Seismic design of steel buildings: lessons from the 1995 Hyogo-ken Nanbu earthquake

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Abstract: Past and current seismic design provisions for steel structures in Japan are presented and compared with Canadian requirements. The performance of steel framed structures during the January 17, 1995, Hyogo-ken Nanbu earthquake is described. Numerous failures and examples of inadequate behaviour could be observed in buildings of various ages, sizes, and heights, and braced with different structural systems. In moment resisting frames, the damage included failures of beams, columns, beam-to-column connections, and column bases. Fracture of bracing members or their connections was found in concentrically braced frames. The adequacy of the current Canadian seismic design provisions is examined in view of the observations made.

Key words: earthquake, seismic design, steel structures.

Résumé : Dans cet article, on présente l'évolution des normes de conception parasismiques japonaises pour les constructions métalliques. Le comportement des charpentes d'acier lors du séisme de Hyogo-Ken Nanbu du 17 janvier 1995 est ensuite décrit. Plusieurs exemples de rupture et de comportement inadéquat ont pu être observés dans des bâtiments de différentes époques, dimensions et hauteurs qui étaient contreventés par divers systèmes de résistance aux charges latérales. Les dommages aux cadres à noeuds rigides consistaient en des ruptures de poutres, de poteaux, d'assemblages poutre-poteau et de plaques de base des poteaux. Dans les contreventements en treillis concentriques, on a observé des ruptures de diagonales ou de leurs assemblages. L'adéquation des normes parasismiques canadiennes est revue sur la base de ces observations.

Mots clés : tremblement de terre, conception parasismique, structures d'acier.

Introduction

The January 17, 1995, Hyogo-ken Nanbu earthquake in Japan produced extensive damage in the industrialized area of Kobe city. The strong ground shaking was very short in duration in the city, typically less than 10–15 s (AIJ 1995*a*), but exhibited high peak accelerations and powerful acceleration pulses which drove many structures to, and even beyond, their capacity.

Damage to building structures was mainly concentrated in a very narrow band, 200–300 m in width, extending for about 25 km long from Suma Beach (west of Kobe) to Nishinomiya City as shown in Fig. 1. The intensity of damage in that region reached the highest level (7) on the Japan Meteorological Agency (JMA) intensity scale (Shindo scale), which approximately corresponds to a MM XI–XII intensity (Kobe University 1995).

An unusually large percentage of steel structures were seriously affected by the earthquake. According to a research group of the Kinki Branch of the Architectural Institute of Japan (AIJ 1995*b*), which conducted an extensive survey in the Kobe area a few days after the event, a total of 3406 buildings were reported to have sustained some level of damage, of which 1247 were steel framed structures. Of these damaged steel buildings, 286 had collapsed or were sufficiently damaged to be considered in danger of collapsing during aftershocks. Most of the damaged steel buildings had three storeys or less and were old apartments, offices, or shopping buildings, and were likely designed according to codes that had less stringent earthquake provisions. Many of them were constructed with light-gauge steel members.

Contrary to first impressions, however, which were based on observations made mostly from the outside of buildings by many reconnaissance teams shortly after the earthquake, a large number of modern steel buildings designed to the latest code requirements have also suffered damage. Similarly to what happened in California after the 1994 Northridge earthquake (Tremblay et al. 1995*b*), a significant percentage of this damage was discovered through more in-depth investigations, typically while searching for the cause of noticeable residual lateral deflections.

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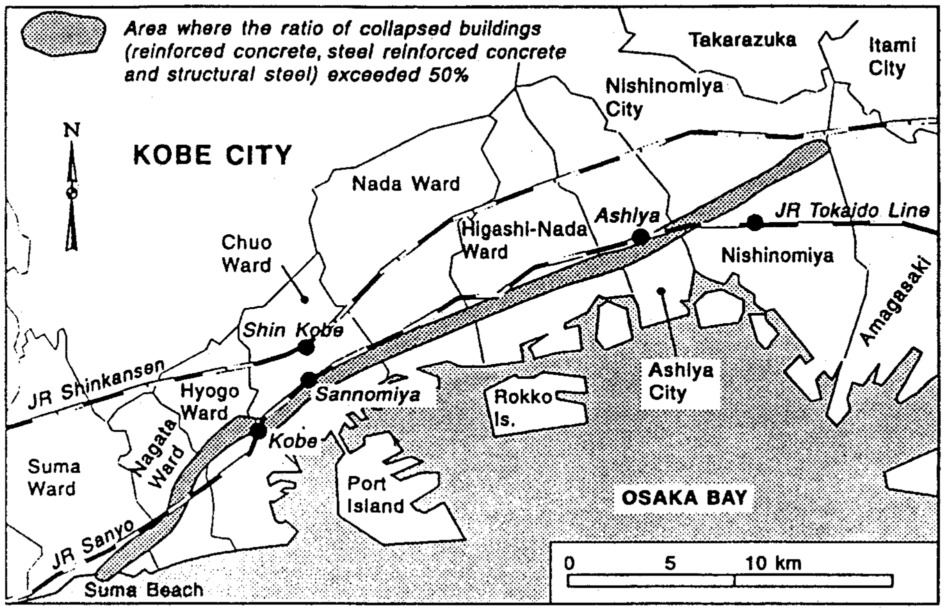
Fig. 1. Area of intense damage to buildings (adapted from Park et al. 1995).



In this paper, past and recent Japanese seismic design requirements and practices are described and compared with current Canadian provisions. The performance of steel buildings during the Hyogo-ken Nanbu earthquake is thereafter presented, with emphasis on the quality or inadequacies in design or construction practices. The paper mainly addresses the performance of moment resisting frames, concentrically braced frames, and structures including a combination of these two systems. The paper concludes with a section including recommendations for possible improvements of Canadian seismic design provisions.

Japanese seismic design practices

The history of earthquake resistant design in Japan can be divided into two main periods: pre-1981 and post-1981. Very few changes have occurred in the seismic provisions of the Japanese building code from its first edition in 1924 until 1981. Since June 1, 1981, a completely revised code has been enforced that includes a two-level design procedure for earthquakes. Minor amendments were made in 1987, but the seismic provisions of the 1981 edition of the Building Standard Law were still in use at the time of the earthquake.

The pre-1981 seismic provisions

The concept of obtaining seismic design loads by multiplying the weight of the structure by a seismic coefficient was proposed for the first time in Japan in 1916 (Ishiyama 1989). This approach was, however, included only in the Urban Building Law of Japan one year after the great 1923 Kanto earthquake (M = 7.8), which killed 140 000 people in Tokyo. The prescribed minimum seismic coefficient was equal to 0.1 and the lateral load was uniformly distributed over the height of the building. Sizing of the members was based on an allowable stress design approach. Until World War II, the stress limit was equal to one third of the yield stress of the material for any loading combination. Because of shortage in material during the war, a dual allowable stress design was implemented where stresses up to 50% of the yield stress were permitted for temporary loading such as earthquake-induced forces.

The Urban Building Law was compulsory in six major cities and some districts of Japan (Umemura 1989). In 1950, it was replaced by the National Building Standard Law in which the seismic coefficient for earthquake design was increased to 0.2. The level of protection for steel structures, however, remained virtually unchanged, as the increase in the seismic coefficient was offset by a comparable increase in the allowable stress. Thus, a structure was deemed adequate if its resistance at first yield was equal to or exceeded the effects of the prescribed seismic loads.

The National Building Standard Law mainly specified the design loads and the corresponding allowable stresses. Details of structural design were issued by the Architectural Institute of Japan in a specific standard for steel structures. The Architectural Institute of Japan standards were revised periodically to incorporate new knowledge about the behaviour of steel frames. On the occasion of holding the Third World Conference on Earthquake Engineering in Japan in 1964, the Architectural Institute of Japan published its structural standards in English (AIJ 1964). This publication is important since many buildings in the damaged area of Kobe had been designed according to its seismic provisions.

The appendix of this standard contains an excerpt from the Building Standard Law, including Article 88 for seismic design forces. It is stated that "the seismic force shall be calculated by multiplying the sum of dead and live loads (including snow loads) with a coefficient of horizontal force (or seismic coefficient)." The coefficient was equal to 0.2 for buildings not exceeding 16 m in height (four storeys) and 0.2 + 0.01 for every increase of 4 m in height (one storey) for other buildings. The vertical distribution of the lateral load was still uniform. The section dealing with structural calculations of steel structures indicated that design was based on an allowable stress, which was equal to the yield stress for load combinations that included earthquake effects.

The standard did not specify any particular grade of steel for earthquake resistant elements nor any specific seismic design detailing requirements.

Also of significance in the evolution of the seismic provisions in Japan is the abolishment in 1963 of the building height limitation of 31 m, which had been in force since the adoption of the Urban Law (Ishiyama 1989).

In 1968, the Tokachi-oki earthquake caused significant damage to buildings designed according to the 1964 edition of the Architectural Institute of Japan standards. As a result, minor revisions were implemented in the Building Standard Law and major changes were incorporated in the Architectural Institute of Japan standards. These changes, however, were aimed mainly to improve the performance of reinforced concrete buildings and did not significantly affect the seismic design of steel structures. Nevertheless, a review procedure was established for evaluating the seismic safety of existing structures, including steel buildings.

The post-1981 seismic provisions

Following the 1971 San Fernando, California, earthquake, the Ministry of Construction of Japan initiated a 5-year national research project to establish a new and rational seismic design procedure for structures. The results of this project were released as a proposal in 1977, which then went through a 4-year review process. In 1978, the Miyagiken-oki earthquake caused severe damage in the urban area of Sendai City. After this event, momentum to implement the Ministry of Construction proposal built up and it was decided to publish an entire new building code based on this proposal. This code includes extensive seismic provisions of which those relevant to this paper are summarized as follows.

The main characteristic of this code is the two-level design procedure. It is meant to ensure a satisfactory performance under earthquakes likely to occur several times during the lifetime of the structure, and to avoid collapse in case of severe earthquakes that are capable of producing damage intensity of 6 or 7 on the Shindo Japanese scale. Significant changes have also been introduced for the calculation of the seismic loads. The loads now vary according to the soil conditions at the site and also depend on the building location, natural period, actual strength, and ductility.

In addition, the designer has a choice of following different design procedures depending on the height and (or) size of the building and the major material (namely steel, concrete, or timber) used for the structure. The different design routes and associated criteria have been presented and discussed by many authors (Aoyama 1981; Ishiyama 1984, 1989; IAEE 1992; Kitagawa and Takino 1994). The design flow chart applicable to steel buildings is shown in Fig. 2. As indicated, no design calculation is required for single-storey buildings smaller than 200 m^2, whereas a special investigation, including nonlinear time-step dynamic analyses, must be performed for structures taller than 60 m. The design of these tall frames is discussed further below. For all other buildings, the two-level design procedure must be followed. For some building categories, however, only the first level is mandatory.

The first-level calculation is very similar to the allowable stress design performed under earlier codes. The allowable stress is still taken equal to the yield stress of the steel material. The magnitude and the distribution of the seismic loads along the height of the structure have been changed, however. The base shear coefficient C is given by

$$[1] \quad C = 0.2ZR_t$$

where Z is the seismic zone factor and R_t is the vibration characteristic factor obtained from a design response spectrum. In Kobe, the Z factor takes a value of 1.0, as in Tokyo, Osaka, and Kyoto. The factor R_t is a function of the fundamental period of the structure and includes soil effects (Fig. 3). Its maximum value is equal to 1.0 for short-period structures. For steel frames, either braced frames or moment resisting frames, the fundamental period is assumed to be $0.03h$, where h is the height of the structure in metres. It is noteworthy that this period is 50% longer than the value prescribed for concrete frames. The period as computed from a dynamic analysis can be used in lieu of the specified value. The value of R_t so obtained, however, cannot be less than 75% of the one determined with the code-specified period. As shown, three soil categories are considered. Soil type 1 includes rock, hard sandy gravel, or stiff sands. Soil type 3 consists of alluvium or other soft soils with a depth of at least 30 m. It also includes recent (less than 30 years) reclaimed lands where the depth of reclaimed material is greater than 3 m. Soils not classified as types 1 or 3 are included in the soil profile 2 category.

The distribution of the earthquake forces along the height of buildings is no longer uniform. Higher forces are applied near the top of the structure. Deviation from a uniform distribution accentuates with the period of the structure.

For buildings of less than three storeys in height, maximum overall height of 13 m, maximum beam span of 6 m, and maximum floor area of 500 m^2, only the first-level design for moderate earthquakes needs to be considered and the second phase can be omitted. To compensate for the lack of design requirements for stronger earthquakes, design forces in the first-level design must, however, be increased by 50%. In braced frames, the ultimate capacity of the bracing member connections must also exceed 1.2 times the yield capacity of the braces to prevent a sudden failure of the main lateral load resisting system.

For buildings not meeting these criteria, the design procedure then requires the storey drift to be evaluated and the degree of uniformity of the structure to be quantified prior to carrying out the second-level design. Storey drifts are computed under the first-level earthquake loads and must not exceed 0.5% of the storey height. The in-plane eccentricity ratios in each of the principal directions and the ratio of the storey shear stiffness to the average storey shear stiffness (computed over the building height) are determined at each level. These ratios are discussed in detail by Mitchell et al. (1996).

At this stage, it is still possible to avoid the second-level design for buildings up to 31 m in height (typically eight storeys), provided the building is of regular shape. At any level, the storey shear stiffness must lie within 40% of the average storey stiffness and the eccentricity ratios must be less than 0.15. If this option is retained, the earthquake loads for the first-level design must be increased by the lesser of 1.5 and $(1 + 0.7\beta)$, where β is the ratio of the storey shear carried by the bracing members, if any, to the total storey shear. Brace connections must be designed to resist 1.2 times

Fig. 2. Design flow chart for steel buildings according to the 1981 Building Standard Law of Japan.

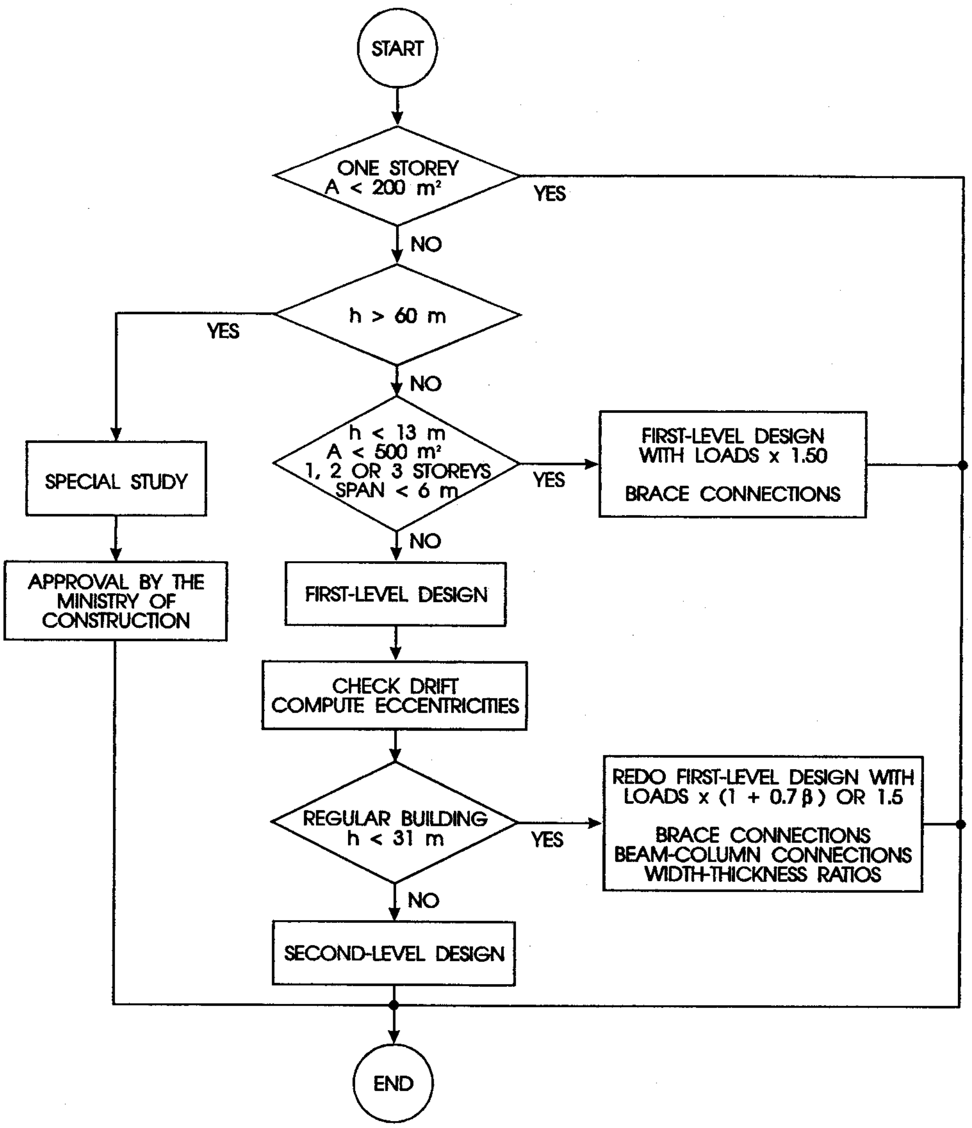
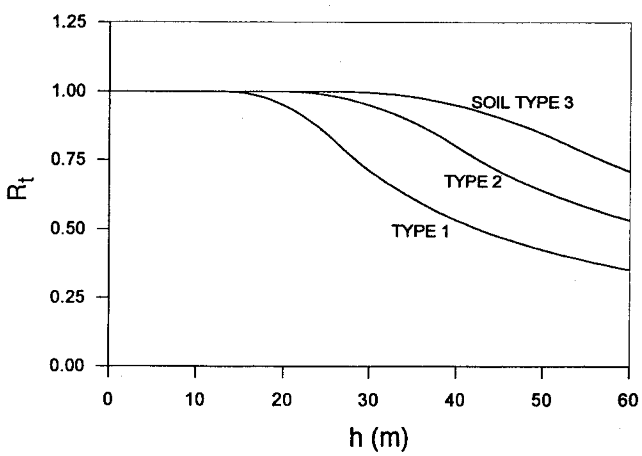


Fig. 3. 1981 Building Standard Law vibration characteristic factor.



the yield load of the brace, and beam-to-column connections subjected to bending moments must be capable of sustaining 1.2 to 1.3 times the full plastic moment of beams and columns. The width-to-thickness ratio of beams and columns must also be limited to avoid the occurrence of local buckling prior to overall yielding of the members. Moment resisting frames must classify as type FA or FB (Table 1) to achieve this behaviour.

The second-level design aims at ensuring that the ductility demand on the elements during strong earthquakes will not exceed the ductility capacity of the structural system. This is achieved by verifying that the actual ultimate storey shear strength of the structure exceeds the storey shear due to amplified earthquake loads. The base shear coefficient in this case is equal to

$$[2] \quad C_u = D_s F_{es} Z R_t$$

The structural coefficient D_s varies from 0.25 to 0.50,

Table 1. Maximum width-to-thickness ratio for beams and columns of moment resisting frames.

Member	Section	Portion	Steel	Type of frame FA	FB	FC
Columns	H-shaped	Flange	SS41*	9.5	12	15.5
			SM50†	8	10	13.2
		Web	SS41	43	45	48
			SM50	37	39	41
	Box		SS41	33	37	48
			SM50	27	32	41
	Round tube		SS41	50	70	100
			SM50	36	50	73
Beams	H-shaped	Flange	SS41	9	11	15.5
			SM50	7.5	9.5	13.2
		Web	SS41	60	65	71
			SS50	51	55	61

*Steel conforming to SS41, SM41, SMA41, STK41, and STKR41 of the Japan Industrial Standard ($F_y \cong 235$ MPa, $F_u \cong 400$ MPa).

†Steel conforming to SM50, SMA50, SM50Y, STK50, and STKR50 of the Japan Industrial Standard ($F_y \cong 325$ MPa, $F_u \cong 490$ MPa).

Table 2. Structural coefficient D_s for steel structures.

Type of moment frame*	Type of braced frame‡: BA, $\beta_u = 0$	BB, $\beta_u \le 0.3$	BB, $0.3 < \beta_u \le 0.7$	BB, $\beta_u > 0.7$	BC, $\beta_u \le 0.3$	BC, $0.3 < \beta_u \le 0.7$	BC, $\beta_u > 0.7$
FA	0.25	0.25	0.30	0.35	0.30	0.35	0.40
FB	0.30	0.30	0.30	0.35	0.30	0.35	0.40
FC	0.35	0.35	0.35	0.40	0.35	0.40	0.45
FD†	0.40	0.40	0.45	0.50	0.40	0.45	0.50

*See Table 1 for types of moment resisting frames.

†Width-to-thickness ratios for beams and columns of type-FD frames exceed the maximum specified for type-FC frames.

‡See Table 3 for types of braced frames.

depending upon the ductility exhibited by the structural system. The level of ductility is based on the width-to-thickness ratio of beams and columns in moment resisting frames, the proportion of the ultimate storey shear resisted by the bracing members (β_u), and the effective slenderness ratio of the braces. Table 2 gives values of D_s as a function of the structural system whereas the requirements for the different types of moment resisting frames and braced frames are presented in Tables 1 and 3, respectively. Lower values of D_s are assigned to frames with more compact sections and where the portion of storey shear carried by bracing members is reduced. The shape factor F_{es} accounts for irregularities in the structure, such as in-plane eccentricities and variation in the storey stiffness along the height of the building. It varies from 1.0 for regular buildings to 2.25 for very irregular buildings. Z and R_t have been defined earlier.

The ultimate strength of the structure can be obtained by any suitable method, including computerized static ultimate load (push over) analysis or simplified methods appropriate for hand calculations. For regular buildings, the design forces at the second design stage lie between 1.25 and 2.5 times the ones used for the first-level design. Higher figures are expected for irregular structures.

Table 3. Brace slenderness ratio for braced frames.

Type of frame	Effective brace slenderness ratio, λ
BA	$\lambda \le 495/\sqrt{F_y}$
BB	$495/\sqrt{F_y} \le \lambda \le 890/\sqrt{F_y}$
BC	$890/\sqrt{F_y} < \lambda \le 1980/\sqrt{F_y}$

Also mandatory in the second-level design is the verification that brace connections and beam–column connections can develop the capacity of the framing members and that beams are properly supported to prevent out-of-plane buckling.

The second-level design thus appears to be a checking procedure rather than a design operation. Member sizes are determined in the first phase of the design process and then modified as needed to comply with the ultimate strength requirement of phase 2. Since the allowable stress in the first-level design corresponds to the yield stress of the material, redundant systems capable of developing substantial overstrength (moment resisting frames with beams carrying substantial gravity loads, tension-compression acting braces, dual systems, etc.) must be employed to develop the extra capacity required in design level 2. If the required amount of overstrength cannot be developed by the structural system chosen, stronger members must be selected to match the required ultimate strength.

Special studies for structures taller than 60 m include two-level time-history dynamic analyses where the structure is subjected to historical or artificial earthquake ground motions. In practice, a first-level design, as described earlier, is performed to obtain preliminary member sizes. Thereafter, the dynamic analyses are performed to check the behaviour of the structure against pre-established performance criteria: (*i*) under frequent earthquakes, the response must remain essentially linear elastic and the storey drift must be less than 0.5% of the storey height, and (*ii*) under rare but severe earthquakes, the storey drift must be within 1% of the storey height and the ductility demand must be less than the prescribed limits. The overall storey drift ductility is limited to 2.0 while the rotational ductility for columns and beams must be less than 2.0 and 4.0, respectively.

Once completed, the design of tall structures must be submitted for review to the High-Rise Building Structure Review Committee of the Building Centre of Japan. Upon a recommendation from this committee, a special approval of the design can be issued by the Ministry of Construction.

For the Kobe area (seismic zone $Z = 1.0$), the input ground motions to be used in the dynamic analyses are normalized to peak horizontal velocities of 0.25 and 0.50 m/s for the serviceability and ultimate levels, respectively (Omika and Sugano 1989; Wada et al. 1994). A 2% damping ratio is recommended for the analysis of steel structures, whereas concrete frames are assigned damping values ranging between 3% and 5%.

According to this procedure, tall frames can withstand higher levels of ground motion than low-rise buildings. Based on the first-level design requirements, yielding of low-rise buildings is expected to occur under earthquakes whose peak acceleration is equal to $0.08g$. This figure is obtained by dividing the seismic coefficient for short-period structures (0.2) by a spectral amplification factor of 2.5. For tall frames, the response must remain elastic under ground motions exhibiting a peak velocity of 0.25 m/s. This clearly is more stringent than the performance level required for low-rise frames, since earthquake ground motions exhibiting such a peak velocity can display peak accelerations ranging between about $0.13g$ and $0.50g$, assuming the peak acceleration to peak velocity ratio to vary from 0.5 to 2.0. Similar comparison can also be made for the ultimate limit state (level 2).

Comparison between design practices

The prescribed base shear at first yield remained essentially unchanged over the years in Japan. In 1981, it was even reduced for long-period structures in recognition of the lower response of these buildings to earthquake ground motions. For these structures, however, a more realistic vertical distribution of the seismic loads (with higher forces prescribed in the upper storeys) was adopted to achieve a better uniformity in the ductility demand throughout the lateral load resisting system. A comparison between the 1964 and 1981 seismic coefficients at first yield is given in Fig. 4 for hard soil conditions in the Kobe region ($Z = 1.0$).

The other major modifications introduced in the 1981 code were mainly aimed at ensuring sound performance and thus improving safety against collapse during strong earthquakes. For example, connection overstrength was prescribed to avoid the occurrence of premature nonductile failure of connections and to maintain the integrity of the structure. The stringent inelastic drift limitations applicable to frames taller than 60 m contribute towards preventing the formation of soft storey mechanisms. In some cases, the design requirements provide a strong incentive for Japanese owners or engineers to produce more desirable structural systems for seismic resistance. For instance, the penalty associated with large eccentricities or vertical nonuniformity (factor F_{es}) strongly motivates designers to adopt regular structural forms, whereas the ultimate strength check encourages the use of redundant lateral load resisting systems made of ductile components.

In the National Building Code of Canada (NBCC) (NRCC 1995), which is similar to other building codes in North America (e.g., ICBO 1994), only the first-phase design is mandatory for buildings of any size. Neither the ultimate strength of the structure nor the anticipated ductility demand needs to be checked explicitly. The seismic coefficient at first yield in the NBCC is equal to

$$[3] \quad C = \frac{vSFI}{\phi}\left(\frac{U}{R}\right)$$

where ϕ is the resistance factor for the material (equal to 0.9 for steel). The zonal velocity ratio v in [3] plays the same role as Z in [1], and the product of the seismic response factor S and the foundation factor F corresponds to the factor R_t in the Japanese code. The force modification factor R accounts for the capacity of the structure to withstand earthquakes in the inelastic range and U is a calibration factor equal to 0.6. There is no equivalent to the importance factor I in the Japanese specifications. Hence, structures in Japan are assigned the same resistance level regardless of their expected postearthquake function.

Considering the base shears only can be misleading when trying to compare safety levels achieved in each country. Many other aspects and parameters of the design process also need to be considered to establish an equal basis for comparison. Nevertheless, such a comparison can provide some valuable insight and helps in identifying possible weaknesses of Canadian code provisions or in assessing the extent of damage that an earthquake similar to the Hyogo-ken Nanbu event could produce in Canada.

In Fig. 4, the range of base shears as given by [3] is illustrated for the range of R factors applicable to steel structures in Canada (1.5 to 4.0). The calculations have been performed for the city of Victoria, B.C. ($v = 0.30$), where the seismic conditions are very similar to those encountered in Kobe: likelihood of nearby intraplate M7 earthquakes and larger interplates events at some distance (Byrne et al. 1995). The foundation and importance factors were set equal to 1.0 and the seismic response factor S was determined according to the NBCC provisions, assuming equal acceleration-

Fig. 4. Seismic coefficient at first yield in Japan and Canada for steel structures (ϕ = 0.9) on hard soil conditions.

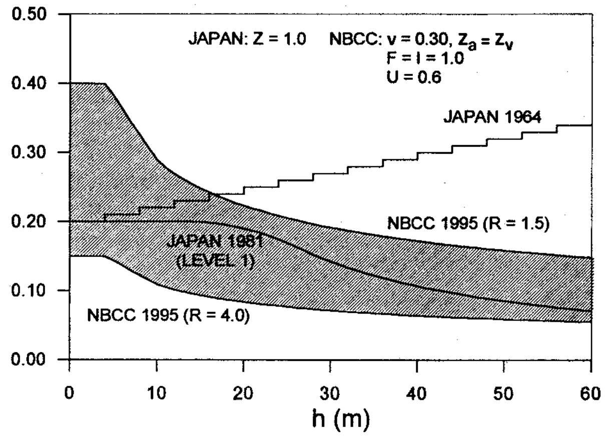
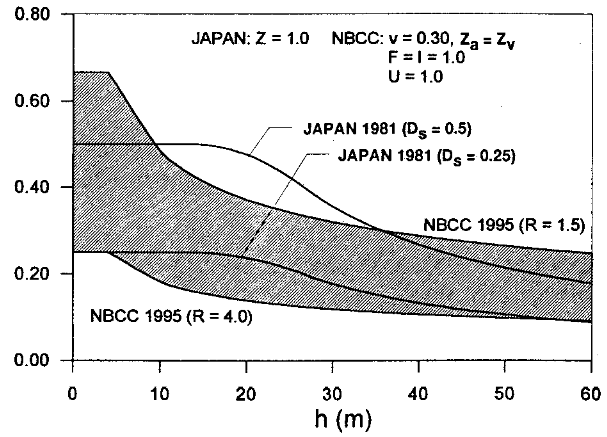


Fig. 5. Seismic coefficient at ultimate in Japan and Canada for steel structures (ϕ = 0.9) on hard soil conditions.



and velocity-related zones ($Z_a = Z_v$), which is the case in Victoria. The period T used in the calculation of the S factor was obtained using [4], which applies for a moment resisting frame type of structure.

[4] $T = 0.085h^{3/4}$

where h is the height of the building in metres. The first observation in Fig. 4 is the fact that, contrary to the philosophy adopted in Canada, all structural systems in Japan are designed for the same earthquake loads, with no consideration of the ductility of the chosen structural system. Since 1981, this shortcoming has been rectified in part by the application of the second-level design or alternative design provisions (Fig. 2).

For structures less than three storeys in height, the seismic loads prescribed in Japan appear relatively low when compared with those specified by the NBCC. Note, however, that for most of these structures built in Japan since 1981, only the first-level design was carried out, for simplicity. Hence, amplified design loads and (or) stringent detailing rules had to be considered in the design.

Taller frames designed according to pre-1981 Japanese specifications (1964 edition or older) would likely possess extra base shear capacity when compared to similar buildings in Canada, but these structures may lack the required storey shear resistance in the upper storeys owing to the uniform vertical distribution of the loads assumed in these old specifications. Conversely, the 1981 Japanese base shear lies within the range of NBCC values for the entire range of building heights. The difference between both base shears varies, however, with the building height. For short-period structures and tall frames, it corresponds to a NBCC base shear computed with an R factor of up to about 3.5, whereas 10–30 m high buildings (4–8 storeys) are assigned a relatively higher resistance, which is equivalent to that obtained with R varying between 2.1 and 2.8. Hence, medium-rise structures in Canada would probably experience a relatively higher inelastic demand than in Japan.

Also, structures taller than 60 m, and designed according to the 1981 Japanese code, would likely remain elastic during the design earthquake for Victoria, since the design peak horizontal velocity in this city (0.26 m/s) is nearly the same as the target peak velocity used in the serviceability limit state dynamic analyses.

Although there is no explicit requirement for the minimum ultimate storey shear resistance in the NBCC, the minimum resistance specified in Japan can be compared to the expected ultimate strength of Canadian structures. The latter can be obtained by multiplying NBCC seismic loads at first yield by a realistic overstrength factor. Figure 5 compares the seismic coefficients at the ultimate limit state for the range of structural systems of each country. For Japan, the values of C_u as obtained from [2] with D_s equal to 0.25 and 0.5 are given. For the expected capacity of structures designed according to NBCC, an overstrength factor equal to $1/U$ (= 1.67) was used, that is, C_u was obtained using [3], assuming U = 1.0. This overstrength factor can be seen as a lower bound estimate of the actual overstrength of ductile systems with some minimum redundancy. With this assumption, Fig. 5 indicates that structures designed according to NBCC would generally develop the minimum ultimate strength prescribed by the Japanese specifications. Stronger structures could, however, be needed in the 10–30 m range.

This examination of design base shears shows that building structures located in Kobe and along the coastal region of British Columbia exhibit comparable ratios of yield and ultimate resistance to earthquake loads, which seems satisfactory in view of the similarities between the seismic settings in both areas. Only buildings taller than 60 m in Japan would appear to have a significantly higher yield resistance than similar structures in Canada.

In Canada, however, many regions other than the Victoria area can be struck by earthquakes similar in magnitude to the Hyogo-ken Nanbu event (Basham et al. 1982). Due to the probabilistic approach adopted in the calculation of the NBCC design ground motion parameters, many of these regions in Canada are currently assigned values of v lower than that for Victoria. Structures located in these regions would thus possess less resistance to earthquakes than buildings in Kobe, and would probably experience more significant inelastic action under a similar seismic event.

Detailing is another key issue in earthquake resistant design. While most of the current seismic detailing provisions in Canada are similar to those provided by the Architectural

Institute of Japan standard, certain aspects are worth some discussion. In Canada, steel buildings, for which a ductile response is expected under seismic loading, must be designed according to seismic design requirements included in the S16.1 Standard for the design of steel structures (CSA 1994). These requirements were introduced for the first time in 1989, eight years later than in Japan. Structures designed before that year essentially rely on the inherent ductility of the steel material and possible structural redundancy to withstand earthquakes.

For moment resisting frames, the maximum width-to-thickness ratio for beams and columns of frames type FA and type FB in Japan (Table 1) closely correspond to the S16.1 class 1 and class 2 requirements, respectively. Hence, members of FA frames would be adequate for use in a ductile moment resisting frame (R = 4.0) in Canada, while FB members would be restricted to moment resisting frames in the nominal ductility category (R = 3.0).

Beam-to-column connections in Japan must be capable of developing 1.2 to 1.3 times the capacity of the joined members, regardless of the frame category. A similar provision exists in Canada, but it only applies to ductile moment resisting frames. No such requirement is imposed on moment resisting frames with nominal ductility (R = 3.0) and ordinary moment resisting frames designed with R = 1.5. Furthermore, the 20% margin specified in the S16.1 for ductile moment resisting frames, which account for overstrength of the connected members (strain hardening, etc.), needs to be considered only for the beams, columns, and column shear panel zones. It is omitted for the moment-resisting beam-to-column connections. In Japan, however, structures of not more than three storeys in height, for which the second-level design is avoided (Fig. 2), do not need to meet the minimum beam-to-column connection strength requirement. As in Canada, no beam-to-column connections are officially preapproved by authorities, but numerous standard details are used by the industry (see below) as a result of a consensus approach.

Very stringent brace slenderness limitations are imposed on braced frames in Japan. For instance, the limit for BA frames is four times more severe than the limit prescribed for the ductile braced frame category (R = 3.0) in Canada. For low strength steels (F_y = 240 MPa), the brace slenderness must not exceed 32 in BA frames, a value more often encountered for columns at lower floors of multistorey structures. In fact, the S16.1 slenderness limit for ductile braced frames corresponds to the limit for the less ductile system in Japan (frame BC). Note, however, that Japanese designers are allowed to use the effective brace slenderness in the calculations, whereas the S16.1 Standard refers to the gross slenderness of bracing members. Interestingly, no maximum width-to-thickness ratio for bracing members is imposed specifically for seismic-related reasons in Japan, although practising engineers have generally used FA as the limit for braces.

In Japan, specified brace connection design forces are 20% higher than member capacities in any braced frame. Canadian provisions do not call for such 20% extra capacity, and the verification of brace connections is only required for ductile braced frames (R = 3.0) and braced frames with nominal ductility (R = 2.0). For ordinary braced frames (R = 1.5), no such verification is required. Moreover, for ductile braced frames and braced frames with nominal ductility, the S16.1 allows the use of reduced connection forces in regions of low seismic activity or if the design engineer can show that a lower resistance is adequate.

As in Canada, Japanese standards do not require a particular type of steel for seismic resistance. Three steel grades, A, B, and C (i.e., referenced as SM50A or SM50B), are typically available in Japan. Type C is rarely used. Type B has a notch toughness of 27 J at 0°C (same as in the U.S.A.). Type A is the most common steel and has no notch toughness requirements. For thick sections of 38 mm or more, conscientious designers will call for type B to ensure welding quality. Note that the Japanese steel designation refers to the ultimate strength. Thus, a SM50 steel has an ultimate strength of 5 t/cm^2 (i.e., 490 MPa) and its yield strength is approximately equal to 325 MPa, as indicated at the bottom of Table 1.

Qualification for structural engineers

Structural engineers in Japan work either for design firms or general contractors and, to act as such, must be registered in a manner comparable to the Canadian practice. After completion of the 4-year curriculum in architecture or civil engineering and 2 years of practice in their field of specialization, Japanese structural engineers can obtain from the Ministry of Construction a 1st class building engineer license which enables them to assume full responsibility for designing or supervising the construction of any structure in Japan (Yano 1994).

Recognizing the growing complexity of projects and the constant evolution in design techniques and materials, the Japan Structural Consultants Association (JSCA) has been established in 1989 to provide a voluntary qualification system for structural engineers (Aoki 1994). The main objectives of the association are the certification of structural engineers active in the design of architectural structures and to gather and keep record of the technical expertise of structural engineers. Applicants must possess a 1st class building engineer license with 4 years of structural design experience. They must also successfully pass a registration examination, which includes a written and an oral part, on structural knowledge and experience. Registration is effective for 5 years only. No such qualification system exists in Canada.

Construction practices in Japan

Small housing and commercial buildings

Single dwelling units in Japan are predominantly built of wood. The traditional post and beam construction method has been the most popular, although North American platform construction has increasingly been used in the past two decades. Japan has a strong and long-standing tradition in steel construction, however, and it is no coincidence that this construction method has found its way also into the residential market. Besides multistorey commercial buildings, light steel framing is often used in residential housing of two or three storeys (Fig. 6). Typically, these houses are infilled with traditional wood panels. Lateral resistance is primarily provided by a moment resisting steel framing system and bracing is incorporated only in rare cases.

Fig. 6. Typical light-gauge steel framed residential building.

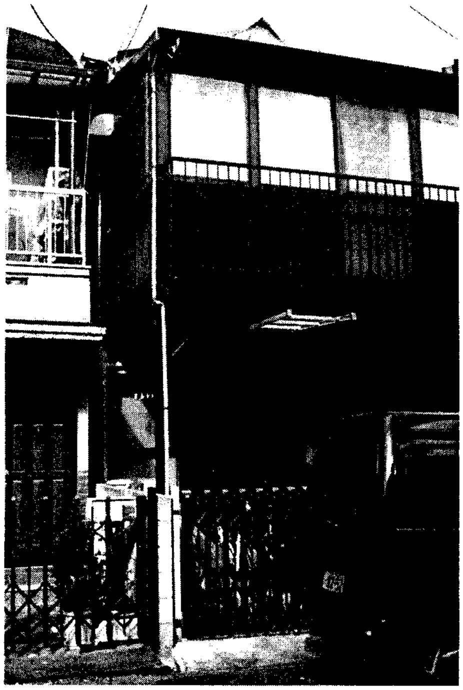


Fig. 7. Typical slender multistorey residential building with large front opening at first floor (cladding loss due to large interstorey drift).

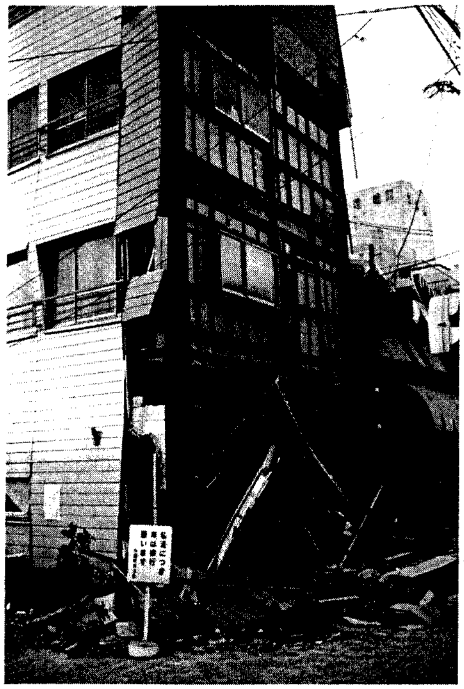
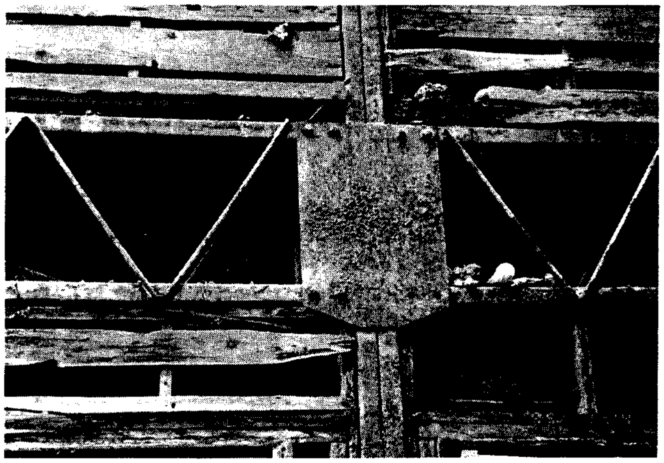


Fig. 8. Latticed beams in residential buildings.



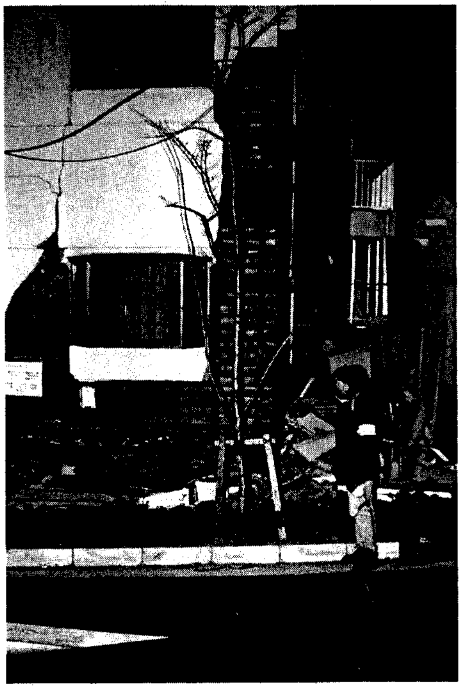
Steel frames are also common for apartment buildings of up to about eight storeys. These buildings are generally very slender, especially in the direction parallel to the street, with the facade being generally less than 4 m wide. Their construction is similar to single dwelling buildings, namely light steel frames with wooden infill walls. While some of the surveyed buildings included diagonal tension-only tie rod bracing members, these structures typically rely on the moment resisting beam–column connections to resist earthquake loads. Many such multistorey single dwelling houses or apartment buildings have a mixed occupancy with a small business at the first floor. As a result, these buildings frequently exhibit large storefront openings framed with portal structures (Fig. 7).

For these buildings, a typical Japanese type of construction was developed in the early 1960s in answer to shortage (and high cost) of structural steel in Japan. This type of construction, abandoned approximately 20 to 25 years ago, consisted of light-gauge cold-formed steel sections for columns and light trusses for beams (Fig. 8). Structural shapes of limited axial capacity were available at that time and sections were bundled together as necessary (Fig. 9). Often, latticed columns would also be used (Fig. 10) or box columns made by welding a pair of steel plates to the outer edges of the flanges of wide-flange cross section to create a tubular section. Steel frames including light-gauge H-shaped beams were also used, as in the building shown in Fig. 7. Interestingly, all beam–column connections in such structures are shop-built with a bolted splice in the beams near the zero moment location. This tree type of construction practice has become the standard procedure for moment resisting frames in Japan.

In more recent structures, wide-flange beams and columns are widely used and cold-formed hollow steel sections (HSS) have become very popular for columns subsequent to the 1981 code changes. Moment resisting frames are typically used in both directions and include shop-welded beam-to-column assemblies. Note that, as a result of the typically Japanese tree-type construction procedure, horizontal diaphragms are necessary when tube columns are used. Three types of diaphragm details are commonly used, namely: through-diaphragm type, interior diaphragm type, and exterior diaphragm. These are conceptually illustrated in Fig. 11. In the more common through-diaphragm connection, a long square tube is cut into three pieces: one used for the column of the lower storey, one for the connection's panel zone (a short piece, often called a dice in Japan), and

Fig. 9. Stacked columns in multistorey residential buildings.

Fig. 10. Latticed beams and columns in three-storey residential building with wood lattice infill (note collapsed first storey).



one for the column of the upper storey. Two diaphragm plates are inserted between the three separated pieces and shop-welded all around. The interior diaphragm and exterior diaphragm types are self-explanatory from Fig. 11. Similar diaphragm construction has also been observed in structures with wide-flange columns (Fig. 12).

Mass produced prefabricated light steel housing structures are also very common in Japan. In these structures, metal wall panels are joined together on site with patented connections after which thin rod tension-only panel braces, typically in every bay, are installed and pretensioned with turnbuckles (Fig. 13). The braces in these structures are connected with button-hole-type latches.

Other steel buildings

Steel is also widely used in other types of construction in Japan. For instance, 53%, by floor area, of the buildings erected in the Hyogo Prefecture (greater Kobe area) in 1993 were steel structures, whereas 35% and 12% were made of concrete and concrete-encased steel, respectively (Araigumi Co. Ltd., Private communication). Steel framing is typically used for resisting both the gravity and lateral loads. Hence, it is very unusual to have a gravity steel frame being braced by concrete shear walls, as is common practice in Canada. Lateral load resisting systems mainly consist of moment resisting frames, concentrically braced frames, or a combination thereof. Eccentrically braced frames have been introduced only very recently and are seldom used. No such frame was observed during the Canadian reconnaissance visit.

Moment resisting frames are widely used in Japan for multistorey commercial and office buildings. As in small residential buildings, shop-fabricated beam–column moment connections with bolted splices in beams, 1 or 2 m away from the columns, are used for these structures. H columns and beams are most often used in taller frames (Fig. 14) while tubular columns are still favoured in smaller structures. Moment connections to wide-flange columns include complete penetration beam flange welds, while web connections consist of bolted shear tabs or direct welding of the beam web to the column. In many structures, the beams are made continuous through the joints and the columns are welded to the bottom and top flanges of the beams. Whether it is the beams or the columns that are continuous at the joint depends upon the size of the connected members. In either case, the beam stub at the column is generally the same as, or heavier than, the beam itself.

X-bracing and chevron bracing are the prevalent concentrically braced frame configurations (Figs. 15*a* and 15*b*, respectively). These are used extensively for elevated parking structures which, incidentally, are made predominantly of steel in Japan. Tension-only bracing members are most often used in older X-bracings with the braces being made from slender steel rods or plates. Though such system is still in use for small buildings, newer structures are typically heavily braced with the bracing members being provided in a remarkably redundant multiple bay arrangement as shown in Fig. 15. Beams and columns are typically moment connected and the structures are very well detailed (Fig. 15*c*).

Small steel-framed parking facilities are also used throughout the city of Kobe. Some are very slender buildings, typically not more than 6 m × 6 m in plan and up to 10 storeys in height (Fig. 16*a*), with concentrically braced frames (chevron or X-bracing). Another type of small parking facilities includes two-storey structures with grating floors (Fig. 16*b*). These structures are concentrically braced in one direction while moment resisting frames are used in the other direction.

In general, steel buildings in Kobe are remarkably uniform in shape, with the lateral load resisting system being particularly well distributed in the frame in both directions. While this situation can be partly attributed to the severe code penalties in the case of modern structures, this characteristic undoubtedly reflects the alertness of Japanese engineers and architects to potential adverse consequences of in-plane torsion and structural irregularities.

Fig. 11. Diaphragm for HSS columns in moment resisting frames: (*a*) through-diaphragm; (*b*) interior diaphragm; (*c*) exterior diaphragm.

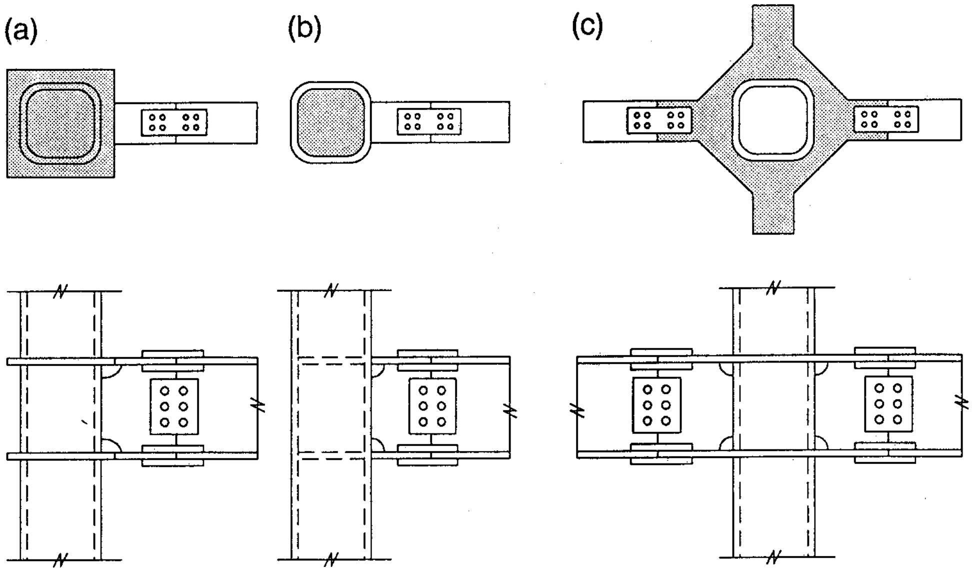


Fig. 12. Diaphragm for H-shaped columns in a low-rise moment resisting frame under construction.

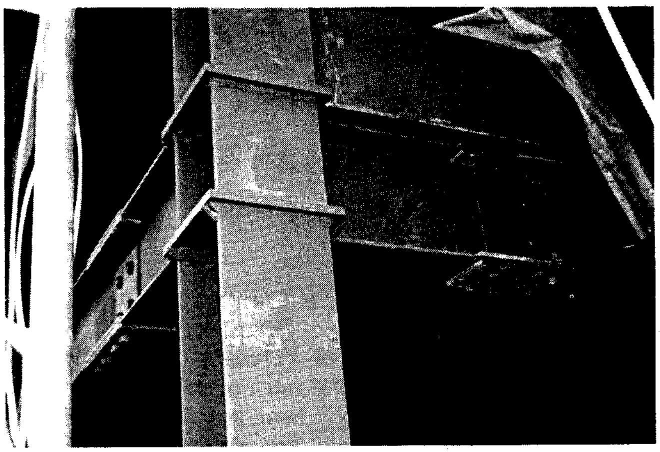
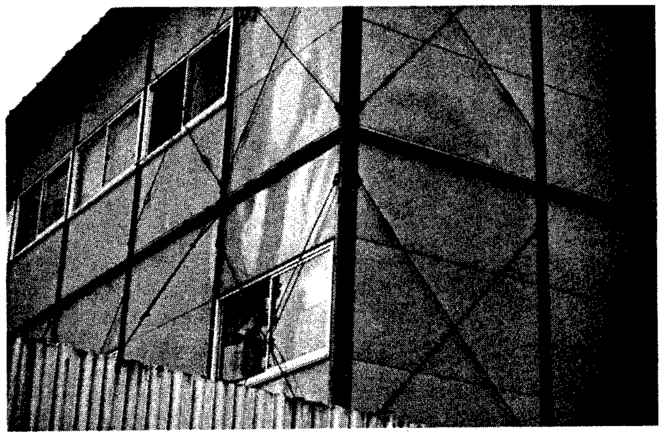


Fig. 13. Mass produced prefabricated steel housing unit with metal panels and light tension bracing.

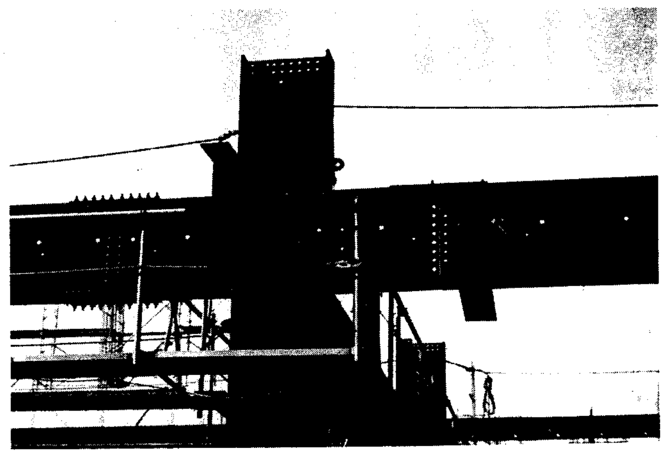


Performance of steel buildings during the Hyogo-ken Nanbu earthquake

This report on the performance of steel building structures during the Hyogo-ken Nanbu earthquake is based on the observations made during the site visit of the Canadian Association for Earthquake Engineering reconnaissance team (Tremblay et al. 1995*a*) and the findings of subsequent investigations performed by the second author, who stayed in Japan for 5 months after the event, and the third author who was actively involved in the survey of damage to steel buildings carried out by the Architectural Institute of Japan.

In many instances in this paper (particularly for the photos obtained with permission from the Architectural Institute of Japan), special care has been taken to avoid showing recognizable features or global exterior views which would make identification of the buildings possible, since many owners are contemplating initiating litigation proceedings against designers.

Fig. 14. Typical shop-built beam-to-column moment connection in a multistorey structure under construction.



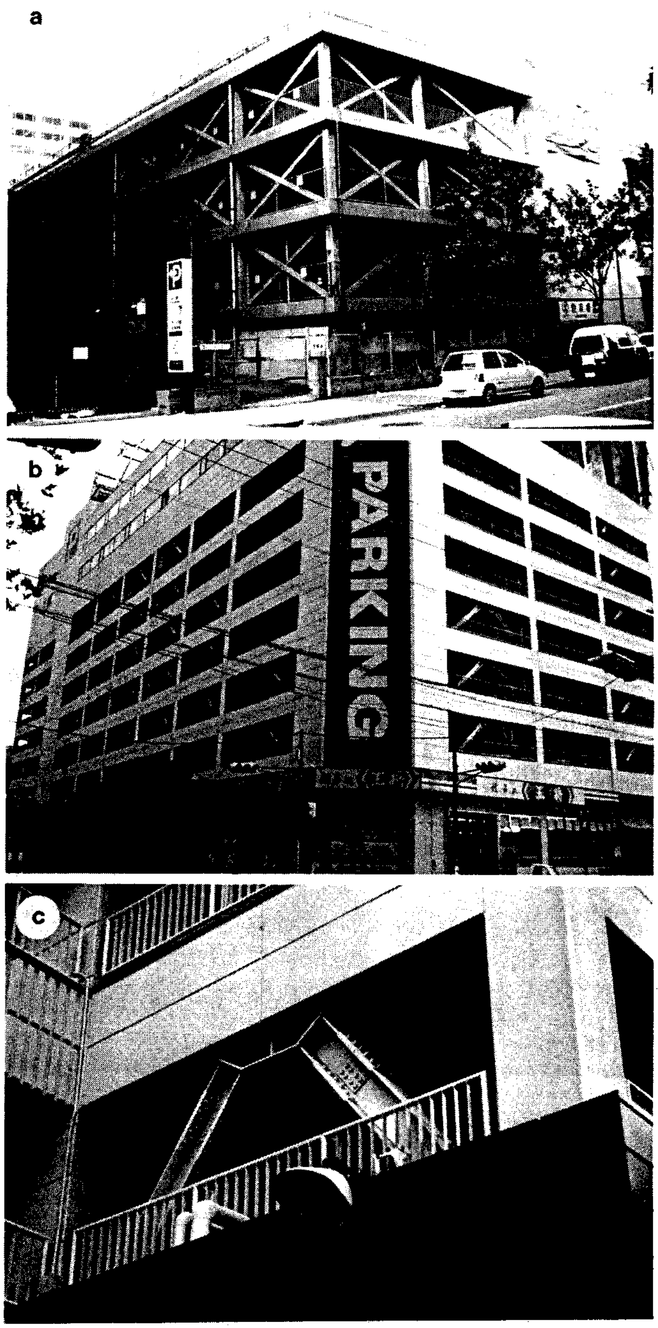


Fig. 15. Typical concentrically braced frame in parking structures: (*a*) X-bracing; (*b*) modern chevron bracing; (*c*) brace connection in the chevron bracing.

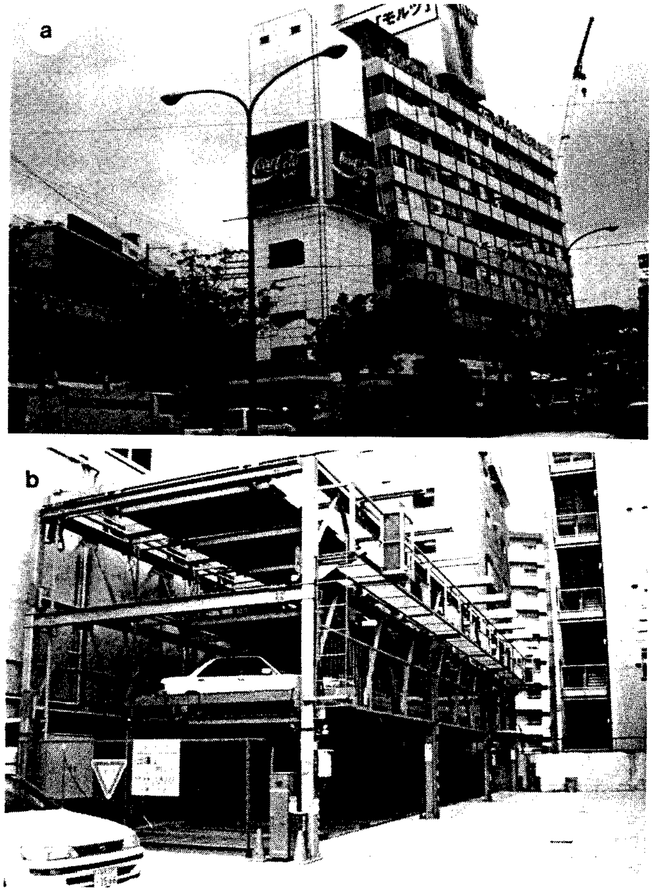


Fig. 16. Small parking facilities: (*a*) slender multistorey structure (to the left of the building with the collapsed 5th storey); (*b*) light two-storey parking structure.

Architectural Institute of Japan survey of steel buildings

The Hyogo-ken Nanbu earthquake is the first event during which so many steel frames have been damaged. As mentioned earlier, reports from the first investigations that followed the earthquake (e.g., Koller et al. 1995; Park et al. 1995; Tremblay et al. 1995*a*) concluded that most of the damage had taken place in older structures, either nonengineered or designed according to less rigorous seismic design provisions.

Recognizing that damage to steel buildings due to earthquakes can be hidden, a special task force of 28 engineers from the Architectural Institute of Japan spent nearly 1 month, starting from the beginning of February 1995, to meticulously survey the damage to 988 modern steel buildings in an area extending from the western boundary of the city of Kobe to the eastern limit of the Hyogo Prefecture (AIJ 1995*d*). Old steel buildings built with light-gauge columns and beams were omitted from that survey, as these structures were frequently in a severely deteriorated condition and constitute an archaic construction type unlikely to be constructed in the future (see next section). Prefabricated light steel housing units were not surveyed either. Nevertheless, at the time of writing, this survey constituted the most comprehensive set of statistics on the performance of steel buildings during this earthquake.

Of the 988 steel buildings examined, 90 were rated as collapsed, 332 as severely damaged, 266 as moderately damaged, and 300 as slightly damaged. The ratios of these numbers are approximately in the proportion 1:3:3:3. The Architectural Institute of Japan damage definitions are as follows (AIJ 1995*d*):

• Minor damage: No damage to major vertical force supporting members like columns and beams. Minor cracking and spalling of exterior finishes and (or) buckling of rod or flat bar braces. No or nearly no permanent lateral deformations.

Table 4. Damage level as a function of structural type and column cross section (translated and adapted from AIJ 1995*d*).

Structural type*	Column cross section†	Damage rating: Collapse	Severe	Moderate	Minor	Total damaged
A		25	53	44	41	163
	H	20	74	40	26	160
	|H|	4	16	17	10	47
B		0	5	4	1	10
	H	7	52	34	15	108
	|H|	0	4	1	0	5
C		0	3	4	2	9
	H	3	8	4	1	16
	|H|	0	0	1	0	1
Unknown	—	31	117	117	204	469
Total	—	90	332	266	300	988

*A, moment resisting frames in two dimensions; B, moment resisting frame in one direction and braced frame in the other direction; C, braced frame in two directions.

†Blank entry, square tube (cold-formed); H, wide-flange; |H|, box-section constructed with wide-flange and cover plates.

Table 5. Damage to various structural elements as a function of structural type and column cross section (translated and adapted from AIJ 1995*d*).

Structural type*	Column cross section†	Damage rating: Columns	Beams	Beam-to-column connections	Bracing members	Column bases
A		46	31	71	—	46
	H	77	16	30	—	43
	|H|	23	2	7	—	14
B		3	2	3	6	4
	H	43	8	13	66	34
	|H|	5	1	1	2	0
C		3	4	3	5	1
	H	5	0	3	12	6
	|H|	1	0	0	1	0
Unknown	—	88	17	32	9	59
Total	—	294	81	163	101	207

*A, moment resisting frames in two directions; B, moment resisting frame in one direction and braced frame in the other direction; C, braced frame in two directions.

†Blank, square tube (cold-formed); H, wide-flange; |H|, box-section constructed with wide-flange and cover plates.

- Moderate damage: Buckling and rupture of bracing members and yielding of columns and beams. Small residual lateral deformations, approximately smaller than 1/100. Any damage not categorized as minor, severe, or collapse.
- Severe damage: Serious damage to columns, beams, and connections to an extent considered difficult to repair. Significant residual lateral deformations, more than 1/100.
- Collapse: Collapse of a storey or of the entire building.

About 71% of buildings whose structural type could be identified were unbraced in both directions (R-R), 24% had braces in only one direction, and the rest were braced in both directions. In 469 instances, the structural system was rated as "unknown," either because it was difficult to identify the structural type, sometimes as a result of the entire collapse of the building, or because the building was completely covered by its architectural finishes.

Table 4 shows the correlation between structural type and damage level for the three types of columns which have typically been used in Japan: square tube (almost exclusively cold-formed, which has been the common practice in Japan

Fig. 17. Collapsed light-gauge steel framing: (*a*) first-storey collapse; (*b*) failure of the second-storey columns; (*c*) local buckling in a failed column.

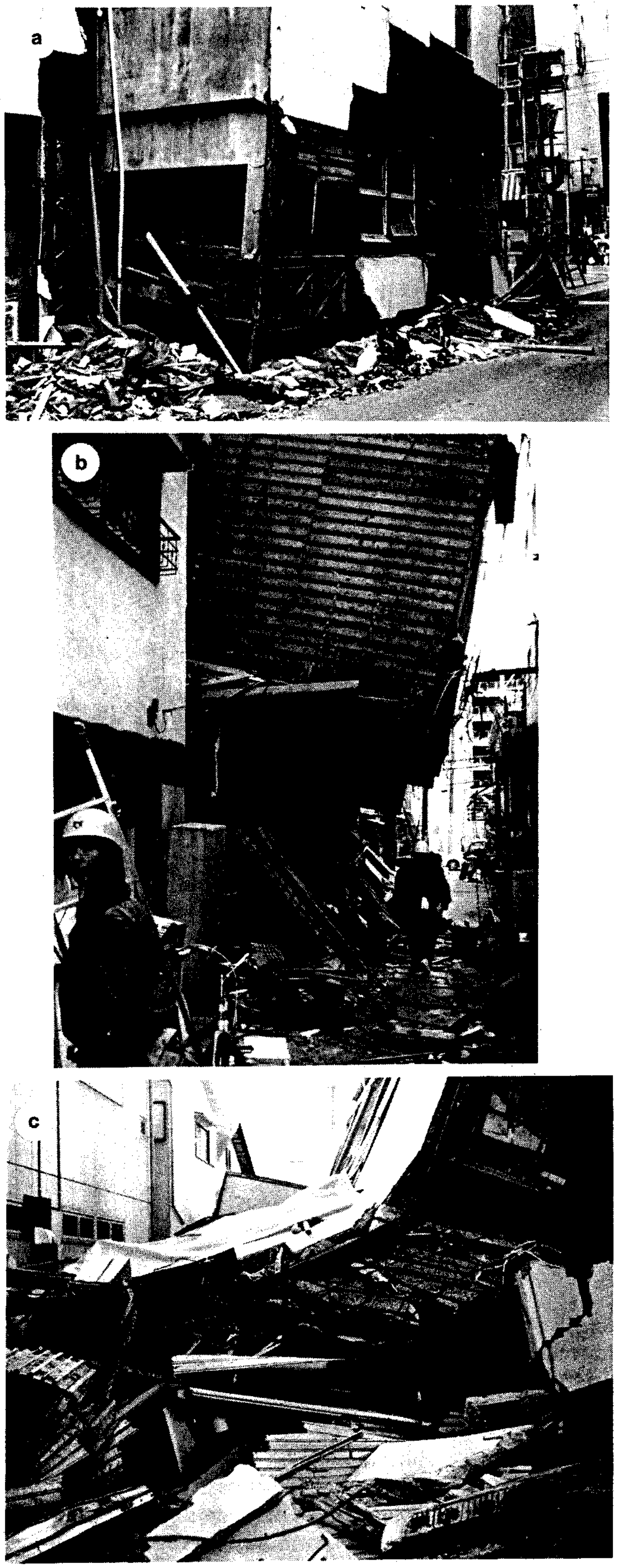
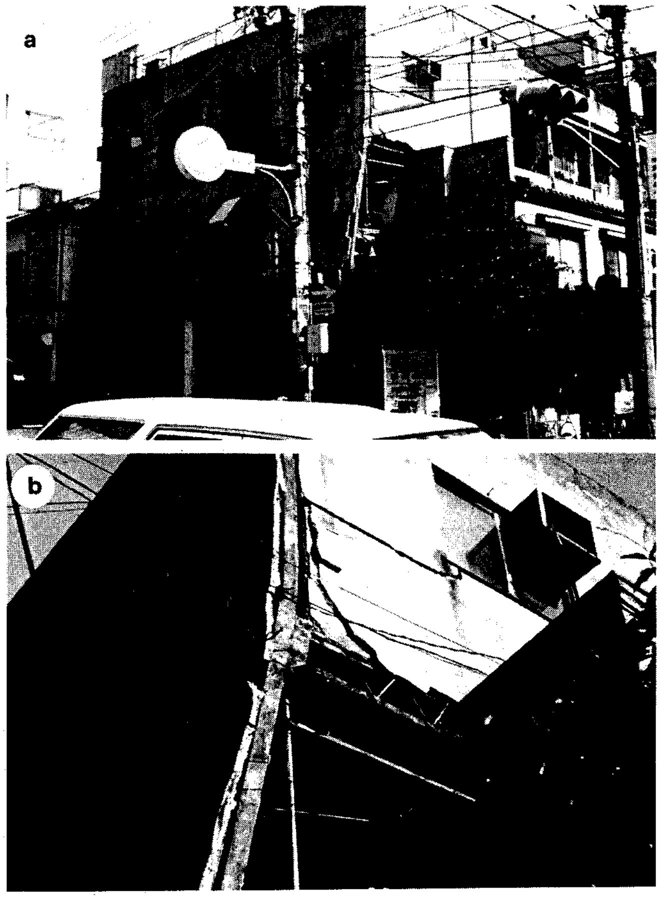


Fig. 18. Tension failure of the bottom chord of a moment-connected floor truss in a three-storey residential building: (*a*) overall view of the building; (*b*) close-up of the failed connection.



for the past 15 years), wide-flange (H), and wide-flange covered with exterior plates (those built prior to the introduction of cold-formed square tubes in Japan). It is observed from that data that no significant difference exists in damage level with respect to the structural type, and that buildings with wide-flange columns have suffered more serious damage.

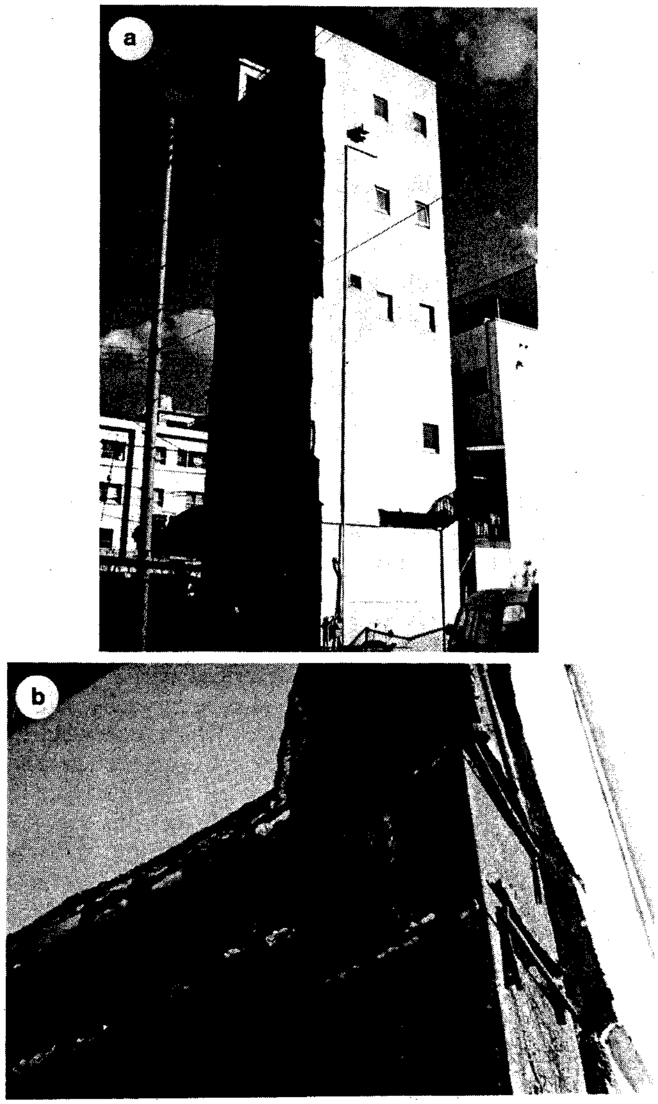
Table 5 shows the location of damage as a function of structural type. In unbraced frames, columns suffered the most damage (in terms of the number of buildings), while in braced frames, braces were naturally the most frequently damaged structural elements.

Old light-gauge structures

Old buildings made of light-gauge steel sections had been designed and constructed without much regard for seismic considerations, which resulted in any combination of low resistance to lateral loads, nonductile members, and inappropriate strength hierarchy. Hence, not surprisingly, these structures suffered a considerable amount of damage during the Hyogo-ken Nanbu earthquake.

Frequently, columns were the weakest elements at beam-to-column joints and yielded prematurely. The ductility of most of these buildings was severely limited, as the cold-formed steel sections used for columns would typically develop local buckling prior to attainment of their plastic

Fig. 19. Peeling off of the cover plate of a built-up box column (at the beam top flange connection) in a five-storey residential building: (*a*) overall view of the building; (*b*) close-up of the first-storey beam-column connection.



moment capacity. Ductile column hinging was not possible and the structure was destined for a brittle collapse unless some other ductile mechanism could develop prior to the column's failure. To aggravate matters, many of these buildings had severely corroded columns as a result of years of water infiltration. Yet, while numerous such buildings have collapsed (Figs. 10 and 17), many others have survived (Fig. 7), although they most frequently shed their veneer finishes in the process. The survivors likely owe their sometimes surprising seismic performance to a combination of nonrigid framing action and interaction with their internal wood-frame partitions.

These old steel frames also experienced other types of structural damage, including tension failure of braces and fracture of beam-to-column connections. For the latter, various failure patterns could be readily observed: tension fracture of truss bottom chords (Fig. 18), peeling-off of cover

Fig. 20. Failure of the beam-to-column welded connection at the first storey of the building shown in Fig. 7.

plates in built-up box columns (Fig. 19), or failure of beam-to-column welds (Fig. 20).

Prefabricated housing structures

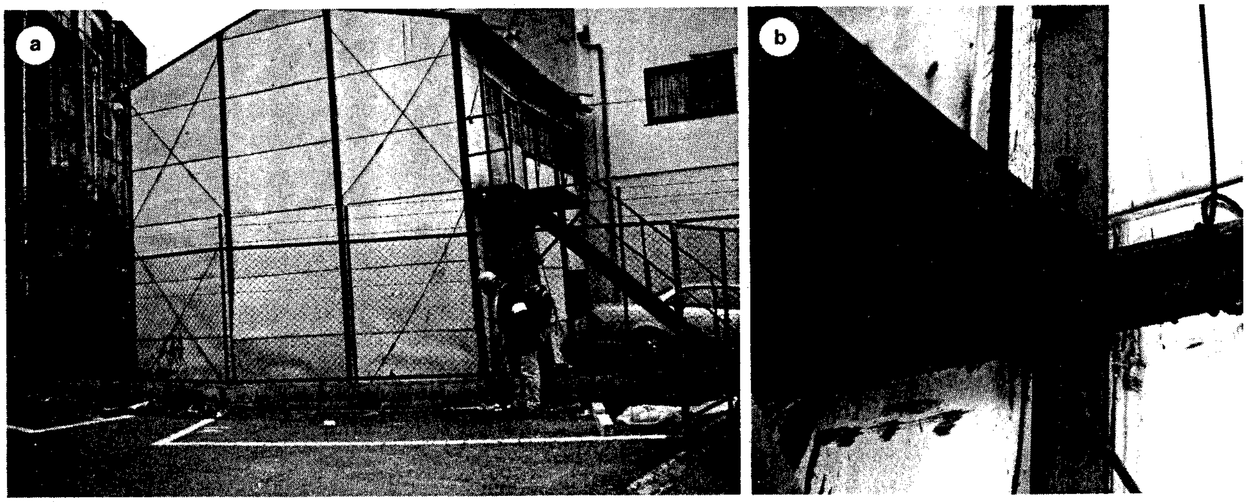
Extensive yielding of bracing members took place in mass produced prefabricated steel housing structures (Fig. 21*a*). In some cases, the connection latches unbuttoned when the braces yielded in tension and became loose (Fig. 21*b*). In most cases, an adequate number of braces remained, however, to keep the structures from collapsing.

Moment resisting frames

An extremely large number of older and modern moment resisting frames suffered dramatic damage. Failure occurred at a number of different locations. As was the case following the Northridge earthquake, a disconcerting number of beam-to-column connection fractures were eventually discovered following the earthquake. The Architectural Institute of Japan survey (1995*d*) reported 113 buildings for which damage to beam-to-column connections was observed. This discovery is particularly upsetting to Japanese engineers, as the tree-type construction procedure described earlier, although more expensive than the North American practice, was mostly adopted based on the conviction that it permitted more reliable higher-quality shop-welded connections capable of developing the plastic moment of the connected members.

The failures observed during this earthquake differed somewhat from the Northridge failures, however, in that cracking and fracture was frequently (but not always) accompanied by yielding of beams. This evidence of inelastic response was mostly observed in the more modern unbraced frames having square-tube columns and full penetration welds at the beams. In the majority of these cases, no sign of yielding was observed in the adjoining columns. Most of the fractures occurred in the lower flange of the beams, and the beams exhibited clear signs of yielding as well as local buckling in many cases (Figs. 22*a*, 22*b*, and 22*c*), although, in some cases, the level of visible yielding was modest. Typically, fracture initiated either from the corner of a weld access hole, near a run-off tab or a weld toe, or in the heat-

Fig. 21. Damage to prefabricated housing units with tension bracing: (*a*) braces yielded and some fell-off; (*b*) brace connection detail (note top bracing missing).



affected zones on the beam flange or diaphragm. In many cases, the fracture progressed into the beam's web (e.g., Fig. 22*b*), and even, in some cases, progressed or initiated into the column flanges (e.g., Fig. 22*d*). Most of the buildings that sustained such cracking and fracture did not exhibit large permanent deformations or significant damage to their exterior and interior finishes, and repairs were expediently accomplished by re-welding through the cracks.

When fillet welds were used in moment resisting frames, many beam-to-column connections cracked and fractured without any sign of yielding, as these welds were apparently too small to develop the capacity of the connected members. Fracture of beam-to-column welds, such as the one shown in Fig. 20, was typical and many of the buildings having such connections suffered serious unrepairable damage. It is noteworthy that when tube columns were used, cracking and fracture also frequently occurred in the columns above or below the top or bottom diaphragm (Fig. 23). In Fig. 23*a*, a 6 mm fillet weld was used to connect a 305 × 305 × 16 mm tube column to the diaphragm plate, which is an unequivocal example of undersized welds. As a result, complete overturning and collapse of the structure frequently occurred (Figs. 24*a*–24*c*). In fact, damage to the beam-to-column connections of at least 59 unbraced frames having square-tube columns was reported by the Architectural Institute of Japan, with about 70% of those rated as either collapsed or severely damaged. It is worth noting that capacity design is not mandatory in the 1981 Japanese specifications for beam-to-column connections in short-span buildings not more than three storeys high for which level-two design is omitted (Fig. 2), which could explain some of the reported failures. While most of these surveyed buildings that collapsed had fillet welds, at least three collapsed in spite of having full penetration welds (AIJ 1995*d*).

In addition to the above, miscellaneous other forms of damage too numerous to include in this paper were also sporadically observed in individual buildings, such as local buckling of beams and columns. In some instances, inadequate beam-to-column connection detailing also led to failure of moment resisting frames as shown in Fig. 25 for a small two-storey parking structure.

Concentrically braced frames

The Architectural Institute of Japan survey of steel buildings reported that rods, angles, flat bars, round tubes, wide flanges, square tubes, and channels were used as braces in 77, 44, 44, 42, 8, 6, and 4 cases, respectively (the actual type of bracing could not be identified in 227 other steel buildings surveyed, for the same reasons described earlier). In the buildings surveyed, braces were found to be generally bolted, except for small rod and flat bar braces, which were welded. Table 6 shows the correlation between the cross section used for braces and the damage level. Damage was clearly more severe in the smaller cross sections. Although this was not quantified, the size of brace cross sections is also strongly correlated with the age of buildings, with the small cross sections used more frequently in older buildings.

Slender plate braces, which have generally been used in older buildings, experienced inelastic stretching and developed significant slackness during the earthquake (Fig. 26). Nonductile fracture at bolt holes was typical, as many connections were not capable of sustaining the brace yield load, amplified by the impact forces that likely developed upon straightening of the braces (Figs. 27 and 28). Brace fracture combined with excessive slenderness prevented effective hysteretic energy dissipation in these structures. It is noteworthy, however, that plate bracing is still favoured and used for some new construction, with the difference that careful attention is now paid to proper ductile detailing. One such example was discovered in a building still in construction at the time of the Hyogo-ken Nanbu earthquake. While this building exhibited no visible damage from the outside, a more thorough inspection revealed extensive buckling of the main braces (Fig. 29). No brittle failures nor visible residual lateral building drifts were evident. Although the braces were T-shapes rather than purely plate braces, the stem of the Ts was very short.

Compression–tension acting bracing systems also suffered extensive structural and nonstructural damage. Figure 30*a* shows the upper floors of a seven-storey braced frame in downtown Kobe. The frame sustained severe inelastic deformations as indicated by the heavily buckled shape of the bracing members, and it exhibited many of the typical failure

Fig. 22. Damage to beam-to-column connections in modern moment resisting frames with square-tube columns and full penetration welds at the beams: (*a*) fracture at the lower beam flange; (*b*) propagation of fracture in the beam web; (*c*) fracture initiated in the heat-affected zone of the diaphragm; (*d*) propagation of fracture in the column. (Courtesy of the Architectural Institute of Japan.)

modes observed for these structures: low-cycle fatigue fracture of braces at plastic hinge location (Fig. 30*b*), fracture of bolted brace-to-gusset connections at net area (Fig. 30*c*), and failure of the welded connections between some gusset plates and the frame (Fig. 30*d*).

Interestingly, measurements taken on the H-shaped braces at the third floor revealed that, assuming that 235 MPa steel material was used, braces would meet the S16.1 width-to-thickness and slenderness ratio limitations for the ductile braced frame category. Thus, the observed damage to the braces would be a good indicator of the ability of bracing members of ductile braced frames built in Canada to dissipate energy and survive under similar earthquake ground motions. In this case, however, the calculations also showed that the brace connections could not develop the full yield load of the bracing members. The severe in-plane deformation pattern exhibited by the bracing members suggests that brace connections also experienced significant bending moments, likely close to the flexural capacity of the braces, during the earthquake. Also noteworthy for this building is the fact that moment connected trussed beams were also used in the structure. These elements showed evidence of severe yielding (Fig. 30*d*), which clearly indicates that they significantly participated in resisting lateral loads subsequent to brace buckling and elongation.

In a four-storey chevron braced frame parking structure (Fig. 31*a*), brace connecting plates fractured in tension at the gusset plate connections. After the earthquake, emergency measures were taken to secure the brace connections (Fig. 31*b*) and replacement of the bracing members was under way at the time of the visit (Fig. 31*c*). As shown in Fig. 31*b*, out-of-plane buckling of gusset plates also occurred in this structure. This phenomenon could be observed in many other braced frames; and in several occasions, buckling (bending) of brace gusset plates or brace connecting plates (Fig. 32) was the only apparent means of hysteretic

Fig. 23. Failures of undersized fillet welds between tube column and diaphragms.

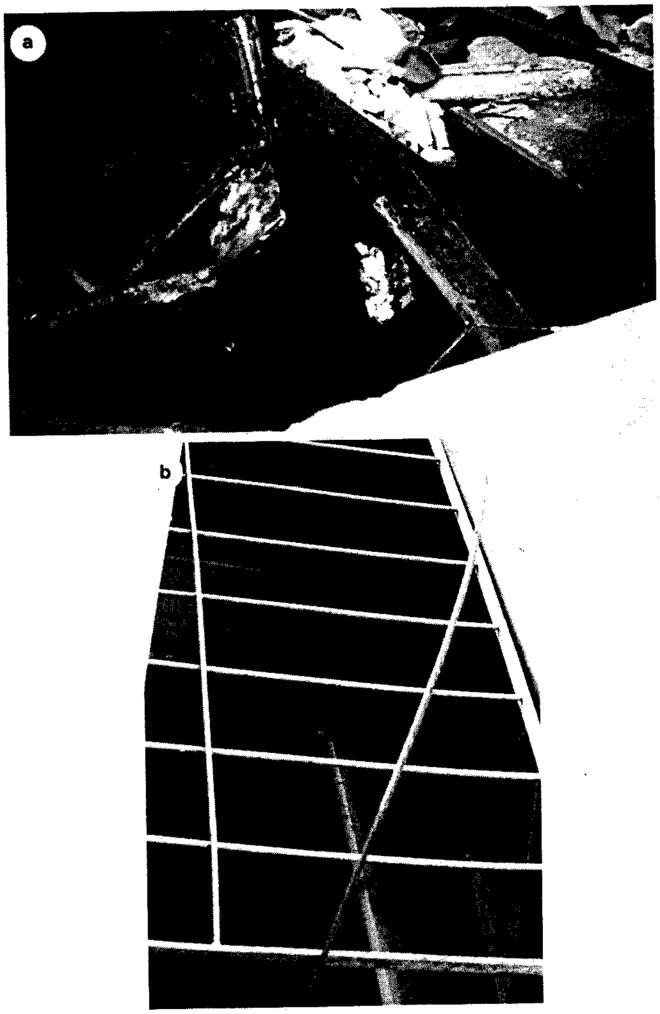
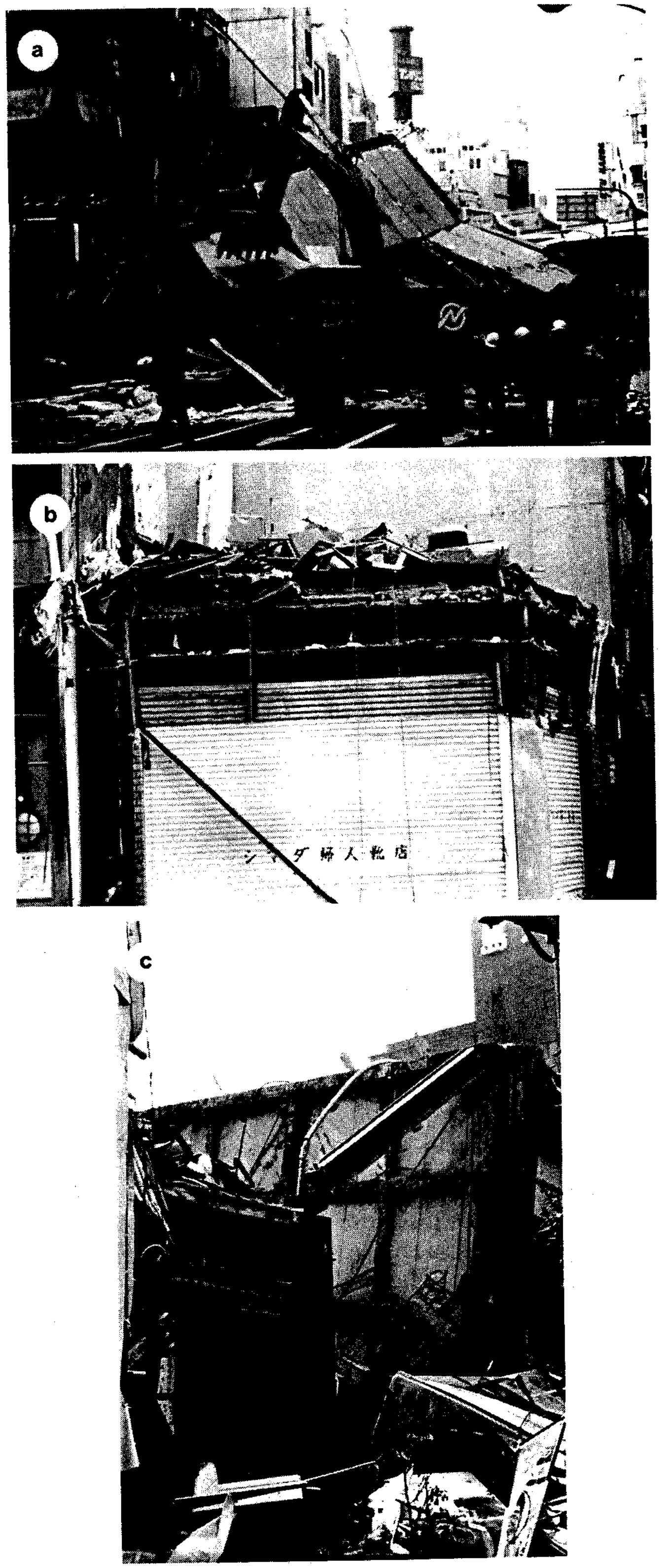


Fig. 24. Collapsed moment resisting frames with fillet welds at beam-to-column connections.



energy dissipation in the structure.

Various other structural failure modes could be observed in another nearby parking structure. In this five-storey structure, square tubing braces were arranged in a split-X bracing configuration (Fig. 33*a*) and brace connecting plates were bolted to angles welded to columns and beams. Some braces failed in tension (Fig. 33*b*) while others experienced local buckling near their ends (Fig. 33*c*). Brace connections also suffered extensive damage as bolts sheared off (Fig. 33*d*) and welds fractured (Fig. 33*e*). Beams were also seriously distorted at the brace connections located at the middle of the Xs as a result of the cyclic reversals of tension and compression brace loads (Fig. 33*f*).

Another type of damage observed in numerous tension–compression braced frames in the Kobe area is the shedding of wall finish, resulting from the out-of-plane buckling of bracing members (Fig. 34). Damage also took place in columns of braced frames. For instance, notable local buckling and brittle fracture of a column at the base of a braced frame is shown in Fig. 35. This failure occurred below the connection of the first brace. Hence, at that location, the column was subjected to severe axial, shear, and flexural stresses, which contributed to precipitate this failure.

While some braced frames collapsed or were on the verge of collapse, many others that suffered a significant amount of damage survived, as shown by the numerous examples in

Fig. 25. Collapsed parking structure: (*a*) overall view; (*b*) close-up of failure of the unstiffened tube wall at beam-to-column connection.

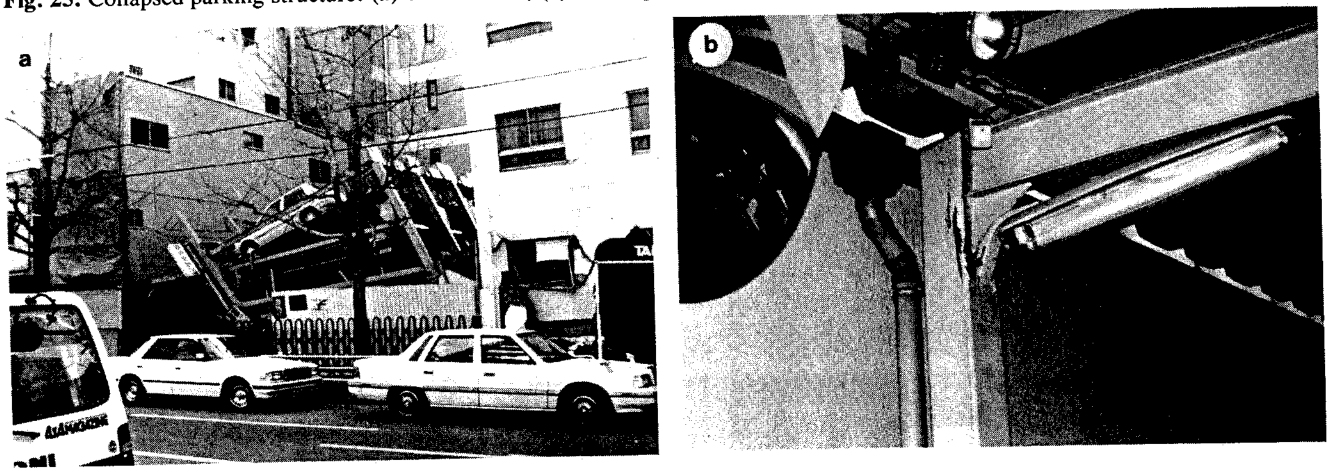


Table 6. Severity of damage to braces in braced frames as a function of type of brace cross section (translated and adapted from AIJ 1995*d*).

	Damage rating				
Bracing type	Collapse	Severe	Moderate	Minor	Total damaged
Rods	9	37	20	11	77
Angles	4	18	19	3	44
Flat bars	1	25	13	5	44
Subtotal (smaller braces)	14	80	52	19	165
Round tubes	0	7	22	13	42
Wide-flanges	0	5	3	0	8
Square tubes	0	0	4	2	6
Channels	0	2	2	0	4
Subtotal (larger braces)	0	14	31	15	60
Unknown	15	47	51	114	227
Total	29	141	134	148	452

this section. This can be partly attributed to the large redundancy designed into these frames by Japanese engineers, as all beam connections are typically made fully rigid in these structures (in addition to the bracing), and to the much larger number of bays that are typically braced in Japanese buildings than in comparable North American buildings.

In general, more recent and heavily braced structures, as the one shown in Fig. 15*b*, performed remarkably well during the earthquake. Proper connection detailing together with well-proportioned bracing members, with very low slenderness ratio, permitted to achieve effective and reliable energy dissipation in the braces (Fig. 36).

Column base connections

In Japan, three types of column base connection are commonly used (Fig. 37): standard base plate connection, concrete-encased column base connection, and embedded column base connection. Interestingly, among the 218 buildings for which footing system could be identified by the Architectural Institute of Japan survey, 127 used standard column base plates. Out of these, 101 sustained serious permanent deformation or collapsed, mostly as a result of anchor bolt fracture (Fig. 38*a*) or anchor bolt pull-out (Fig. 38*b*). Concrete-encased column base connections also suffered damage in many instances (Fig. 38*c*).

Ashiyahama Residential Complex

Very few occurrences of damage have been reported so far in tall steel frames exceeding 60 m in height. The superior performance of these structures can be partly attributed to the fact that most, if not all, have been constructed recently and, hence, were designed according to the seismic provisions of the 1981 code, which demand higher resistance for taller frames.

The steel superstructure of several buildings of the Ashiyahama Seaside Town buildings experienced dramatic failures during the earthquake. This prestigious residential complex of 52 buildings with a total of some 3400 high-rise apartments (Fig. 39) is located on a 50 acre site of reclaimed land in the southern part of Ashiya city (east of Kobe). Buildings are 14, 19, 24, or 29 storeys and range from 41 to 85 m in height. While dimensions in plan slightly vary from one unit to another, the same layout and structural concept have been used for all buildings: stacks of four (two at the

Fig. 26. Elongated plate braces in a four-storey parking structure.

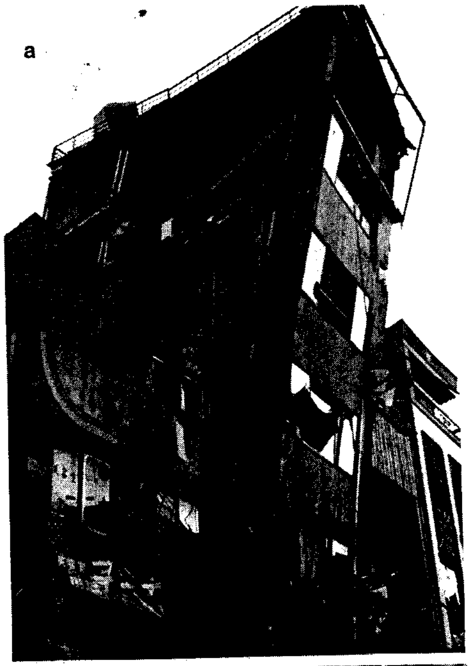
top of the structures) prefabricated concrete residential units are supported by a mega steel frame that includes trussed beams and columns. In each building, the first six storeys rest directly on the reinforced concrete foundation mat that supports the structure. The mat is founded on steel piles, typically 35 m long, and soil improvement has been performed below and around the buildings to mitigate soil liquefaction related problems (Hisatoku 1984). Resistance to lateral loads in the buildings is provided by means of the moment resisting action that develops in both directions between the horizontal and vertical steel trusses.

The construction of the complex, which was awarded the Architectural Institute of Japan Prize in 1980, spanned over many years, as some buildings nearest the seashore were noticed to be under construction at the time of the visit. The project is the result of a competition organized in 1973 by the Ministry of Construction of Japan to build a "new city" on that site. Out of the 22 entries, the winning team included one of the Japanese Big-Five Contractors as the designer/architect/engineer which, with a staff of approximately 6000 architects and engineers, has been frequently ranked as one of the top international contractors. Thus, the discovery of brittle fractures throughout the steel structures of that complex attracted considerable attention.

The fractures took place along the external trussed legs of the frames which include square heavy boxed steel columns and H-shaped web members. The Architectural Institute of Japan task force (AIJ 1995*d*) reported observing three types of fracture patterns in those mega-frames: (A) fracture in the base metal, 200–300 mm above a column splice (Fig. 40*a*); (B) fracture of the column at the welded column splices (Fig. 40*b*); and (C) fracture at the column to brace connection, with extension of the fracture into the brace (Fig. 40*c*). A fourth type of fracture (say, type D) was observed by the first two authors of this paper: fractures of the brace to column welds, without adjacent column fracture (Fig. 40*d*).

According to the Architectural Institute of Japan statistics, as well as by Nikkei Architecture (1995), among all the buildings of that complex, the tallest (29 storeys) buildings had no column fracture, and the shortest (14 storeys) buildings had only a few fractures. Type A failure was only observed at the first and second storeys (10 and 3 fractures respectively reported at those storeys), type B failure in the lower 13 storeys (37 fractures), and type C failure in the 2nd to 9th storeys (7 fractures). No statistics exist for the type D fractures observed by the authors. No fracture was observed in or above the 14th storey. Also, it is worthwhile to mention that although minor evidence of yielding was sometimes witnessed in columns (AIJ 1995*d*), column fracture was essentially brittle, exhibiting a fracture surface typical of brittle crack propagation, as the surfaces were rather rough, involving shear lips and tear ridges.

Fig. 27. Tensile connection failure of plate braces in a seven-storey residential building: (*a*) general overview; (*b*) close-up of brace fracture.



A considerable amount of public domain technical information is available for the 24- and 29-storey buildings of this complex (e.g., BCJ 1975), since they both exceeded the 60 m height limit that triggers the need for a design approval process, as described earlier. Technical information on the structures is also provided by Hisatoku (1984). Interesting aspects considered during design include the following:

(*i*) Limitation of effective brace slenderness ratio to less than 30.

Fig. 28. Tensile connection failure of plate braces in a four-storey car lift: (*a*) overview of the structure; (*b*) close-up of brace fracture.

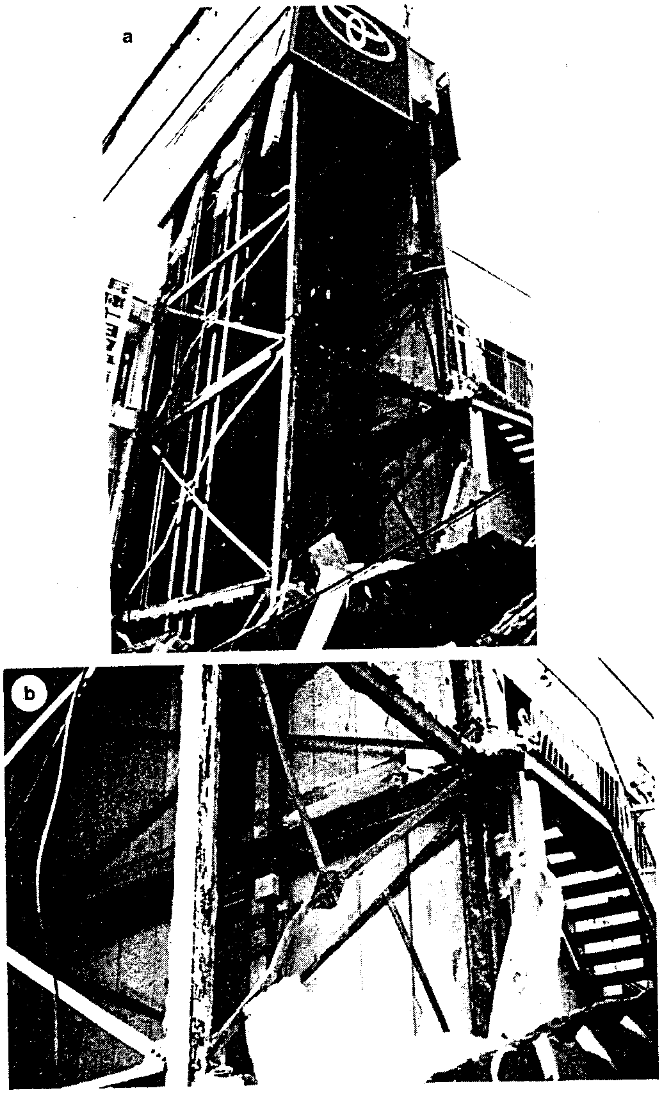
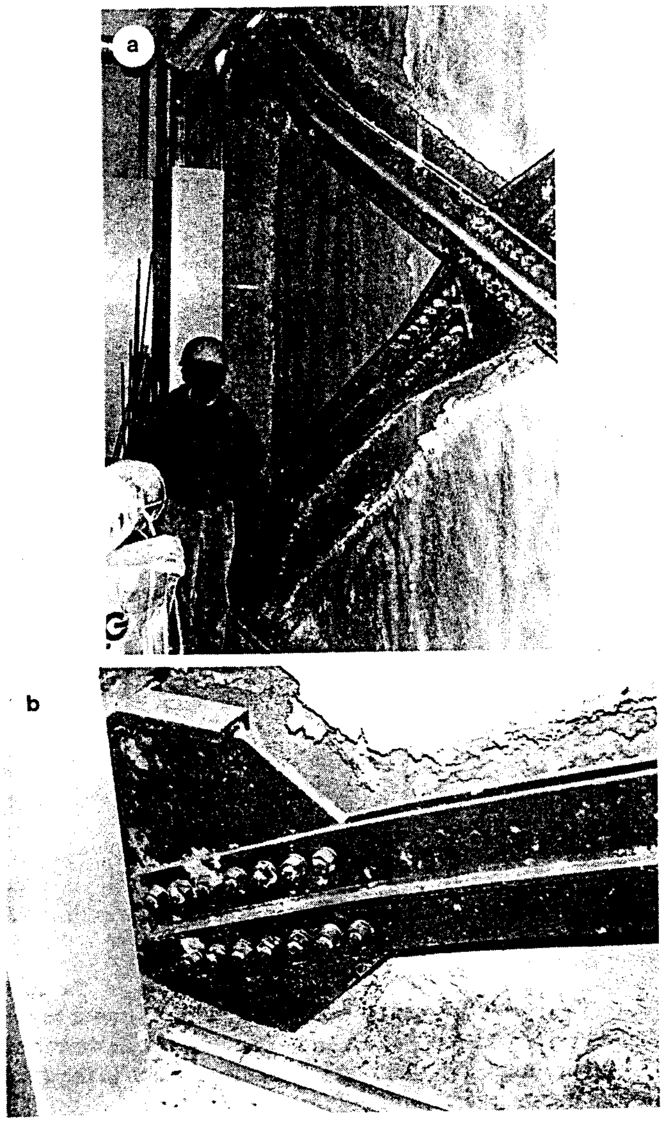


Fig. 29. Properly detailed slender tension-only bracing in a modern structure: (*a*) overview; (*b*) close-up of the end connection.



(*ii*) Braces designed not to buckle before stresses 50% larger than those corresponding to the design shear strength are reached.

(*iii*) Box-columns at the lower storeys were built using two thick rolled C-shapes of SM50 or SM53 steel (weldable steel having a yield strength of 325 MPa). For instance, at the first floor of the 19-storey buildings, columns were 500 mm deep and made of 55 mm thick material.

(*iv*) Structural fundamental periods of 1.4, 1.75, 2.3, and 2.6 s were calculated for the long direction of the 14-, 19-, 24-, and 29-storey buildings, respectively. The second mode corresponded to periods of 0.5, 0.6, 0.8, and 1.0 s, respectively. Slightly longer periods were obtained in the short direction.

(*v*) Base shear coefficients in the long direction equal to 0.24, 0.19, 0.15, and 0.125 were considered for the 14-, 19-, 24-, and 29-storey buildings, respectively.

(*vi*) Nonlinear time-history analyses were conducted using a quadrilinear hysteretic model of the superstructure and an elastic substructure model with stiffnesses determined from loading tests. Damping equal to 3% of critical damping was used for the superstructure, and two intensities of ground motion were considered: intensity V, having a return period of 50 years and peak ground accelerations of $0.2g$, and intensity VI occurring once every 100 years with peak ground accelerations of $0.3g$. Five different earthquake records were considered in the analysis: El Centro 1940 and Taft 1952 scaled to $0.2g$, Osaka 201 1963 scaled to $0.3g$, and two other artificial records called Kobe P.I.-B scaled to $0.2g$ and $0.3g$. Maximum interstorey drifts were limited to 1/150 and 1/100 at the $0.2g$ and $0.3g$ levels, respectively; and the maximum ductility factor was 1.33 at the $0.3g$ level.

Inelastic analyses conducted during the original design process predicted that braces would yield first, followed by beams, and finally columns (Nikkei Architecture 1995). In some of the damaged buildings, however, no evidence of inelastic deformation could be seen after the earthquake. It

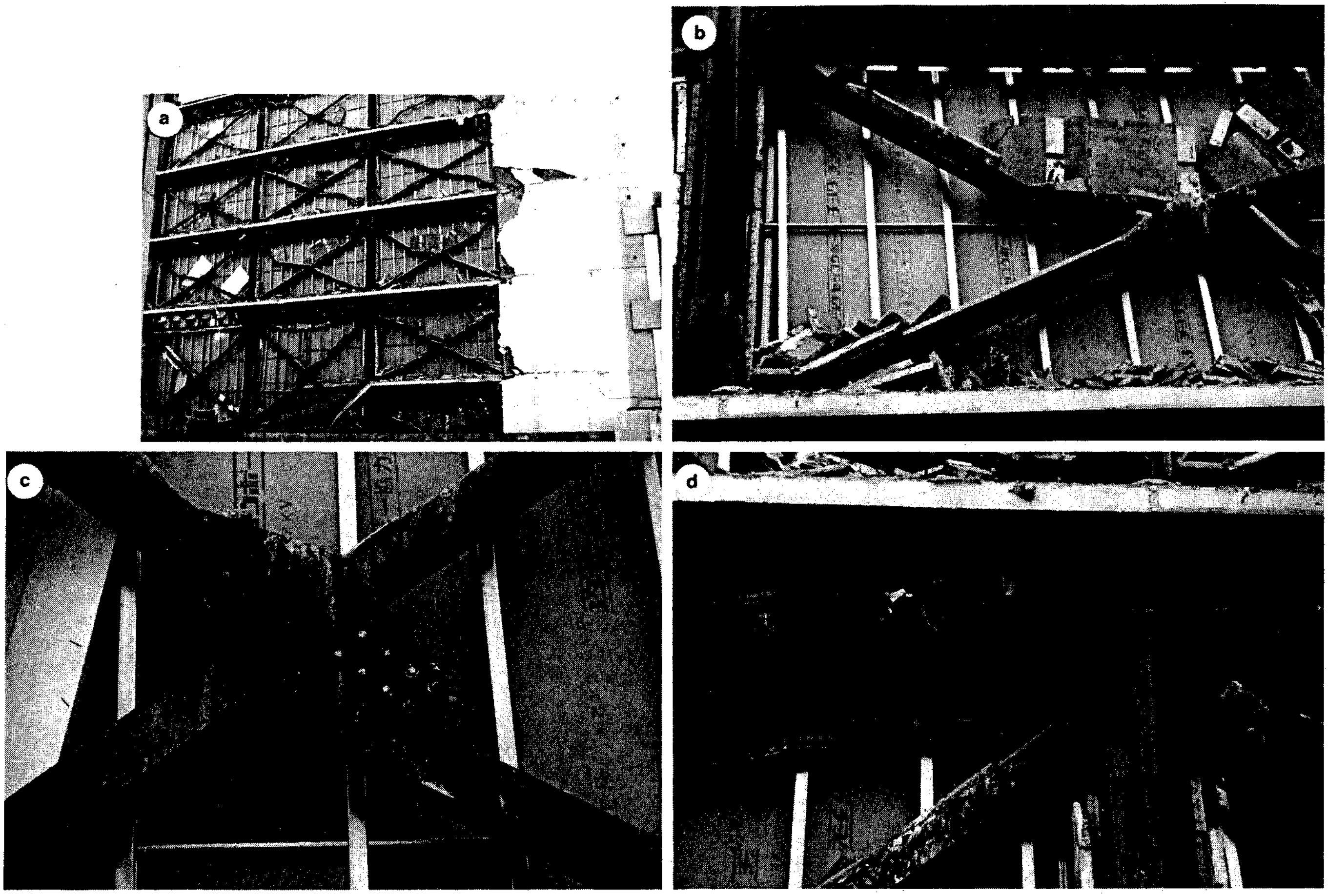


Fig. 30. Performance of a seven-storey braced frame: (*a*) overall view of a braced wall at the upper floors; (*b*) fracture of a bracing member at location of plastic hinging; (*c*) brace fracture at net area; (*d*) failure of a gusset plate to frame connection and damage to trussed beams.

is also noteworthy that the diagonal members of the legs were interrupted at the first storey to provide entrance to the buildings. Hence, the box columns had to resist in shear and flexure the total horizontal base shear, in addition to the axial forces due to overturning.

The causes of the brittle column failures are not known at the time of writing. A very large number of factors can be easily identified as potential contributors to the observed undesirable behaviour, namely: metallurgical problems related to size effects, severe triaxial stress conditions due to restraints at the welds, possible low dynamic brittle-to-ductile transition temperature coupled with inordinate strain rates, preexisting crack conditions introduced by lesser quality welds executed on the alignment plates, and compounding effects from some of the known undesirable properties of thick steel sections. However, the exact cause of this failure may never be reliably determined without full-scale tests of identical structural shapes subjected to the same stress conditions, particularly since it is suspected that this problem is intricately related to the section sizes used. A universal testing machine capable of testing in tension the largest sections that failed at the Ashiyahama complex does not exist anywhere in the world at this time (it would require a tension capacity of about 45 000 kN), and a test simulating the axial, flexural, and shear forces these columns resisted at the first storey is even further beyond reach. In that perspective, the absence of a satisfactory, nonspeculative, explanation for the cause of these column failures should not be surprising.

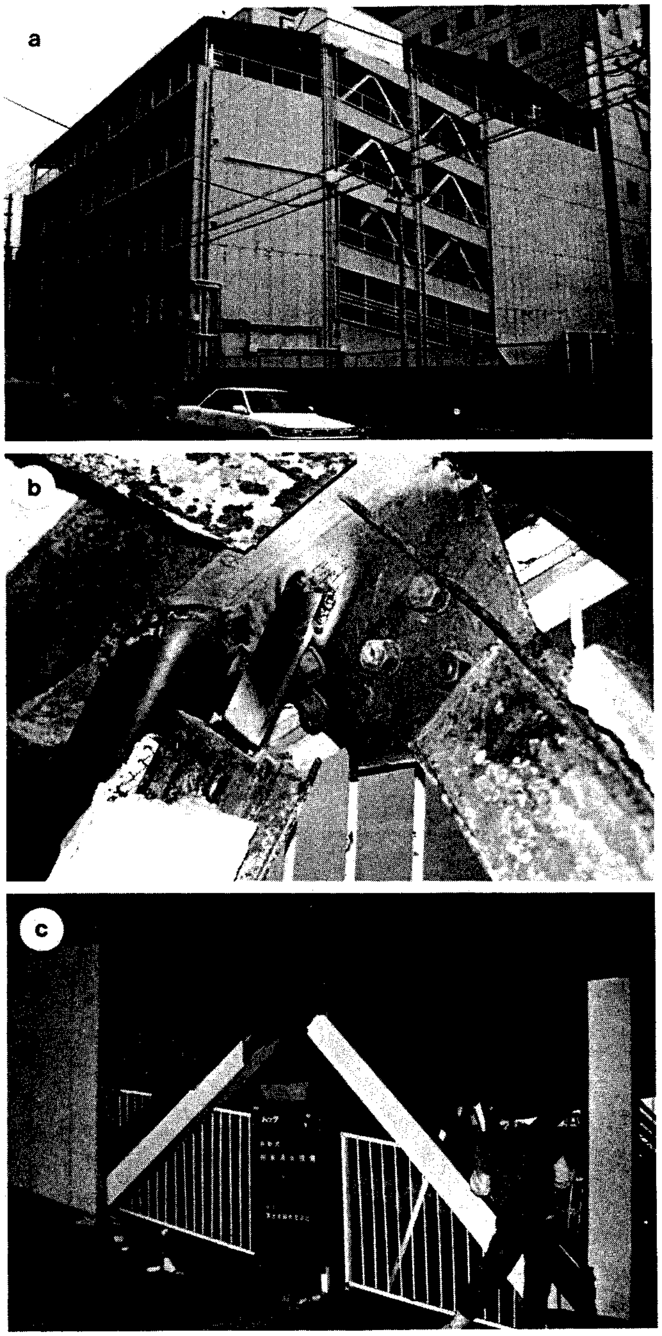
Repairs were accomplished within a month of the earthquake by welding thick steel plates to reinforce the fractured columns, and closing the existing cracks using weld metal (Fig. 41). While this strengthening was intended to be a temporary measure, it should be obvious that the added plates will become part of the permanent repair solution if further studies indicate that additional reinforcement is necessary (Nikkei Architecture 1995).

Discussion

Uncertainty in ground motion and structural systems

Only one year after the Northridge event, the Hyogo-ken Nanbu earthquake again dramatically demonstrated the destructive potential of large earthquakes occurring close to modern populated cities. During such events, structures located near or directly above the fault rupture are exposed to strong impulsive ground motions, exhibiting high peak velocities and large displacements, often in excess of code-specified values. In Kobe, most of the structural damage occurred in the vicinity of the faulting mechanism, while building structures in Osaka, which is located about 30 km

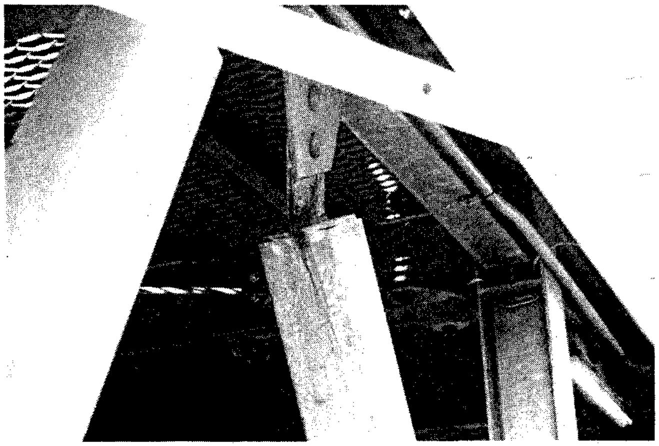
Fig. 31. Damage to a four-storey chevron bracing parking structure: (*a*) overall view; (*b*) fracture of a brace connecting plate, buckling of a gusset plate, and temporary repair scheme; (*c*) installation of additional bracing members.



east of Kobe, withstood the earthquake virtually intact.

Recent studies have shown that these near-source ground motions can be very demanding on building structures (Anderson and Bertero 1987; Hall et al. 1995) and that design based on an elastic response spectrum approach as currently implemented in the NBCC may not be sufficient to ensure safe seismic response for structures located in the near-field of important seismic events. Given the potential for large seismic events in urban regions of Canada, more research is needed to learn more about the characteristics of near-fault ground motions and their effects on building structures.

Fig. 32. Typical out-of-plane buckling of brace connecting plates.

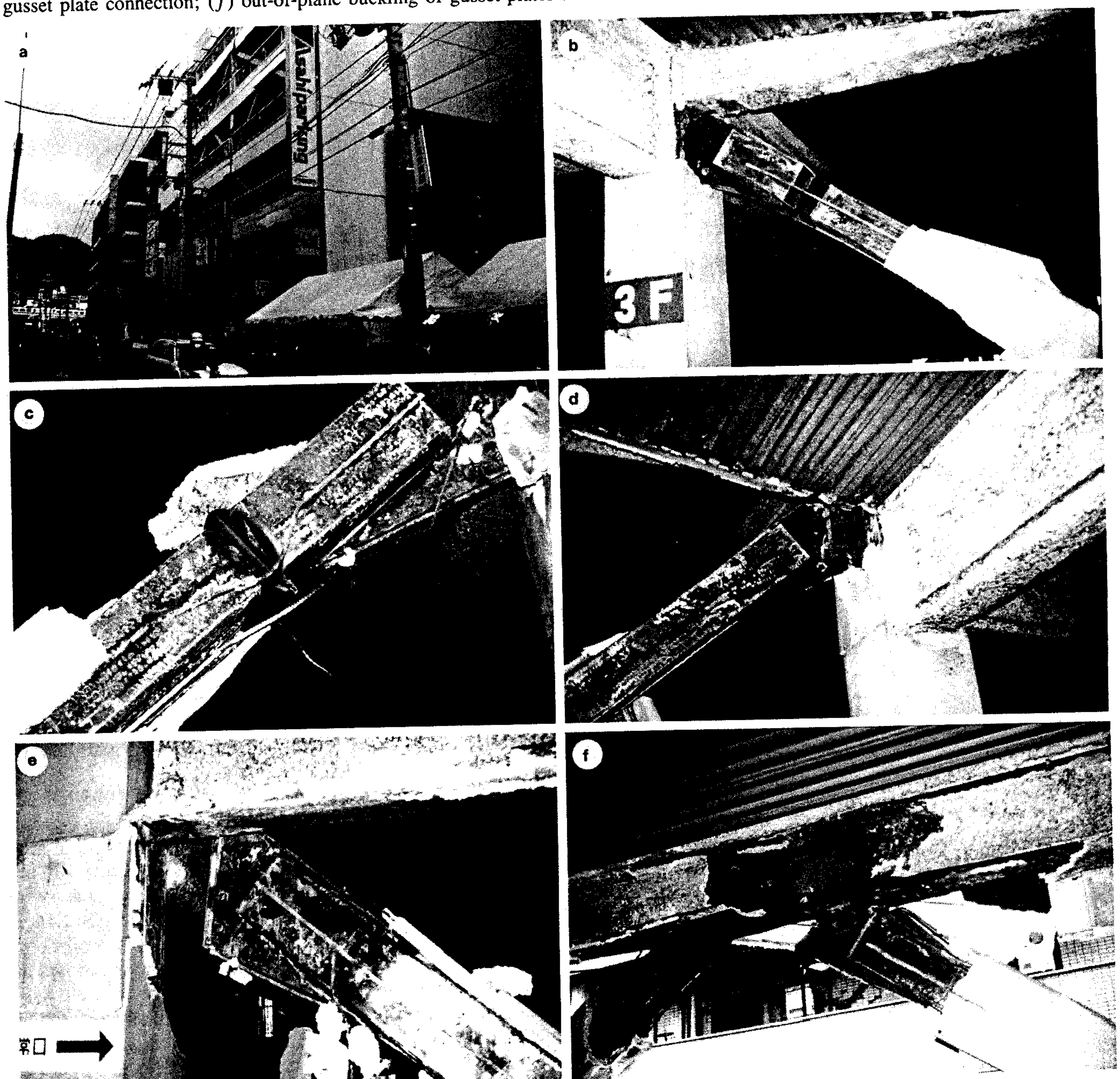


Considering the unavoidable high level of uncertainty associated with any prediction of both ground motion and building response, the main effort should be directed, however, towards ensuring that buildings in high seismic regions of Canada can behave in a ductile and stable manner under strong earthquakes. This can be achieved only by encouraging building designers to use ductile structural systems, with minimum irregularities and maximum redundancy. Recent seismic design provisions included in Canadian codes and standards undoubtedly represent a significant step in this direction; and the Kobe experience provides an excellent opportunity to further improve these provisions and thereby enhance the seismic safety level in Canada.

An example of a recent improvement in the 1995 NBCC is that buildings more than three storeys in height and located in a velocity- or acceleration-related seismic zone of 2 and higher must include a structural system qualifying for an R factor equal to or greater than 2.0. Such systems must comply with the capacity design requirements of the S16.1 Standard, which aim at ensuring proper inelastic behaviour of the ductile links in the lateral load resisting system and at providing an appropriate strength hierarchy within the system. Possible overloading due to unexpected severe ground motions can then be accommodated through increased inelastic deformation, with proper connection detailing, instead of risking occurrence of brittle failures with loss of structural integrity.

In Kobe, catastrophic failures resulting from brittle weld fractures in moment resisting frames with undersized beam-to-column connections, and premature fracture of brace connections, demonstrated the disastrous consequences of relaxing ductile detailing provisions for some categories of frames, such as low-rise structures or frames for which higher loads are specified. This suggests that the NBCC requirement for ductile systems (R of 2.0 or higher) in active seismic regions should be applicable to buildings of any height. Alternatively, undesirable brittle failure modes in low-rise structures could be avoided by prescribing a capacity design procedure for systems classifying for an R factor of 1.5 in high seismic zones.

Fig. 33. Damage to a five-storey split-X braced parking structure: (*a*) view of the facade; (*b*) tension failure of a tube bracing member; (*c*) local buckling of a tube bracing member; (*d*) failure of a bolted gusset plate connection; (*e*) failure of a welded gusset plate connection; (*f*) out-of-plane buckling of gusset plates and severe beam distortion at brace connection.

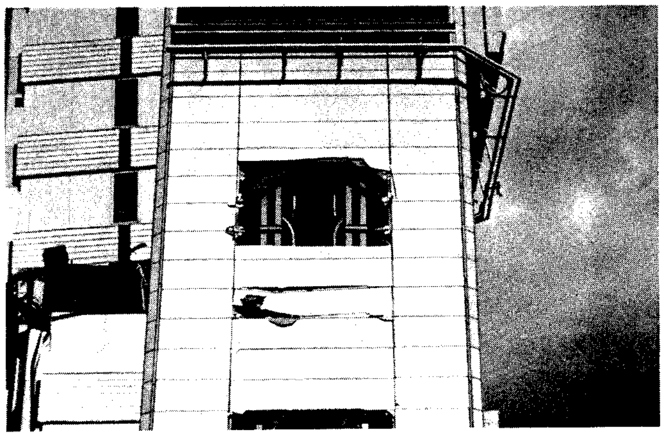


The undeniable benefits of structural redundancy and uniformity in achieving a high degree of earthquake resistance should also be emphasized in Canadian codes. Redundant systems generally exhibit higher overall toughness and are less prone to total collapse due to the failure of a single connection or member. Several buildings in Kobe did not collapse mainly because of the high redundancy designed into the structure by Japanese engineers.

Structural plan and vertical irregularity may cause significant concentrations of inelastic demand and premature failures in the structural system. As mentioned earlier, structures in Japan generally exhibit a uniform configuration, both in plan and elevation. Nevertheless, undesirable weak (or soft) storey behaviour could be observed in many old structures with large front openings at the first floor. A similar behaviour may have also contributed to some extent to the damage observed at the base of the trussed legs of the Ashiyahama residential buildings, as the web members at the ground level were omitted to allow access into the buildings.

Current NBCC provisions put little emphasis on the benefits of system redundancy and dangers of irregularity. A reward-based strategy should be implemented in Canada to encourage building designers to use redundant seismic force resisting systems. This could be done through an ultimate

Fig. 34. Shedding of the exterior cladding of the slender parking structure shown in Fig. 16*a* as a result of the out-of-plane buckling of chevron braces.



strength check of the structure, as in the Japanese code, or by reducing the specified seismic loads according to the level of redundancy introduced in the system. Conversely, systems with irregular configurations should be penalized, either by increasing the specified seismic loads (Japanese code) or by calling for more stringent detailing requirements, as suggested in the 1994 NEHRP seismic provisions (BSSC 1994).

Detailing provisions for moment resisting frames and braced frames

The Hyogo-ken Nanbu earthquake also highlighted some deficiencies in the fabrication and detailing practices that may need to be examined.

One problem that was confirmed by the Kobe event is the potential for brittle fractures in welded beam-to-column connections of steel moment resisting frames, even those carefully designed and built to achieve a desirable ductile behaviour. Current design assumptions and construction methods for welded beam-to-column connections in ductile moment resisting frames undoubtedly need to be critically revisited and assessed, and explicit guidelines must be made available to designers and fabricators. As reported by Tremblay et al. (1995*b*), such a process has already been initiated following the Northridge earthquake and preliminary guidelines have been proposed for the evaluation, repair, and design of welded moment resisting frames (SAC 1995), but definite answers are still awaited.

Several experimental investigations conducted so far have confirmed the lack of ductility of the commonly used moment connections with welded flanges and bolted web. The beam flange to column welds were found to fracture before any or very little plastic hinging develops in the beams. Furthermore, some limited tests indicated that extensive inelastic shear deformation of the column panel zone could initiate the fracture of the beam flange welds. Therefore, this energy dissipation mechanism, which is currently allowed by the S16.1 Standard for ductile moment resisting frames, may not represent a reliable substitute for beam hinging to achieve ductile behaviour. Alternative promising beam-to-column connections are currently being examined to

Fig. 35. Column damage in a braced frame.

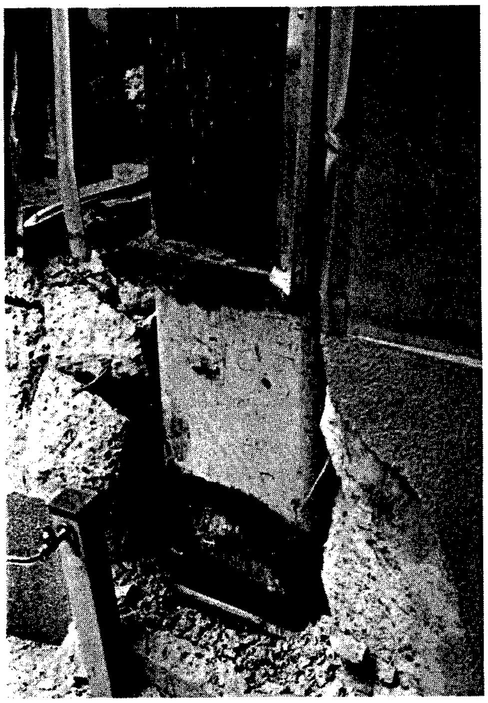
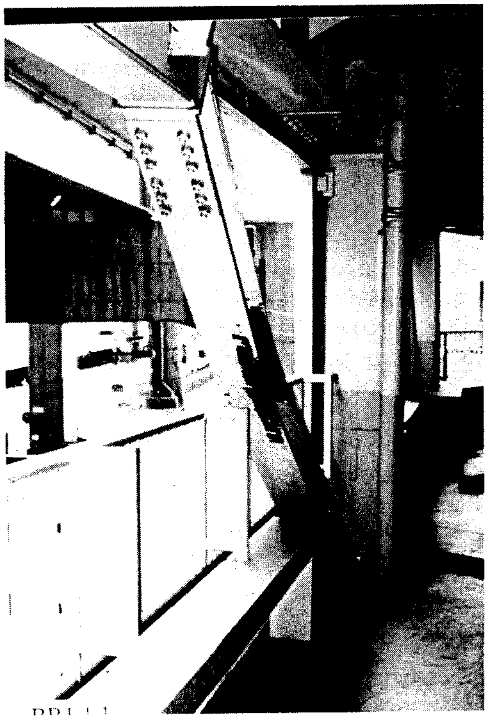


Fig. 36. Adequate inelastic performance of bracing members in a modern braced frame. (Courtesy of the Architectural Institute of Japan.)



reduce stresses in beam flange welds: haunch connections or reduced beam section connections. More research is still needed, however, before these systems can safely be used in practice.

Fig. 37. Types of column base connection: (*a*) base plate connection; (*b*) concrete encased column base; (*c*) embedded column base (adapted from AIJ 1995*d*).

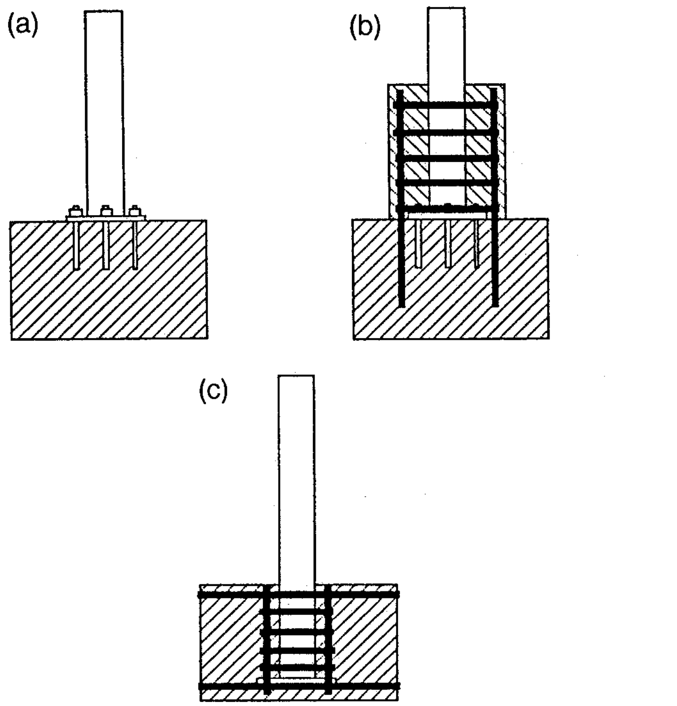
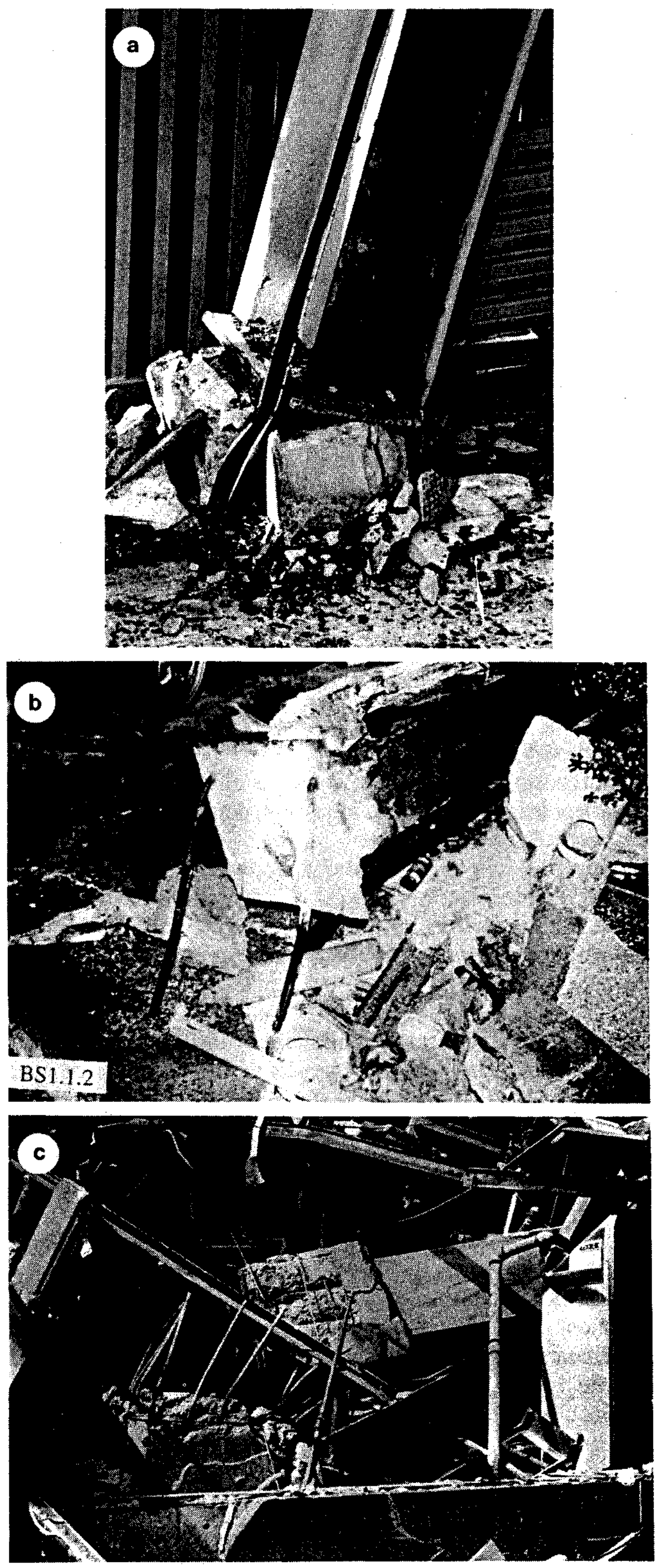


Fig. 38. Damage to column base connections: (*a*) anchor bolts fractured; (*b*) anchor bolts pulled out; (*c*) concrete encased column base rupture. (Courtesy of the Architectural Institute of Japan.)



Until final guidelines become available, practising engineers should thus be cautious when specifying moment resisting frames for seismic resistance and may consider other lateral load resisting systems such as concentrically or eccentrically braced frames. For eccentrically braced frames, when the link beam is connected to the column flange, it is advisable to use partially restrained beam-to-column connections, since fully rigid connections as currently specified in the S16.1 Standard could be prone to problems similar to those observed in moment resisting frames (SAC 1994).

The investigation of the damage sustained by concentrically braced frames in Kobe once again demonstrated the key importance of proper connection design. As discussed earlier, current S16.1 provisions require that these connections be sized to sustain the nominal brace tensile yield load in systems classifying for an R factor of 2.0 or more. The appropriateness of amplifying that force to account for possible brace overstrength (ratio of the actual to nominal yield strength, strain hardening, strain rate effects, etc.), as is currently done in Japan and elsewhere (e.g., SANZ 1989), should, however, be examined.

In frames with compression acting bracing members, numerous occurrences of gusset plate failures in bending and (or) buckling were observed as a result of the applied brace compression load or buckling of the brace. In some cases, this phenomenon led to the complete fracture of the gusset. The S16.1 provisions only require that the gusset be designed for the expected brace load, with no reference to the compression load. The observed performance of these structures in Kobe suggests that gusset plates be able to sustain the maximum anticipated brace compression load, which would be based on an upper bound estimate of the compressive strength of the brace. When in-plane buckling of the braces is expected during an earthquake, the gusset plate and its connections should be able to sustain, in addition to the aforementioned axial load, a bending moment equal to the actual flexural capacity of the brace.

In several buildings in Kobe, out-of-plane buckling of stocky compression acting braces produced failure of adjacent walls. The free fall of wall components represents a serious life hazard, especially when heavy cladding parts fall off the facade of multistorey structures. Future code provisions could address this potential hazard by calling for a minimum horizontal spacing between bracing members and walls.

The Kobe experience also demonstrated (surprisingly perhaps) that tension-only braced frames with well-designed brace connections (Fig. 29) can behave in a satisfactory manner during strong ground shaking. Although this system generally exhibits lower energy dissipation capability due to a severely pinched hysteretic behaviour, it can represent a reliable solution for resisting earthquake-induced forces. Slender braces buckle in a nearly elastic mode, and hence are less vulnerable to low cycle fatigue than stockier compression acting braces, which experience intense inelastic action within plastic hinge regions. In addition, gusset plates in tension-only braced frames are exposed to lower compression loads than in tension–compression systems. Incidentally, in the 1994 NEHRP seismic provisions, the maximum slenderness ratio for bracing members in special concentrically braced frame systems (the equivalent in the United States of the Canadian ductile braced frame category) has been eliminated, as a result of the longer post-buckling life exhibited by more slender braces. A similar relaxation should be examined for ductile braced frames in Canada.

The common occurrence of damage to column bases raises concern about the lack of explicit seismic design provisions for these elements in current Canadian codes. Column anchorage plays a key role in transmitting ground motion to the superstructure, and hence affects its response characteristics. Intentionally undersizing column anchorage may be beneficial for some superstructures because of the isolation mechanism that develops from rocking of the structure during an earthquake. Such an approach may, however, lead to excessive building deformations. Moreover, as was observed in Kobe, most anchorage systems exhibit limited energy absorption and dissipation capabilities and may fail prematurely and impair the integrity of the structural system. In view of their importance, more attention should be given to these elements in seismic design code provisions.

One of the main design issues resulting from this earthquake is the potential for brittle fracture in heavy steel shapes, as observed in the Ashiyahama Town Buildings. The ductility and the weldability of thick steel material may not be appropriate for application in seismic resistant structures, but research is still needed to establish appropriate acceptance testing procedures and criteria.

Existing structures

The much larger vulnerability of buildings designed under earlier, less stringent earthquake code requirements was again demonstrated by the Kobe earthquake. While upgrading to ensure full compliance to the latest codes and standards is conservative and generally possible, it is an expensive proposition, as it normally implies neglecting the contribution of the lateral load resistance of most of the noncomplying existing elements. Hence, as currently attempted by some bridge codes (e.g., CSA 1996), building design codes and standards must start to address explicitly issues germane to seismic rehabilitation. This issue is of significance in Canada, given that comprehensive seismic design requirements for steel buildings were not existent before 1989.

In that same perspective, significant research work is still needed to determine how to retrofit older, less ductile, steel structures in a cost-effective way. A nonexclusive list of items worthy of such consideration should include the quantification of the seismic resistance of old connections in redundant structural systems including, for instance, semi-rigid connections, and a better quantification of the threshold of undesirable slenderness ratios for both global and local buckling of various bracing member cross sections, column base connections, etc.

Fig. 39. Partial overview of the Ashiyahama Residential Complex.

Conclusions

The 1995 Hyogo-ken Nanbu earthquake produced extensive damage to steel building structures in the Kobe area. Various types and sizes of steel framing suffered from the earthquake and many different structural failure modes were observed. These consequences are of particular interest to the Canadian engineering community, as the seismic setting for the Kobe region and parts of Canada are very similar and analogous design and construction practices are used in Japan and Canada.

Given the high uncertainty associated with the prediction of ground motions and building responses, the NBCC should require that a ductile system, for which a comprehensive capacity design is performed, be provided for any building size in high seismic regions of Canada. Numerous instances of dramatic failures and collapses due to nonductile detailing demonstrated the detrimental effect of relaxing this requirement in active seismic zones. The observations made after

Fig. 40. Structural failures in the Ashiyahama Residential Complex: (*a*) fracture of the boxed columns away from the column splice; (*b*) failure of the boxed columns at the welded column splice; (*c*) fracture of the boxed columns at the column to brace connection, with extension into the brace; (*d*) failure of the web member to column welded connection, without adjacent column fracture.

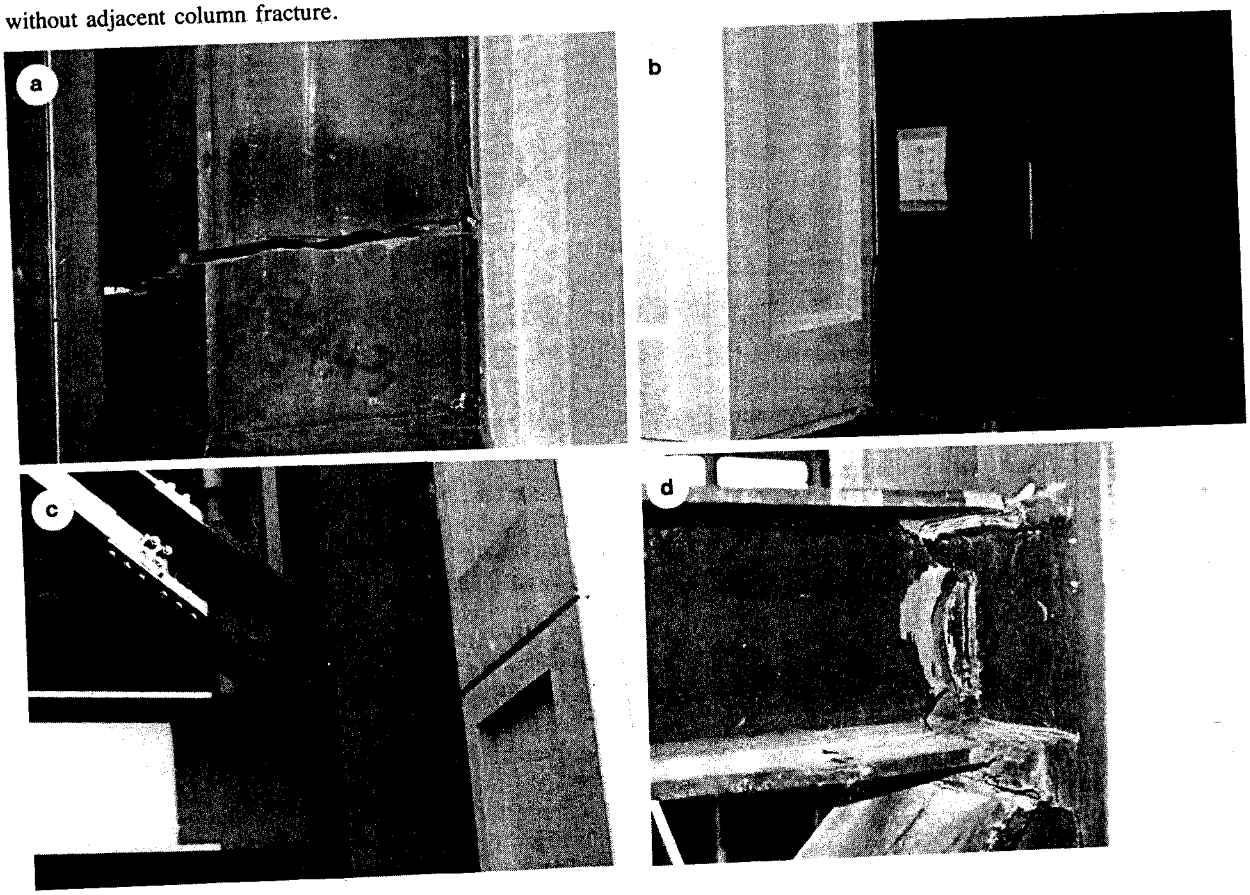
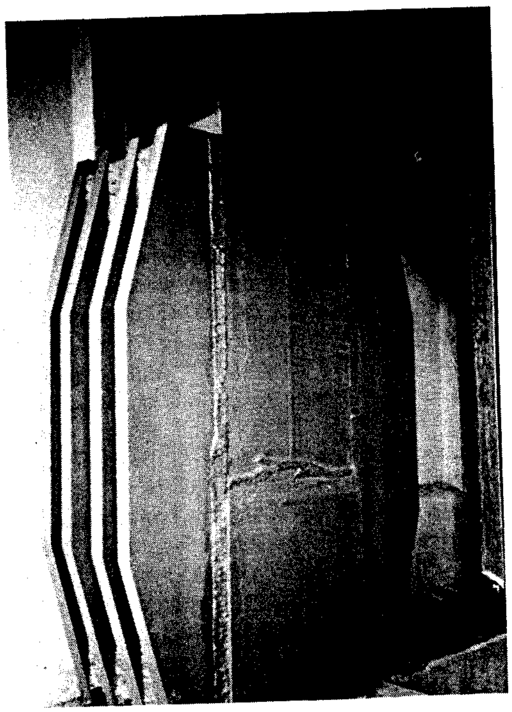


Fig. 41. Welded of and addition of reinforcement plates to cracked columns in the Ashiyahama Residential Complex.



the earthquake also suggest that stronger incentives should be introduced in the NBCC to encourage the use of highly redundant and uniform lateral load resisting systems.

The Kobe earthquake reinforced the concern brought up after the 1994 Northridge earthquake about the capability of welded beam-to-column connections in steel moment resisting frames to safely withstand earthquake ground motions. This issue is currently receiving much attention and new design and retrofit provisions should be made available to designers in the short term. Meanwhile, structural engineers should be cautious when using this system and stay alert to upcoming developments. Failures in braced frames mainly took place in brace connection elements or at locations of minimum net area of bracing members. The S16.1 Standard currently addresses the tensile resistance of brace connections, but design provisions are needed to ensure adequate compressive strength of the gusset plates. In addition, the appropriateness of including a brace overstrength factor for brace connection design, as this is done in Japan, should be examined.

A large number of failures also occurred in column base connections of steel frames and this earthquake clearly exposed the serious hazard potential associated with this failure mode. Explicit design requirements should be included in the S16.1 Standard to prevent this undesirable behaviour. The poor performance of the Ashiyahama residential high-rise steel frames raised some concern with respect to the use of heavy steel sections for earthquake-resistant design. This topic undoubtedly deserves further investigation in the near future.

The Hyogo-ken Nanbu earthquake also emphasized the large vulnerability of old structures designed according to less stringent seismic provisions than exist at present. In Canada, many such structures are located in high seismic zones and practical guidelines to efficiently retrofit these buildings are still needed. The experience from that earthquake also corroborated the fact that damage to steel frames is often hard to detect, and that careful postearthquake inspection of steel structures should be mandatory.

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References

AIJ. 1964. AIJ structural standards. Structural Standards Committee, Architectural Institute of Japan, Tokyo, Japan.

AIJ. 1995*a*. Quick report on the Hanshin earthquake. Architectural Institute of Japan, Osaka, Japan. (In Japanese.)

AIJ. 1995*b*. Investigation of the disaster due to the Hyogo-Ken Nanbu earthquake — The urgent meeting report. Architectural Institute of Japan, Tokyo, Japan.

AIJ. 1995*c*. Preliminary reconnaissance report of the 1995 Hyogoken-Nanbu earthquake. English edition. Architectural Institute of Japan, Tokyo, Japan.

AIJ. 1995*d*. Performance of steel buildings during the 1995 Hyogoken-Nanbu earthquake. Architectural Institute of Japan, Tokyo, Japan. (In Japanese with English summary.)

Anderson, J.C., and Bertero, V.V. 1987. Uncertainties in establishing design earthquakes. ASCE Journal of Structural Engineering, **8**: 1709−1724.

Aoki, S. 1994. Qualification system for structural engineers in Japan. Report ATC 15-5, Proceedings of the Sixth U.S. − Japan Workshop on the Improvement of Structural Design and Construction Practices, Victoria, B.C., pp. 9.1−9.5.

Aoyama, H. 1981. Outcome of earthquake provisions in the recently revised Japanese building code. Bulletin of the New Zealand National Society for Earthquake Engineering, **14**: 63−80.

Basham, P.W., Weichert, D.H., Anglin, F.M., and Berry, M.J. 1982. New probabilistic strong seismic ground motion maps of Canada: a compilation of earthquake source zones, methods and results. Earth Physics Branch, Energy, Mines and Resources Canada, Ottawa, Ont., Open File Number 82-33.

BCJ. 1975. Data Evaluation Sheet #BCJ-H242. Building Center of Japan, Tokyo, Japan, pp. 45−46.

BSSC. 1994. NEHRP recommended provisions for seismic regulations for new buildings. Building Seismic Safety Council, Washington, D.C.

Byrne, P.M., Finn, W.D.L., and Ventura, C.E. 1995. Hyogo-ken Nanbu (Kobe) earthquake: 17 January 1995. *In* The Hyogo-ken Nanbu earthquake — Preliminary reconnaissance report. Canadian Association for Earthquake Engineering, Vancouver, B.C., pp. 1−22.

CSA. 1994. Limit states design of steel structures. Standard CAN/CSA-S16.1-94, Canadian Standards Association, Rexdale, Ont.

CSA. 1996. Canadian highway bridge design code. Standard CSA-S6-96, Canadian Standards Association, Rexdale, Ont.

Hall, J.F., Heaton, T.H., Halling, M.W., and Wald, D.J. 1995. Near-source ground motion and its effects on flexible buildings. Earthquake Spectra, **4**: 569−605.

Hisatoku, T. 1984. A unique mechanism of high-rise residential buildings by large steel structural framework. *In* Comparison of building seismic design practices in the United States and Japan. Applied Technology Council, Redwood City, Calif., Report ATC-15, pp. 199−215.

IAEE. 1992. Earthquake resistant regulations. A world list — 1992. International Association for Earthquake Engineering, Tokyo, Japan, pp. 23.55−23.71.

ICBO. 1994. Uniform building code (UBC). International Conference of Building Officials, Whittier, Calif.

Ishiyama, Y. 1984. New seismic design method for buildings in Japan. *In* Comparison of building seismic design practices in the United States and Japan. Applied Technology Council, Redwood City, Calif., Report ATC-15, pp. 15−23.

Ishiyama, Y. 1989. Japanese seismic design method and its history. Proceedings of the Third U.S. — Japan Workshop on the Improvement of Building Structural Design and Construction Practices, Applied Technology Council, Tokyo, Japan, Report ATC-15-2, pp. 55−64.

Kitagawa, Y., and Takino, F. 1994. Japanese building code. *In* International handbook of earthquake engineering — codes, programs and examples. *Edited by* M. Paz. Chapman & Hall, New York, N.Y. Chap. 24.

Kobe University. 1995. Second report on Great Hanshin earthquake. Reconnaissance Team on Great Hanshin Earthquake, Department of Civil Engineering, Faculty of Engineering, Kobe University, in collaboration with the Hyogo Prefectural Government, the Kobe City Government, and the Construction Engineering Research Institute Foundation, Kobe, Japan.

Koller, M.G., Studer, J.A., and Wenk, T. 1995. The Great Hanshin earthquake of January 17, 1995. Ingénieurs et Architectes Suisses, **8**: 136−143. (In French.)

Mitchell, D., DeVall, R.H., Kobayashi, K., Tinawi, R., and Tso, W.K. 1996. Damage to concrete structures due to the January 17, 1995, Hyogo-ken Nanbu (Kobe) earthquake. Canadian Journal of Civil Engineering, **23**: this issue.

Nikkei Architecture. 1995. **2**: 78−83. (In Japanese.)

NRCC. 1995. National building code of Canada. 11th ed. Associate Committee on the National Building Code, National Research Council of Canada, Ottawa, Ont.

Omika, Y., and Sugano, T. 1989. Current practice in dynamic analysis. Proceedings of the Third U.S. − Japan Workshop on the

Improvement of Building Structural Design and Construction Practices, Applied Technology Council, Tokyo, Japan, Report ATC 15-2, pp. 311–323.

Park, R., Billings, I.J., Clifton, G.C., Cousins, J., Filiatrault, A., Jennings, D.N., Jones, L.C.P., Perrin, N.D., Rooney, S.L., Sinclair, J., Spurr, D.D., Tanaka, H., and Walker, G. 1995. The Hyogo-ken Nanbu earthquake (The Great Hanshin earthquake) of 17 January 1995. Report of the NZNSEE Reconnaissance Team. Bulletin of the New Zealand National Society for Earthquake Engineering, **28**: 1–98.

SAC. 1994. Proceedings of the Invitational Workshop on steel seismic issues. SAC Joint Venture, Sacramento, Calif., Report No. SAC-94-01.

SAC. 1995. Interim guidelines: evaluation, repair, modification and design of welded steel moment frame structures. SAC Joint Venture, Sacramento, Calif., Report No. SAC-95-02.

SANZ. 1989. Steel structures code, New Zealand Amendments to AS1250, 1981. Standard NZS 3404, Part 1: 1989, Standards Association of New Zealand, Wellington, New Zealand.

Tremblay, R., Bruneau, M., Prion, H.G., Filiatrault, A., and DeVall, R. 1995*a*. Preliminary observations of the performance of steel building structures during the January 17, 1995 Hyogo-ken Nanbu earthquake. *In* The Hyogo-ken Nanbu earthquake — Preliminary reconnaissance report. Canadian Association for Earthquake Engineering, Vancouver, B.C., pp. 46–70.

Tremblay, R., Timler, P., Bruneau, M., and Filiatrault, A. 1995*b*. Performance of steel structures during the 1994 Northridge earthquake. Canadian Journal of Civil Engineering, **22**: 338–360.

Umemura, H. 1989. Seismic damage prevention in Japan. Proceedings of the Third U.S. – Japan Workshop on the Improvement of Building Structural Design and Construction Practices, Applied Technology Council, Tokyo, Japan, Report ATC 15-2, pp. 3–5.

Wada, A., Okoshi, T., Teramoto, T. and Yamaguchi, S. 1994. Practical use of dynamical response analysis in seismic design. Proceedings of the Sixth U.S. – Japan Workshop on the Improvement of Structural Design and Construction Practices, Victoria, B.C., Report ATC 15-5, pp. 17.1–17.8.

Yano, K. 1994. Qualification system for building engineers in Japan. Proceedings of the Sixth U.S. – Japan Workshop on the Improvement of Structural Design and Construction Practices, Victoria, B.C., Report ATC 15-5, pp. 11.1–11.4.

List of symbols

C seismic coefficient
C_u seismic coefficient (for ultimate strength)
D_s structural coefficient
F foundation factor
F_{es} shape factor
F_u tensile strength of steel (MPa)
F_y yield strength of steel (MPa)
h building height (m)
I importance factor
R force modification factor
R_t vibration characteristic factor
S seismic response factor
T fundamental period of vibration of a structure (s)
U calibration factor
v zonal velocity ratio
Z seismic zone factor
β portion of storey shear resisted by bracing members
β_u portion of the ultimate storey shear resisted by bracing members
ϕ resistance factor
λ effective brace slenderness ratio