reducing the risk that rollers will be dislodged from the bearing assembly.

In Canada, some provisions to accommodate expected movements at expansion joints were included for the first time in 1983 in Ontario, and 1988 for the rest of the country (by magnifying the calculated deformations produced by the prescribed seismic loads), but the first empirical expression for minimum seat width was only introduced in the 1991 edition of the OHBDC. Also, until 1998, no Canadian bridge design code had provisions for seismic restrainers.

*2.5. Seismic design of railroad bridges*

The earthquake-resistant design standard for railway structures[20] has generally not included special provisions for the ductile detailing of steel structures. Only strength requirement are addressed by the consideration of an horizontal seismic coefficient equal to 16–25% of the structure's dead load, depending on the soil conditions and period of the structure. A few clauses discuss the consideration of simultaneously applied live, load and earthquake forces and prevention of resonance with the natural 'rolling' period of trains. While numerous railroad steel bridges suffered damage in past Japanese earthquakes, that damage prior to the Kobe 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.

The design of steel railway bridges in North America is essentially based on the Manual for Railway Engineering[3]. 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, the AREA has proposed a new chapter concerned with providing post-earthquake operation procedures restricting speed as a function of earthquake magnitude and distance to epicentre, and broad general informative design guidelines. 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 earthquakes. Partial explanations for this past performance are presented in Bruneau *et al.*[7]. However, observations in the Kobe area revealed that railroad bridge structures, even when designed with consideration to earthquake movements, could experience significant damage that could lead to the closure of the lines: unseating of girders at support, failure of bearings, brittle fracture of substructures, etc. Clearly, earthquake motions should be thoroughly accounted for in the design and analysis of railway bridges in Canada, particularly in view of the low redundancy of the Canadian railroad system.

## 3. Damage to steel bridges

The damage suffered by short and medium span bridges can be grouped according to the following categories:

*3.1. Reinforced concrete substructure failures*

Prior to the Hyogoken–Nanbu earthquake, many engineers alleged that steel bridges were immune from seismic damage by virtue of their lighter superstructure mass compared to concrete bridges, even if supported by non-ductile substructure elements. This optimistic attitude was shattered as numerous concrete piers supporting steel superstructures failed all over Kobe during the 1995 earthquake. One such example is shown in *Figure 1*. Failure modes germane to reinforced concrete piers and observed during this earthquake include:

- Shear failures at the base of piers (*Figure 2*) due to inadequate shear reinforcement;

*Figure 1* Example of non-ductile reinforced concrete substructure supporting a steel superstructure: (a) span collapse; (b) close-up of fatal damage to non-ductile pier

- Shear failures in the middle of piers, often initiating at the termination point of longitudinal reinforcement, and due to inadequate shear reinforcement (*Figure 3*);
- Flexural failures due to inadequate confinement (*Figure 4*);
- Failure of column longitudinal reinforcement through the butt weld of welded splices;
- Failure and sliding shear at construction shears (*Figure 5*);
- Shear splitting failure at the top of narrow piers.

In the author's opinion, reinforced concrete piers having non-ductile details are seismically deficient irrespective of the type of superstructure supported. Further information on the seismic behaviour of reinforced concrete piers during earthquakes can be found in Priestley *et al.*[25], Anderson *et al.*[4] and Mitchell *et al.*[22].

*3.2. Column failures—local buckling*

A number of steel box-columns supporting portions of elevated expressways buckled, some rather severely, and collapse occurred at two locations as a result of steel column failures. The steel box-columns that failed appeared to have been squashed vertically, almost as if the plates on each of their four sides were 'peeled-off', as shown in *Figure 6*. Although little information can be obtained from those completely destroyed columns, damage to a non-collapsed steel box-column on an adjacent span (*Figure 7*) provided a clue as to the triggering event: failure of a weld-seam at the bottom corner of the box-column. Such performance

*Figure 2* Shear failure at bottom of reinforced concrete pier supporting a steel superstructure

could be predicted based on the results of Japanese research performed in recent years[7].

Mild to severe local buckling of round and square built-up hollow steel columns supporting the expressway of Kobe was particularly extensive, and sometimes very severe, with rupture of the buckled steel sometimes taking place due to excessive inelastic deformations (*Figure 8*). Many of these columns supported double-deck highway structures. Local buckling sometimes occurred at the base of circular columns, reminiscent of the so-called 'elephant-foot' buckling often observed in large cylindrical tanks following earthquakes. Buckling was also observed at the third point along the height of circular and square columns. Such above-base column damage typically occurred at or near a structural discontinuity, such as: (i) at the location of a door-hatch (*Figure 9*); (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 (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).

### *3.3. Column failures—brittle fractures*

Brittle fractures were sporadically discovered in columns which otherwise showed no signs of local buckling (*Figure 10*). Also, in at least one instance, brittle failure of the columns of a railroad-supporting steel portal-frame was also observed (*Figure 11*). These columns were apparently of cast-steel, formed using a centrifugal procedure developed in Japan. Whether or not column failure was the triggering failure-event, the sight of a brittle steel failure without evidence of prior yielding is disconcerting.

### *3.4. Seismic restrainers*

Many restrainers were observed to have worked effectively during this earthquake, preventing simply-supported spans from falling from their supports (*Figure 12*). Numerous seismic restrainers showed signs of plastic yielding and/or buckling. Others were strained to their limit, often due to excessive sub-structure displacements, and failed (*Figure 13*). 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 (*Figure 13*). 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. Variations on the same principle were also used (*Figure 12*).

In absence of effective seismic restrainers, many simply supported spans collapsed as their support length was insufficient to accommodate the large seismically induced longitudinal displacements (*Figure 14*).

### *3.5. Bearing failures*

Bearings suffered a considerable amount of damage during this earthquake (*Figure 15*). They frequently were the second structural element to fail following major sub-structure damage, but in some cases, they were observed to have failed in spite of the substructure remaining intact (*Figure 16*). Roller supports proved particularly vulnerable as their design provided limited resistance against seismic forces applied laterally. Fixed supports at end-spans also frequently suffered damage. In many of those instances, 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.

Failure of the bearing anchorage sometimes occurred in the concrete, 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 sometimes observed.

### *3.6. Bridge girder failures*

The lateral displacement observed for bridge spans which fell-off their bearings was often impressively large, sometimes producing localized severe lateral-bending of the steel girders and even rupture of the end-diaphragms (*Figure 17*). Tensile fracture of the bolts connecting end-diaphragms to the main girders, and fracture through the diaphragm extension haunch near the tip of the haunch, was typical in such cases (*Figure 18*).

### *3.7. Examples of satisfactory performance*

Braced substructure steel frames were generally constructed of large stocky braces, and none were found to have suffered damage by the author. Unbraced rigid substructure 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. By


*Figure 3* Shear failure at middle of reinforced concrete pier supporting a steel superstructure: (a) global view; (b) close-up showing large spacing of transverse reinforcement

contrast, columns pinned at both ends evidently provided no contribution to the lateral load resistance, which sometimes proved critical for narrow railroad-type bridges.

## 4. Relevance to Canadian practice

In the aftermath of the Hyogoken–Nanbu earthquake and the extensive damage it imparted to steel bridges in the Kobe area, recognizing that earthquakes of similar magnitude are anticipated in many regions of Canada and that past Canadian earthquake-resistant design and construction practices have been generally less stringent than the Japanese ones, it is expected that comparable or more severe structural damage would be experienced by existing bridges in Canada in a future earthquake generating severe ground shaking. Fortunately, new bridges to be built in Canada will benefit from a recently developed comprehensive set of seismic design provisions included in the new Canadian Highway Bridge Design Code[12]. While these new provisions were mostly drafted prior to 1995, the Hyogoken–Nanbu earthquake experience will help Canadian practising engineers appreciate the importance of the new ductile detailing provisions, capacity design requirements, and specifications for preventing bearing failures in all types of bridges.

The new seismic provisions of the CHBDC are divided into 12 sections, namely: Scope, Definitions, Notation, Earthquake Effects, Analysis, Foundations, Concrete Structures, Steel Structures, Joints and Bearings, Seismic Base Isolation, Seismic Evaluation, Seismic Retrofit. In this paper, the emphasis is placed on those provisions that differ most from the existing AASHTO seismic provisions (and from a recent draft for the next edition of that AASHTO document) and that are most relevant to steel bridges.

### *4.1. Earthquake design philosophy—capacity design, performance objectives, importance categories*

The CHBDC promotes capacity-design, using a terminology that clearly spells out the expected seismic performance. It refers to what is termed a 'ductile substructure element', defined as any 'element of the substructure expected to undergo reversed-cyclic inelastic deformations without significant loss of strength, and detailed to develop the appropriate level of ductility while remaining stable', and 'capacity protected element', defined as any 'substructure or superstructure element which has a force demand limited by the capacity of the ductile substructure element'. This capacity design approach is intended to ensure an adequate margin of strength between non-ductile failure modes and the designated ductile mode of deformation, as described below. Clearly, the ductile substructure elements are expected to be the only structural elements providing ductile energy dissipation in a bridge.

The CHBDC also describes the expected seismic performance requirements for bridges as a function of their importance. The performance objectives are nearly identical to those formulated in AASHTO, but different names are used for the 'importance categories'. The three importance categories considered are: lifetime bridges, emergency-route bridges and other bridges. The lifetime structures must be open immediately to all traffic after the design earthquake (return period of 475 years) and be

*Figure 4* Two examples of flexural failure due to lack of confinement at bottom of reinforced concrete piers supporting a steel superstructure

*Figure 5* Sliding failure along construction joint of reinforced concrete pier supporting a steel superstructure (first pier on left of photo)

usable by emergency vehicles after a very large earthquake (e.g. 1000 years return period event). Emergency-route facilities must remain operational for emergency vehicles immediately after the design earthquake. *Table 1*, taken from the CHBDC, clearly summarizes these performance objectives.

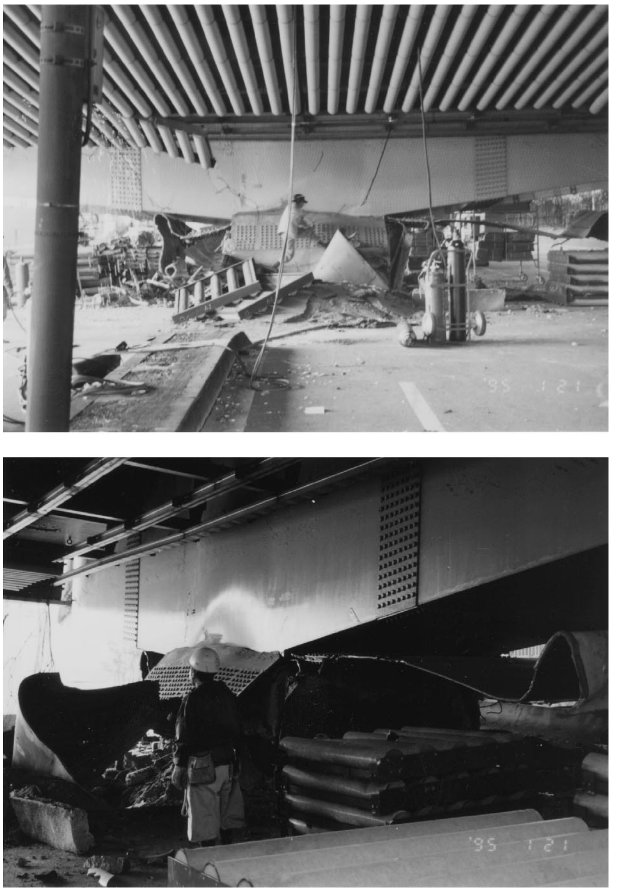

*Figure 6* Example of collapse as a result of steel pier failure: (a) failed pier; (b) close-up view of split steel plates

Note that these three importance categories correspond to the critical, essential, and other bridges classes in the AASHTO seismic design requirements. It is noteworthy the AASHTO specifications include a number of clauses to ensure that design requirements are more stringent for critical bridges than for essential or other bridges. In particular, a number of design clauses that are normally not applicable to essential and other bridges located in zones of low seismicity, cannot be waived for critical bridges exposed to the same seismic risk. This same philosophy was adopted in the CHBDC and implemented by the use of *Table 2*. The CHBDC approach ensures that the expression of that intent is not buried in sub-clauses throughout the specifications.

As such, a lifeline bridge in a location exposed to a peak ground acceleration having 10% chances of being exceeded in 50 years of 0.06g, would be designed to satisfy all the requirements applicable for bridges in Seismic Performance zone 2, and a Zonal Acceleration Ratio of 0.05 would be used in calculation of the elastic seismic response coefficient (described below).

### *4.2. Design forces and ductile substructure elements*

Earthquake loads to consider in design are determined in a manner similar to the AASHTO procedure, with some differences. *Table 3* summarizes the prescribed minimum analysis requirements. *Table 4* specifies the requirements for regular and irregular bridges. The uniform load method, conservative for the regular bridges for which its use is specified, gives an equivalent uniformly distributed static seismic loading, $P_e$, equal to:

*Figure 7* Condition of steel pier at other end of collapsed span: (a) global view of pier; (b) close-up view of failure of a weld-seam at bottom corner of box-column

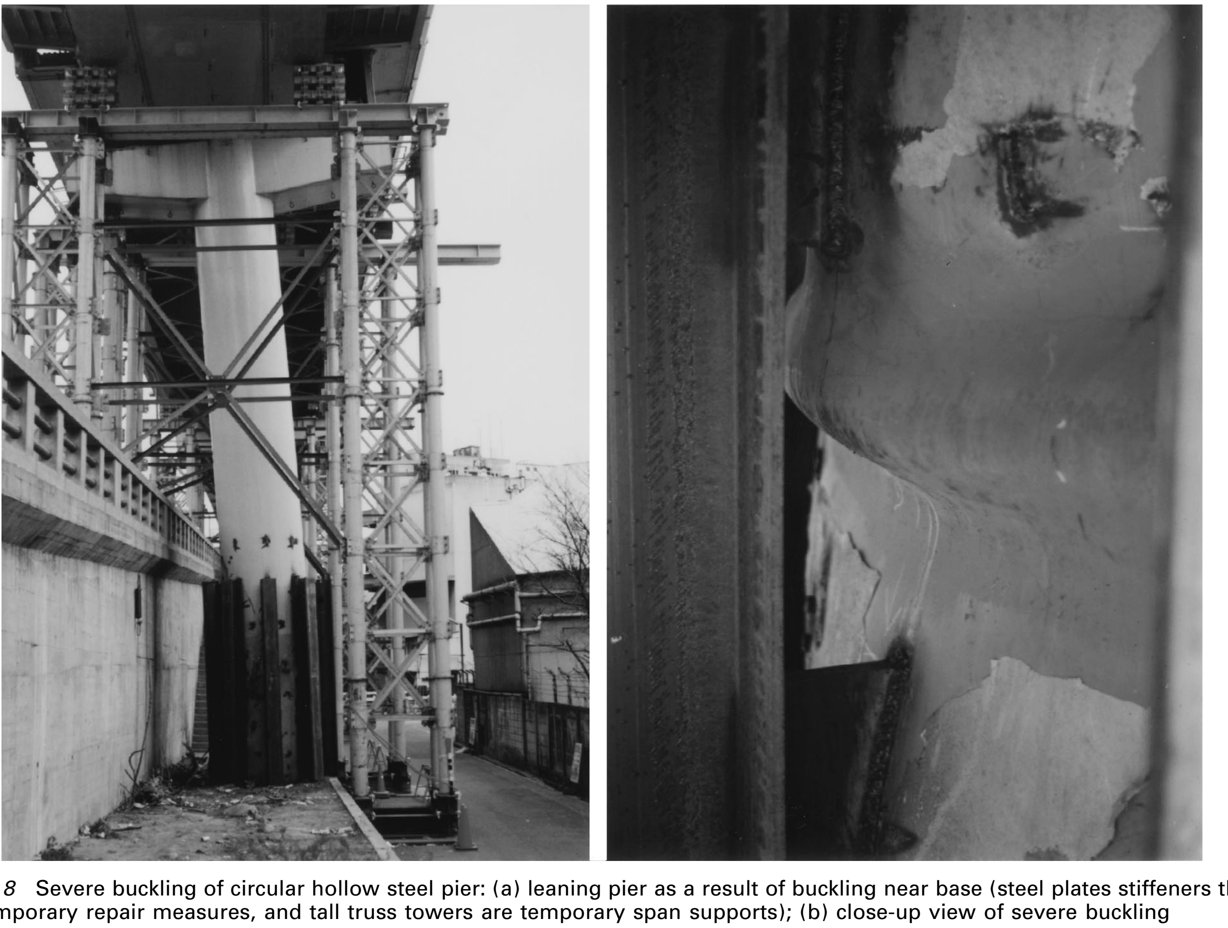

*Figure 8* Severe buckling of circular hollow steel pier: (a) leaning pier as a result of buckling near base (steel plates stiffeners there are temporary repair measures, and tall truss towers are temporary span supports); (b) close-up view of severe buckling


*Figure 9* Examples of buckling in rectangular hollow steel piers: (a) mild buckling at pier base; (b) severe buckling at pier base; (c) buckling at the discontinuity created by door-hatches in steel pier

$$P_e = \frac{C_{sm}W}{L} \qquad (1)$$

where $L$ is the total length of the bridge and $W$ is the 'Effective Weight' of the bridge. This later term is defined by the CHBDC as the 'total unfactored dead load of the superstructure and the portion of the substructure elements that contribute to the inertia mass'. In that case, for calculation of $C_{sm}$, the period of vibration of the bridge is specified as:

$$T = 2\pi\sqrt{\frac{W}{gK}} \qquad (2)$$

where $K$ is the lateral stiffness of the bridge, equal to $p_oL/V_{s,max}$, where $V_{s,max}$ is the maximum static displacement of the bridge due to an arbitrary uniform lateral load, $p_o$, and $g$ is the acceleration due to gravity (m/s$^2$).

The single-mode and multi-mode spectral methods, as

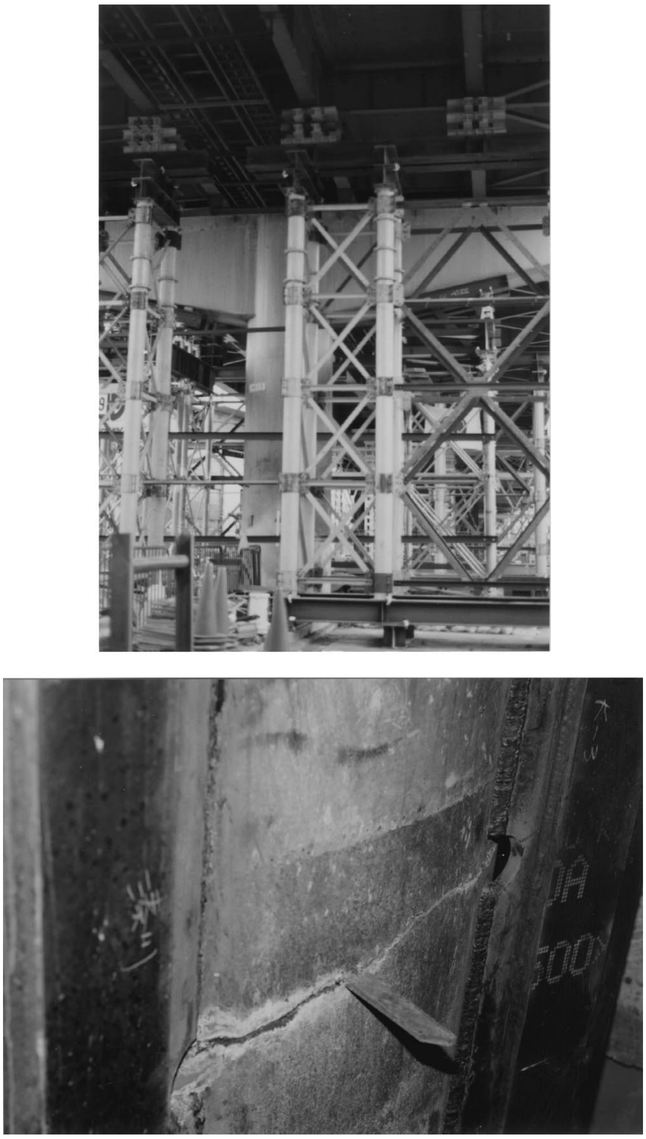

*Figure 10* Brittle fracture in steel pier: (a) global view of pier (tall truss-towers) are emergency temporary span supports); (b) crack in steel plate, wide enough to permit insertion of a small steel plate (large steel plate stiffners are temporary repairs)

well as the time history method, are identical to what is specified in AASHTO[1].

The resulting efforts acting in the ductile substructure elements are then divided by a force modification factor, *R*, conceptually similar to that used in most building codes worldwide. Different *R*-values have been assigned to different ductile substructure elements as a function of their ductile capacity. However, the CHBDC slightly differs from the AASHTO in its expression of the seismic response coefficient and of the force modification factors. In the CHBDC, the basic elastic seismic response coefficient, $C_{sm}$, for the *m*th mode of vibration is (omitting here for brevity the three exceptions that are similar for both codes for very soft soils or for bridges having very long period):

$$C_{\mathrm{sm}} = \frac{1.2AIS}{T_m^{2/3}} \leqslant 2.5AI \tag{3}$$

where $T_m$ is the period of vibration of the *m*th mode (s), $A$ is the Zonal Acceleration Ratio determined as described below, $S$ is a Site Coefficient for soil effects, and $I$ is an

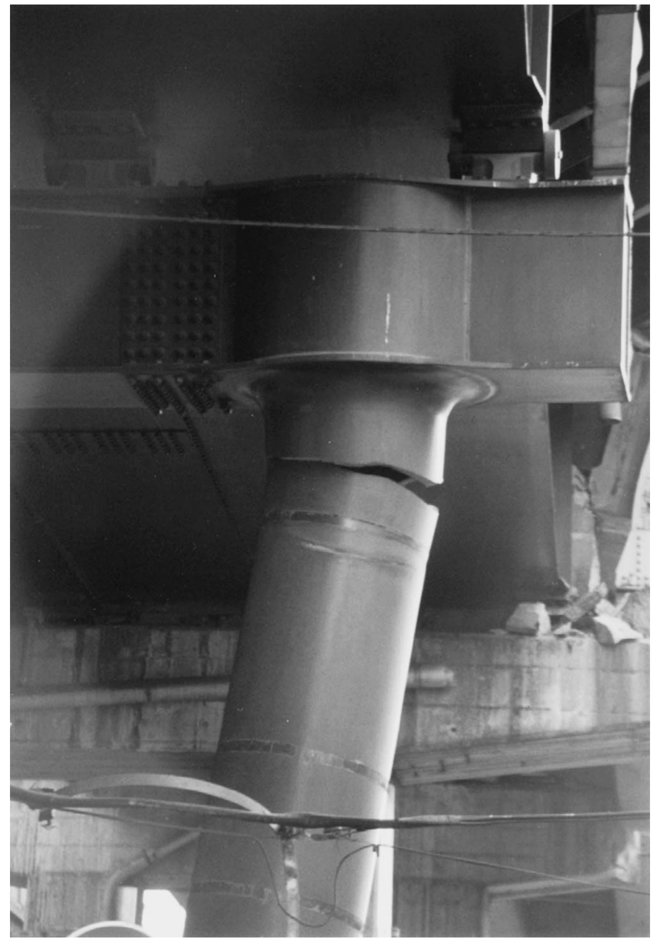

*Figure 11* Brittle failure, in a portal frame, of a steel column apparently formed using a centrifugal procedure developed in Japan

importance factor that depends on the importance category defined above. Hence, the CHBDC departs from the AASHTO by the introduction of this importance factor, taken as 3.0 for lifeline bridges, 1.5 for emergency-route bridges and 1.0 for other bridges.

Various types of ductile substructure elements are assigned a value of the force modification factor, *R*. These *R*-factors do not vary as a function of the bridge importance. The values of *R*, specified by the CHBDC for the special type of ductile substructure elements it recognizes, are summarized in *Table 5*.

These response modification factors reflect the capacity of ductile substructure elements to dissipate the seismic energy through inelastic load-deformation behaviour provided by special detailing provisions. These values also make allowance for redundancy in a bridge structure. This can be demonstrated by comparing the values for multi-column bents, assigned an *R*-factor of 5, with single column bents which are assigned an *R*-factor of 3 because they are not redundant.

The CHBDC also states that: ‘For bridges of slab, beam-girder or box girder construction and with a structurally continuous reinforced concrete deck from pier to pier (or abutment to abutment), a detailed analysis of earthquake effects on superstructure components is not required. However, analysis of end-diaphragms between girders at the abutments and piers is required’. This ensures that end-diaphragms are adequately designed to transfer the seismic forces from the deck to the substructure elements.

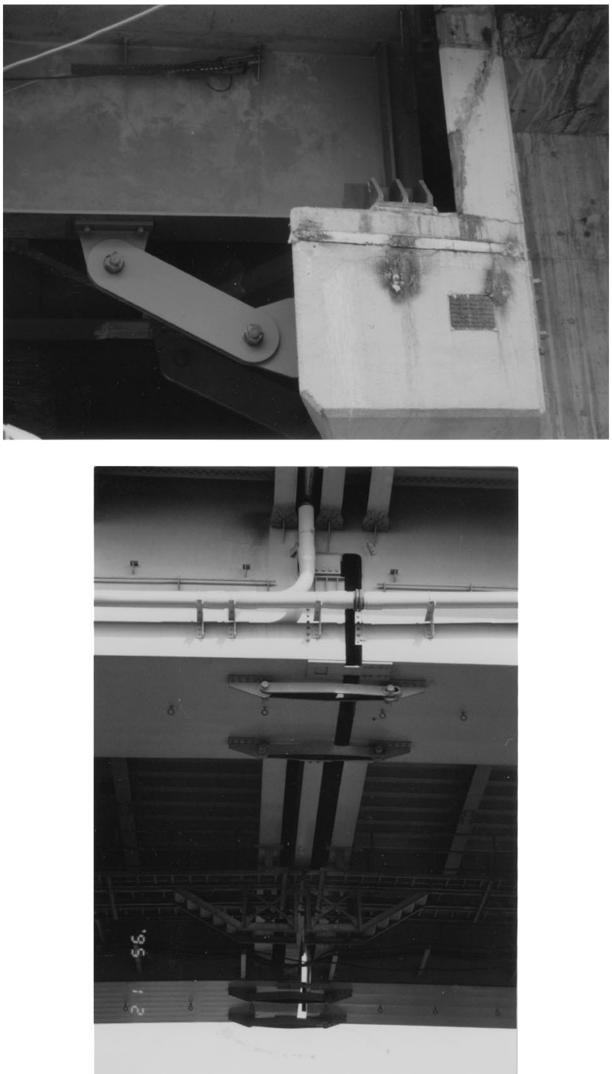

*Figure 12* Examples of satisfactory seismic restrainer behaviour

*Figure 13* Example of unsatisfactory seismic restrainer behaviour

Note that importance factors of 1.5 and 3.0, for emergency-route bridges and lifeline bridges respectively, are not intended to provide design spectra whose ordinates would be related to a probabilistic treatment of seismic demand. Rather, for these bridges, they provide an incentive for the use of ductile detailing, even though essentially elastic per-

*Figure 14* Example of collapsed span as a result of insufficient support length: (a) global view; (b) close-up view of top pier

formance is sought by the performance criterion. This ensures that the expected seismic performance will be met if earthquakes larger than expected strike at the site.

Once importance factors are combined together with the response modification factors for the most ductile type of systems, the resulting design forces to consider are similar to those implied in the AASHTO LRFD Specifications. The CHBDC approach, however, ensures that only ductile detailing will be used in emergency-route bridges and lifeline bridges.

Note that importance of the bridge is also considered when selecting one of the structural analysis methods permitted for design: uniform load method, single mode method, multi-mode method and time-history analysis. Likewise, more refined analysis must be performed for non uniform or irregular structures and a quantitative criteria is provided to establish whether or not a structure can be considered as irregular.

### *4.3. Capacity protected structural elements*

As for the seismic design forces for the capacity protected elements that must remain elastic, such as superstructures, cap-beams, beam column joints and foundations, they can be determined by using directly the elastic design forces

*Figure 15* Example of damaged bearing and collapsed span: (a) global view, looking at the West direction; (b) close-up view of bearing damage, looking at the North-East direction (almost reversed view)

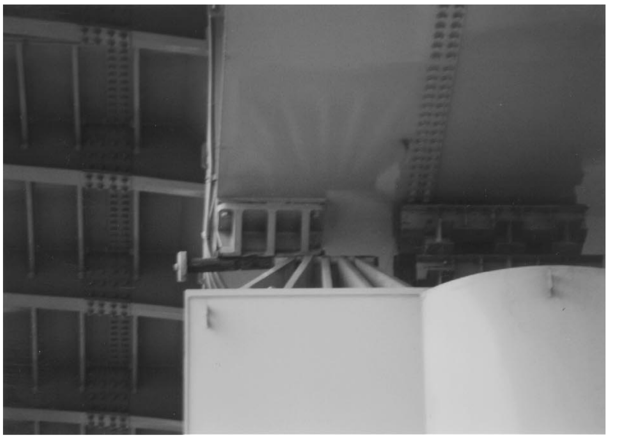

*Figure 16* Examples of severe bearing damage: (a) near-collapse of span due to bearing keeper-plate failure; (b) near-collapse of span due to sliding in transverse direction following bearing failure

(i.e. $R = 1.0$). Alternatively, as it may frequently be more economical in the most severe seismic zones (i.e. zones 3 and 4), and because it is an approach consistent with the capacity design philosophy, capacity protected elements may be designed to have a favoured resistance equal to or greater than the maximum force effects that can be developed by the ductile substructure element(s) attaining their probable resistance.

Note that probable resistance is defined by the CHBDC as being 'the resistance of a member, connection or structure based on the nominal dimensions and details of the final section(s) chosen, calculated accounting for the expected development of large strains and associated stresses larger than the minimum specified yield values'.

For comparison, in zone of moderate seismic risk (i.e. zone 2), instead of the probable resistance, only the nominal resistance of the ductile substructure element need be considered, it being defined as the 'resistance of a member, connection or structure based on the specified material properties and the nominal dimensions and details of the final section(s) chosen, calculated with all material resistance factors taken as 1.0'.

Finally, following a similar philosophy, but including a 25% additional safety factor in recognition of the key role these structural elements play in ensuring structural integrity, the CHBDC requires that connectors be designed in their restrained directions to transmit the maximum force effects determined by 1.25 times the elastic seismic forces (obtained with $R = 1$). Again, for the same reasons as above, these connectors need not exceed the force that can be developed by the ductile substructure element attaining 1.25 times its probable resistance.

This is particularly important for steel bridges which tend to be supported on more seismically vulnerable types of bearings. Examples of the potentially dramatic consequence that can ensue from bearing failure is shown in *Figures 15* and *16*.

### 4.4. Support length requirements for displacements versus seismic restrainers

To prevent span collapses of the type shown in *Figure 14*, the new CHBDC specifications explicitly require that restrainers be provided at expansion bearings unless the support length is sufficient to accommodate the expected deflections. The minimum length of the support is given by the empirical equation:

$$N = K\left(200 + \frac{L}{600} + \frac{H}{150}\right)\left(1 + \frac{\psi^2}{8000}\right) \tag{4}$$

where $L$ is the length of the bridge to the adjacent expansion joint (mm), $H$ is the average substructure height (mm),

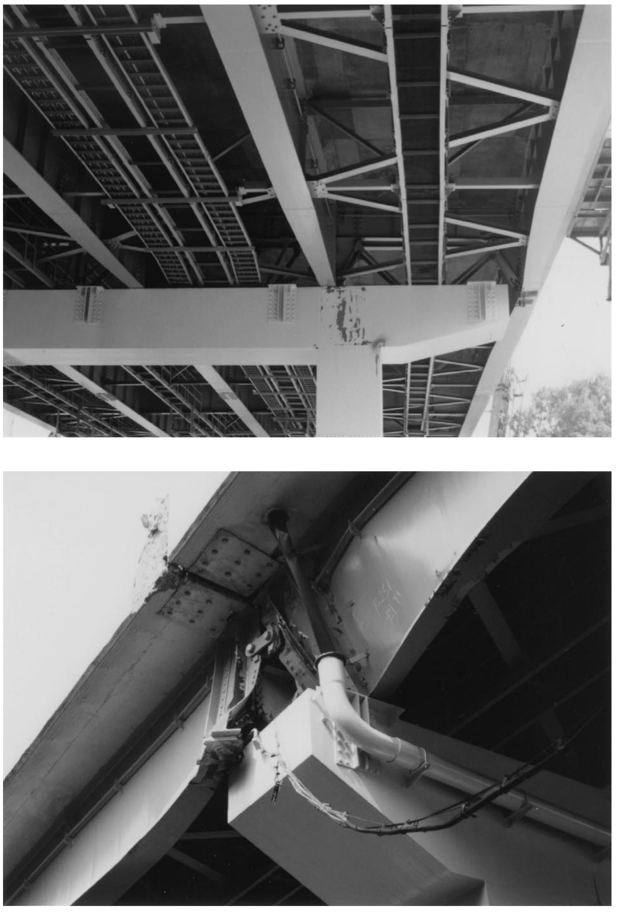

*Figure 17* Severe lateral-bending of steel girders as a result of bearing failure in the transverse direction

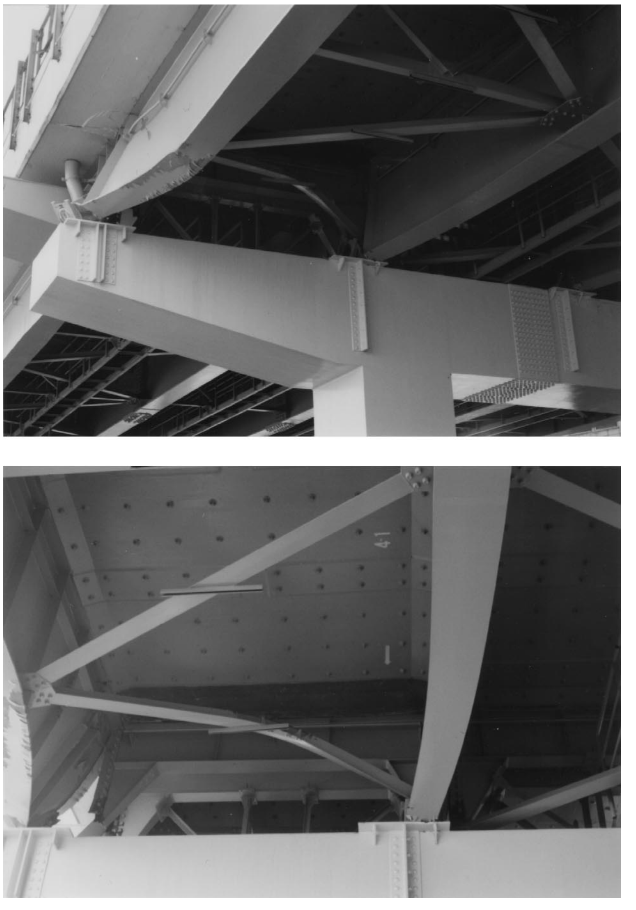

*Figure 18* Failure of connection between end-diaphragm and steel girder

$\psi$ is the skew angle in degrees, and $K$ is a modification factor which varies between 0.5 and 1.5, depending upon soil type and seismic risk at the site.

It is instructive to compare the support length calculated with the CHBDC equation with that calculated using the 1971 edition of the Japanese 'Specifications for Highway Bridges'[21] presented earlier. For comparison, a typical straight simple span of 30 m length and 11 mm pier height would require a minimum bearing seat length of 485 mm according to the new Canadian specifications (with $K$ of 1.5), while the 1990 Japanese Specifications would only require a 350 mm long seat.

Alternatively, seismic restrainers shall be used, such as ties, cables or other devices specifically designed for the purpose of limiting displacements at expansion bearings. The importance of these restrainers has already been illustrated in this paper. However, it was also observed that in many cases of spans collapses in Kobe, restrainers between adjacent spans were not connected to the underlying piers or columns. Typically, in most bridge codes, including the Japanese[21], connection of the girders to the piers and columns is strongly recommended, but not mandated. However, this is required by the new CHBDC for structures on poor soil conditions.

The CHBDC specifies that restrainer elements must be designed to resist a force equal to at least three times the Zonal Acceleration Ratio, $A$, multiplied by the dead load of the lighter of the two adjoining spans or parts of the structure. However, that factor $3A$ shall never be less than 0.2. In a manner compatible with the above capacity design philosophy, connections of the restrainer to the superstructure or substructure must be designed to resist 125% of the ultimate restrainer capacity.

### 4.5. Steel ductile substructure elements

The large tubular steel piers used in Japan are uncommon in Canada, and the new CHBDC does not include any provisions for their design. It provides, however, specific ductile detailing requirements for substructure elements consisting of ductile braced frames or ductile moment frames. These requirements are comparable to those currently enforced for ductile steel buildings in the National Building Code of Canada, with some minor modifications.

The objective is to ensure that steel substructure elements are detailed to be capable of exhibiting a ductility consistent with the $R$-values assumed in their analysis and design. Experience in past earthquakes[5,7,16,28] emphasizes the importance of ductile detailing in the critical elements of steel bridges. Research on the seismic behaviour of steel bridges[6,14,15,29] and findings from recent seismic evaluation and rehabilitation projects[9–11,17,18,30] further confirm that seismically induced damage is likely in steel bridges subjected to large earthquakes and that appropriate measures must be taken to ensure satisfactory seismic performance.

The same capacity design principles presented earlier also apply here. Explicit detailing requirements are presented in the CHBDC for ductile moment frames/bents and ductile concentrically braced frames used as substructure elements. The specifications refer to the Canadian Standard CAN/CSA S16.1-94, Limit States Design of Steel Structures for information on ductile eccentrically braced frames if necessary, and recommends using an $R$-factor of

*Table 1* Importance categories and performance objectives[12]

| Bridge | Other | Emergency-route | Lifeline |
|---|---|---|---|
| Return period | | | |
| Small to moderate earthquake | All traffic<br>Immediate use | All traffic<br>Immediate use | All traffic<br>Immediate use |
| Design earthquake (475 year) | Repairable damage | Emergency vehicles<br>Immediate use | All traffic<br>Immediate use |
| Large earthquake (1000 year) | No collapse | Repairable damage | Emergency vehicles<br>Immediate use |

*Table 2* Seismic performance zones[12]

| Range of peak horizontal ground acceleration (PHA), $g$, for 10% probability of exceedence in 50 years | Zonal acceleration ratio, $A$ | Seismic performance zone | |
|---|---|---|---|
| | | Emergency-route and other bridges | Lifeline |
| $0.00 \leq \text{PHA} < 0.04$ | 0 | 1 | 2 |
| $0.04 \leq \text{PHA} < 0.08$ | 0.05 | 1 | 2 |
| $0.08 \leq \text{PHA} < 0.11$ | 0.1 | 2 | 3 |
| $0.11 \leq \text{PHA} < 0.16$ | 0.15 | 2 | 3 |
| $0.16 \leq \text{PHA} < 0.23$ | 0.2 | 3 | 3 |
| $0.23 \leq \text{PHA} < 0.32$ | 0.3 | 4 | 4 |
| 0.32 or greater | 0.4 | 4 | 4 |

*Table 3* Minimum analysis requirements for multi-span bridges[12]

| Seismic performance zone | Lifeline bridges | | Emergency-route bridges | | Other bridges | |
|---|---|---|---|---|---|---|
| | Regular | Irregular | Regular | Irregular | Regular | Irregular |
| 1 | Not applicable | Not applicable | None* | None* | None* | None* |
| 2 | MM | MM | UL | MM | UL | SM |
| 3 | MM | TH** | MM | MM | UL | MM |
| 4 | MM | TH** | MM | MM | SM | MM |

*See Clause 4.4.5.1.
**In some cases, the use of the multi-mode method may be deemed appropriate for these bridges.
None = no seismic analysis required.
UL = uniform load method.
SM = single mode spectral method.
MM = multi-mode spectral method.
TH = time-history method.

5 in that case. Special bracing, energy-absorbing devices, or special ductile superstructure elements may also be used, but only if published research results, observed performance in past earthquake, or special investigation can demonstrate their adequate performance, and if permitted by the regulatory authority.

A complete review of the fundamentals of ductile steel detailing is beyond the scope of this paper, and available elsewhere[8,26,27]. However, a few noteworthy differences exist between the steel ductile detailing requirements features in the CHBDC specifications and those commonly found in building design standards, and some of the most important nuances are summarized below.

### 4.6. Materials

The specifications require that ductile substructure elements be constructed of steels capable of developing a satisfactory hysteretic energy during earthquakes, even at low temperatures if such service conditions are expected. Typically, such steel have $Fy \leq 0.8Fu$, can develop a longitudinal elongation of 0.2 mm/min in a 50 mm gage length prior to failure, and have probable-to-nominal strength ratios consistent with those implied in these specifications.

### 4.7. Ductile moment frames and bents

The prevailing philosophy in the seismic resistant design of ductile frames in buildings is to force plastic hinging to

*Table 4* Regular bridge requirements[12]

| Parameter | Value | | | | |
|---|---|---|---|---|---|
| Number of Spans | 2 | 3 | 4 | 5 | 6 |
| Maximum subtended angle (curved bridge) | 90° | 90° | 90° | 90° | 90° |
| Maximum span length ratio from span-to-span | 3 | 2 | 2 | 1.5 | 1.5 |
| Maximum bent or pier stiffness ratio from span-to-span (excluding abutments) | | | | | |
| (a) Continuous superstructure or multiple simple spans with longitudinal restrainers and transverse restraint at each support or a continous deck slab | – | 4 | 4 | 3 | 2 |
| (b) Multiple simple spans without restrainers or a continous deck slab | – | 1.25 | 1.25 | 1.25 | 1.25 |

Note: All ratios expressed in terms of the smaller value.

*Table 5* Response modification factors, *R*, for various ductile substructure elements[12]

| Ductile substructure element | Design and detailing requirements | *R* |
|---|---|---|
| Wall-type piers in direction of larger dimension | See note 1 | 2.0 |
| Reinforced concrete pile bents | | |
| • vertical piles only | See note 1 | 3.0 |
| • with batter piles | See note 1 | 2.0 |
| Single columns | | |
| • ductile reinforced concrete | See note 1 or note 2 as appropriate | 3.0 |
| • ductile steel | See note 1 or note 2 as appropriate | 3.0 |
| Steel or composite steel and concrete pile bents | | |
| •vertical piles only | See note 1 or note 2 as appropriate | 5.0 |
| •with batter piles | See note 1 or note 2 as appropriate | 3.0 |
| Multiple column bents | | |
| • ductile reinforced concrete | See note 1 or note 2 as appropriate | 5.0 |
| • ductile steel columns or frames | See note 1 or note 2 as appropriate | 5.0 |
| Braced frames | | |
| • ductile steel braces | See note 2 | 4.0 |
| • nominally ductile steel braces | See note 2 | 2.5 |

Note 1: As per special detailing provisions for ductile reinforced concrete elements.
Note 2: As per special detailing provisions for ductile steel elements.
Note 3: Other structural systems or materials shall have an *R* of 1.0 unless demonstrated otherwise to the regulatory authority.

occur in the beams rather than in columns, to better distribute hysteretic energy throughout all stories and avoid soft-story type failure mechanisms. However, for steel bridges such a constraint is not realistic, nor is it generally desirable. Steel bridges frequently have deep beams which are not typically Class 1 sections (i.e. compact sections as per U.S. designation), and which are much stiffer flexurally than their supporting steel columns. Moreover, bridge structures in Canada are generally 'single-storey' structures, and all the hysteretic energy dissipated is concentrated in this single story. The CHBDC provisions are therefore written assuming that steel columns will be the ductile substructure elements in moment fractures and bents. It is understood that extra care would be needed to ensure the satisfactory ductile response of multi-level steel frame bents since these are implicity not addressed by these specifications.

For that reason, ductile detailing requirements are only specified for columns in ductile moment frames and bents. Hence, columns must be Class 1 sections (i.e. U.S. compact sections), must have lateral supports at the potential plastic hinge locations (near their top and base) and other lateral supports as necessary to limit the unsupported length to $980r_y/\sqrt{F_y}$, and cannot be subjected to factored axial compression in excess of $0.30A_gF_y$ due to the combined effect of seismic load and permanent loads if in seismic zones 3 and 4 (twice that value if in seismic zone 2). Other usual detailing requirements for ductile columns also apply[8].

Beams, panel zones, column bases, and moment resisting connections are designed as capacity protected elements, following the principles presented earlier, to remain elastic. To ensure the strong beam-weak column behaviour implied by the CHBDC, the sum of the factored resistance of the beams at any beam-to-column joint cannot be less than the sum of the probable resistance of the column(s) framing into the joint. The Probable Resistance of columns shall be taken as 1.25 times their nominal flexural capacity given by:

$$1.18M_{px}\left[1-\frac{C_f}{AF_y}\right] \leqslant M_{px} \tag{5}$$

## 4.8. Ductile concentrically braced frames

The same capacity design principles also apply to ductile braced frames. As normally done for ductile braced frames in building designs[2,8]:

- Braces are the energy dissipating elements.
- The load redistribution following the yielding or buckling of braces must be taken into account, and capacity-protected elements (such as columns, beams, beam-to-column connections and column splices, to name a few) must be designed to resist the most detrimental conditions that could result from this redistribution.
- Diagonal braces shall be oriented such that, in any planar frame, at least 30% of the horizontal shear carried by the bracing system shall be carried by tension braces and at least 30% shall be carried by compression braces.
- Chevron bracing, V-bracing, K-bracing and knee-bracing are not considered as ductile concentrically braced frames.
- Braces must have a slenderness ratio, $L/r$, less than $1900/\sqrt{F_y}$.
- Symmetrical open sections shall be Class 1 (i.e. compact sections). For other sections, the width-thickness ratios is limited to $145/\sqrt{F_y}$ for angles, tees, and flanges of channels, $330/\sqrt{F_y}$ for rectangular and square HSS, and $13\,000/Fy$ for circular HSS.
- The factored compressive resistance of a brace must be reduced to account for the loss of compressive resistance under cyclic loading. This reduction is a function of the brace's slenderness ratio.

The CHBDC specifications also provide detailing requirements for concentrically braced frames with nominal ductility. Their energy dissipation capabilities is somewhat less than ductile braced frames, and this is reflected by a lower $R$-factor. However, Chevron-type braced frames are included in this designation.

## 5. Conclusion

When compared to concrete bridges of similar vintage, the seismic performance of steel bridges was generally good during the Hyogoken–Nanbu earthquake. However, many older and some new steel bridges suffered considerable 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. Bearing damage was extensive, and numerous spans collapsed. Some seismic restrainers failed to perform as intended. Given the excellent earthquake preparedness and superior past design practice in Japan compared to Canada, this earthquake has provided valuable insight into the potentially disastrous seismic-performance of comparable bridge structures in future Canadian (and likely North American) earthquakes. Hopefully, the seismic provisions of the new Canadian Highway Bridge Design Code will ensure that only bridges with ductile substructure elements will be designed and constructed in the future in Canada. The extensive detailing requirements for steel substructure elements provided for that purpose as part of that code have been briefly reviewed in this paper along with other key relevant features of those bridge design specifications.

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