Seismic force modification factors for the proposed 2005 edition of the National Building Code of Canada[1]

Denis Mitchell, Robert Tremblay, Erol Karacabeyli, Patrick Paultre, Murat Saatcioglu, and Donald L. Anderson

Abstract: This paper describes the proposed changes to the 2005 edition of the National Building Code of Canada related to the force modification factors. A description of the ductility- and overstrength-related force modification factors is given. The selection of the values proposed for these two factors for the various seismic force resistance systems is given in light of the design and detailing provisions that are specified in the Canadian Standards Association standards for steel, concrete, timber, and masonry building structures.

Key words: buildings, ductility, earthquakes, force modification factors, overstrength, seismic.

Résumé : Cet article décrit les changements proposés à l’édition 2005 du Code National du Bâtiment du Canada (CNBC) concernant les facteurs de modification de force. Une description des facteurs de modification de force reliés à la ductilité et reliés à la sur-résistance est donnée. La sélection des valeurs proposées pour ces deux facteurs pour les différents systèmes de résistance des forces sismiques est donnée en lumière des dispositions de conception et des épures spécifiées dans les normes CSA pour l’acier, le béton, le bois et les structures en maçonnerie.

Mots clés : bâtiments, ductilité, tremblements de terre, facteurs de modification de force, sur-résistance, sismique.

[Traduit par la Rédaction]

Introduction

The base shear equation for seismic design has undergone significant evolution over the years. A summary of this evolution in the United States is given in ATC (1995*a*) and changes to the National Building Code of Canada (NBCC) have been documented by Heidebrecht and Tso (1985) and Tso (1992). The 1995 NBCC expressed the minimum lateral seismic force at the base of the structure, V, as

$$V = (V_e/R)U \qquad [1]$$

where V_e is the equivalent lateral force at the base of the structure representing elastic response, R is the force modification factor, and U is a calibration factor (U = 0.6). The force V_e was determined from the product of the zonal velocity ratio, the seismic response factor, the importance factor, the foundation factor, and the seismic weight (NBCC 1995).

The force modification factor, R, reflected the capability of a structure to dissipate energy through inelastic behaviour. It was intended to characterize the important aspects of the hysteretic behaviour of different structural systems undergoing inelastic response under severe earthquake events. This factor was often referred to as a general “ductility” factor, indicative of the ability of the structure to undergo deformations beyond yielding, but also included several other key features such as energy absorption and the ability to sustain load and stiffness under reversed cyclic loading.

The values of R ranged from 1.0 for very brittle systems to 4.0 for the most ductile systems. These values were estab-

Received 13 May 2002. Revision accepted 5 December 2002. Published on the NRC Research Press Web site at http://cjce.nrc.ca on 14 April 2003.

D. Mitchell.[2] Department of Civil Engineering and Applied Mechanics, McGill University, 817 Sherbrooke Street West, Montréal, QC H3A 2K6, Canada.
R. Tremblay. Department of Civil, Geological and Mining Engineering, École Polytechnique, Montréal, QC H3C 3A7, Canada.
E. Karacabeyli. Wood Engineering Department, Western Division, Forintek Canada Corporation, Vancouver, BC V6T 1W5, Canada.
P. Paultre. Department of Civil Engineering, University of Sherbrooke, Sherbrooke, QC J1K 2R1, Canada.
M. Saatcioglu. Department of Civil Engineering, University of Ottawa, Ottawa, ON K1N 6N5, Canada.
D.L. Anderson. Department of Civil Engineering, The University of British Columbia, Vancouver, BC V6T 1Z4, Canada.

Written discussion of this article is welcomed and will be received by the Editor until 31 August 2003.

[1]This article is one of a selection of papers published in this Special Issue on the Proposed Earthquake Design Requirements of the National Building Code of Canada, 2005 edition.
[2]Corresponding author (e-mail: Denis.Mitchell@mcgill.ca).

lished from test results on energy-dissipating components and on subassemblages, studies of structural systems using nonlinear analyses, and assessment of the behaviour of structures in major earthquakes. To make use of the higher R values, the engineer must satisfy the design and detailing requirements given in the appropriate Canadian Standards Association (CSA) standard.

Significant changes are proposed for the 2005 edition of the NBCC for the determination of the seismic base shear, as given by

$$[2] \qquad V = \frac{S(T_a)M_v I_E W}{R_d R_o}$$

where $S(T_a)$ is the design spectral response acceleration, expressed as a ratio of gravitational acceleration, for the fundamental lateral period of vibration of the building T_a; M_v is a factor to account for higher mode effects on base shear; I_E is an earthquake importance factor of the structure; W is the dead load plus 25% of the design snow load plus 60% of the storage load and full content of tanks; R_d is a ductility-related force modification factor that reflects the capability of a structure to dissipate energy through inelastic behaviour; and R_o is an overstrength-related force modification factor that accounts for the dependable portion of reserve strength in a structure designed according to the NBCC provisions.

The design spectral response acceleration values, $S(T_a)$, are determined from a site-specific uniform hazard spectrum (UHS), which is then modified to account for the soil profile characteristics at the site (Adams and Atkinson 2003). The product $S(T_a)M_v I_E W$ is the equivalent lateral force at the base of the structure representing elastic response, as described by Heidebrecht (2003) and Humar and Mahgoub (2003). A major change from the 1995 NBCC is the introduction of two force modification factors and the elimination of the calibration factor U. The purpose of this paper is to describe the rationale for selecting the proposed values for the ductility- and overstrength-related force modification factors. The design and detailing requirements, consistent with these new force modification factors, are also summarized in this paper.

General approach for determining *R* factors

Although past codes have recognised the importance of ductility in seismic design, only recently have design approaches attempted to consider the additional influence of the inherent overstrength in different structural systems. It is proposed for the 2005 NBCC to include two separate factors, one for ductility and one for overstrength.

Ductility-related force modification factor, R_d

The ductility-related force modification factor, R_d, essentially corresponds to the R factor used in previous editions of the NBCC. In the proposed code, this factor ranges from 1.0 for brittle systems such as unreinforced masonry to 5.0 for the most ductile systems such as ductile moment-resisting steel frames. It is believed that this range is realistic for multi-degree-of-freedom structures (Park and Paulay 1975; Paulay and Priestley 1992). After the collapse of many structures in the 1985 Mexico earthquake, the code for the design of structures for the Federal District of Mexico City (Instituto de Ingeniería 1987) was changed, resulting in a reduction of the maximum value of the ductility factor, Q, from 6.0 to 4.0 for the ductile moment-resisting frames of concrete or steel. These changes provide guidance for the practical limits of ductility-related factors for some structural systems. The fact that the 2001 draft of Eurocode 8 (ECS 1998) provisions for seismic design specifies a ductility-related force modification factor, q, varying from 1.0 to 5.0 provides further evidence of the realistic range for the factor R_d.

Some other design codes have specified higher values of force modification factors than those proposed for the 2005 NBCC. For instance, the National Earthquake Hazard Reduction Program (NEHRP 1997) provisions prescribe a combined force modification factor, R, as high as 8.0 for the most ductile systems. Designers are cautioned not to use these higher R factors out of context, however, as they represent more than just the ductility of the system. These factors must be used only in conjunction with the corresponding ground motion design level.

To exhibit the necessary ductility and energy absorption to qualify for a given value of R_d specified in the NBCC, the structural system must be carefully designed and detailed in accordance with the relevant CSA standard. These requirements are discussed later in the paper. For the more ductile systems, one must not only ensure ductile response of individual elements of the seismic force resisting system (SFRS) but also apply capacity design principles (Park and Paulay 1975). Capacity design is aimed at providing significant yielding in those elements known to have the most ductile response, while limiting inelastic demand in the other elements and avoiding all potential brittle failure modes. This results in a structural system with a controlled hierarchy of yielding to maximize the energy dissipation.

Overstrength-related force modification factor, R_o

Although past codes have always attempted to calibrate the seismic design force values to historical levels that were deemed appropriate, a major departure has been undertaken for the proposed 2005 NBCC. Site-specific UHSs have been provided for all locations in the country to give realistic estimates of the elastic force demand as a function of the period. The ground motions have been chosen to represent a relatively rare event with a probability of exceedance of 2% in 50 years (return period of 2500 years). During such a severe event, it is expected that structures having a “normal” importance category would be damaged but would not collapse. Consequently, the actual capacity of the structure may be fully mobilized, with the more ductile structures undergoing significant inelastic action.

Traditionally, structures have been designed such that the members have factored resistances equal to or greater than the effects from factored loads. However, it has been shown that structures, particularly the more ductile ones, can have a considerable reserve of strength that is not explicitly considered in the 1995 NBCC (Fajfar and Fischinger 1990; Osteraas and Krawinkler 1990; Nassar and Krawinkler 1991; Paulay and Priestley 1992; Mitchell and Paultre 1994; ATC 1995*a*, 1995*b*, 1997; Rahgozar and Humar 1998).

Fig. 1. Stages in the response of a frame structure.

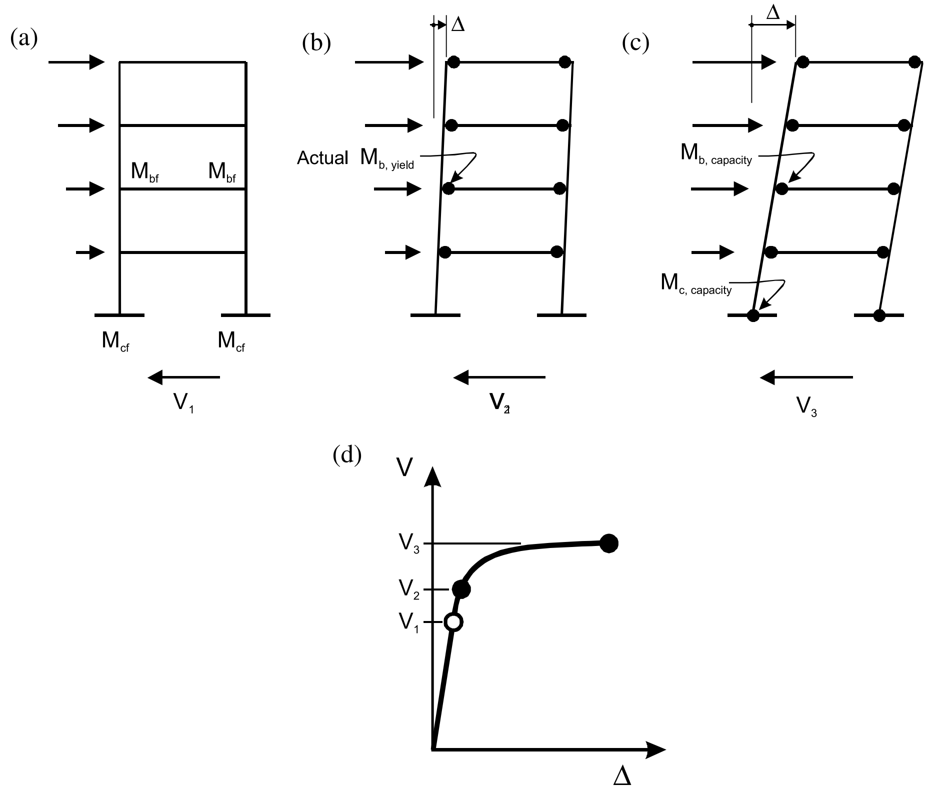
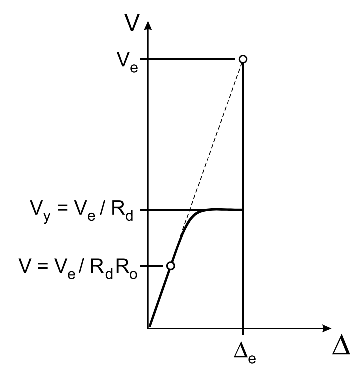


Figure 1 illustrates the stages of response of a simple frame structure as the lateral load is increased from the design factored load, V_1, to the load V_3 that produces a collapse mechanism. The lateral load V_1 corresponds to factored moments M_{bf} and M_{cf} in the beams and the columns, respectively. It takes a greater load, V_2, to develop the actual yield strength of the beams $M_{b,yield}$. This larger resistance is because the size of the beams is typically somewhat larger than that required and the actual yield stress is generally greater than the minimum specified yield strength. When capacity design procedures have been adopted, a further increase in the resistance of the structure is possible. For the simple frame shown, with the columns fixed at their bases, capacity design requires that the columns be designed to ensure that plastic hinging will form first in the beams, with the full capacity of the system being reached only when the columns yield at their bases (i.e., weak-beam, strong-column concept). For this to be possible, the ductile beams must be carefully detailed to sustain their capacity ($M_{b,capacity}$) under large inelastic deformations without strength degradation until the column capacities ($M_{c,capacity}$) are reached to form a collapse mechanism under load V_3.

The proposed revisions to the 2005 NBCC include an explicit overstrength-related force modification factor, R_o, to account for this reserve of strength. In lieu of increasing the factored resistance to account for overstrength, the design force level is reduced by including the R_o factor in the denominator of eq. [2]. This approach is more in line with usual design procedures where the factored resistance is compared with the factored load effects as obtained from linear analysis. Figure 2 shows the resulting reduced design force, *V*. For design purposes, only the so-called dependable

Fig. 2. Determination of the lateral design force, *V*, including ductility- and overstrength-related force modification factors. V_y, lateral force at yielding; Δ, roof displacement; Δ_e, roof displacement corresponding to V_e.



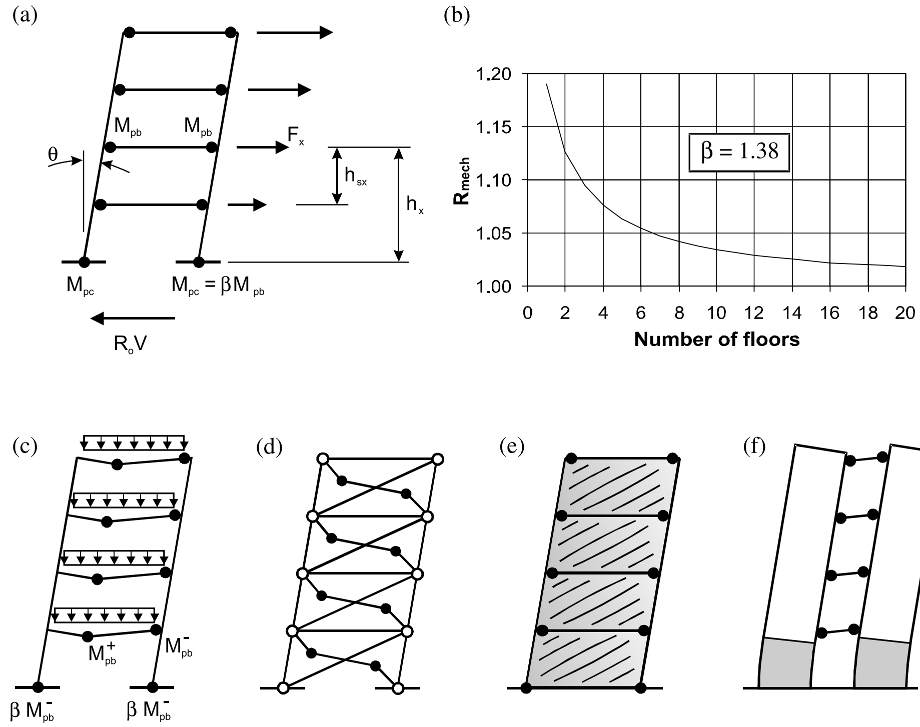
or minimum overstrength may be used. For a particular structural system, this dependable overstrength arises from the application of the design and detailing provisions prescribed in the appropriate CSA standard. The proposed 2005 NBCC has overstrength factors, R_o, that have been determined in a consistent manner for all systems in conformance with the CSA provisions.

To account for the various components contributing to the overstrength-related force modification factor, R_o, the following formulation was chosen:

[3] $$R_o = R_{size}R_{\phi}R_{yield}R_{sh}R_{mech}$$

where R_{size} is the overstrength arising from restricted choices for sizes of members and elements and rounding of sizes and

Fig. 3. Overstrength arising from the formation of collapse mechanisms: (*a*) simple frame collapse mechanism; (*b*) influence of number of floors on R_{mech} for simple frames; (*c*) reinforced concrete frame; (*d*) concentrically braced steel frame; (*e*) steel plate wall; (*f*) reinforced concrete coupled wall. F_x, lateral force at level *x*; h_x, height above base at level *x*; h_{sx}, storey height at level *x*; M_{pb}, plastic hinging moment in beam; M_{pc}, plastic hinging moment in column; θ, column plastic hinge rotation.



dimensions; R_ϕ is a factor accounting for the difference between nominal and factored resistances, equal to 1/ϕ, where ϕ is the material resistance factor as defined in the CSA standards; R_{yield} is the ratio of "actual" yield strength to minimum specified yield strength; R_{sh} is the overstrength due to the development of strain hardening; and R_{mech} is the overstrength arising from mobilizing the full capacity of the structure such that a collapse mechanism is formed.

R_{size} accounts for the fact that designers have restricted choices for sizes of members and elements. For example, only standardized choices are available for structural steel shapes, plates, reinforcing steel bars, timber members, and masonry units. In addition, practical considerations often lead to conservative rounding of dimensions such as spacing of connectors and reinforcing elements.

The factor R_ϕ is included in eq. [3] because it is appropriate to use nominal resistances when designing for an extremely rare event such as earthquake effects corresponding to a return period of 2500 years. There is some precedence for using unfactored resistances to evaluate near-collapse conditions under extreme or accidental load effects (e.g., for design of structural integrity reinforcement in slabs to prevent progressive collapse).

The factor R_{yield} accounts for the fact that the minimum specified material strength typically underestimates the actual strength.

R_{sh} accounts for the ability of strain hardening to develop in the material at the anticipated level of deformation of the structure. Therefore, it varies with the type of material and the extent of inelastic action that can develop in the structural system. Hence, more ductile structures, designed with higher R_d values, have larger R_{sh} values.

R_{mech} accounts for the additional resistance that can be developed before a collapse mechanism forms in the structure. A structure can display this additional resistance only if it is redundant and if yielding takes place in a sequence rather than all at once (see Fig. 1). Figure 3*a* illustrates the static collapse mechanism for a simple frame structure with *N* storeys. If it assumed that due to design requirements, the flexural strength of each column is β times that of each beam, then it can be shown from plastic analysis (equating internal work and external work) that the overstrength arising from hierarchy of yielding is given by

$$[4] \qquad R_{mech} = \frac{N + \beta}{N + 1}$$

Figure 3*b* shows that R_{mech} from eq. [4] decreases with an increase in the number of storeys, *N*. In Fig. 3*b*, a typical value for β for ductile moment-resisting concrete frames of 1.38 has been assumed. The ratio β equals 1.34 for ductile moment-resisting steel frames. The reduction of R_{mech} with increasing values of *N* is because the contribution of the yielding of the columns at their bases to the capacity of the system diminishes with an increase in the number of storeys. The assessment of R_{mech} for more realistic frames is generally more complex because other parameters must be considered. This is illustrated in Fig. 3*c* for a reinforced concrete frame in which the beams carry gravity loading and have different flexural resistances, $M_{pb}{}^+$ and $M_{pb}{}^-$, in positive and negative bending. The R_{mech} factor, however, typically exhibits a similar reduction with an increase in building height.

Figures 3*d*–3*f* show the mechanism that can develop in other structural systems. In tension–compression concentrically braced steel frames, overstrength arises once buckling of the compression brace has occurred and additional force is required to develop yielding in the tension brace (Tremblay 2001). For the ductile steel plate wall system, yielding occurs first in the plate, with the full mechanism developing only after plastic hinging occurs in the more flexible surrounding steel frame. In ductile reinforced concrete coupled walls, yielding develops first in the coupling beams followed by flexural yielding at the base of the walls.

Relationships between design and detailing requirements and R_d and R_o values

In the proposed 2005 NBCC, the values of R_d and R_o were determined to be consistent with the design and detailing requirements of the CSA standards for each structural system.

Steel structural systems

Table 1 presents the different types of structural steel systems in the 2005 NBCC and the corresponding force modification factors, R_d and R_o. Table 1 also summarizes the corresponding design and detailing requirements of standard CSA-S16-01 (CSA 2001*a*) that must be satisfied for each system. In Table 1, steel systems of “conventional construction” with R_d = 1.5 include moment-resisting frames, braced frames, or plate walls that are designed with the non-seismic provisions of standard CSA-S16-01, except that in high-seismic regions, connections must have a ductile failure mode or be designed for increased seismic loads. Specific minimum capacity design and detailing provisions of clause 27 in CSA-S16-01 must be satisfied for systems with R_d greater than 1.5. In addition, the yield strength in ductile elements is limited to ensure a minimum level of plastic deformation, and requirements are also given to reduce the risk of brittle failures in thick plates, heavy shapes, and welds.

Figure 4 shows some of the detailing requirements for steel moment-resisting frame systems that qualify for values of R_d of 2.0 and greater. The ductility-related force modification factors for the moderately ductile and ductile systems have been increased in the 2005 NBCC (to 3.5 and 5.0 compared with 3.0 and 4.0 in the 1995 NBCC) in view of the experience gained in recent earthquakes and the more stringent detailing requirements that must now be applied (Bruneau et al. 1998; SAC 2000; Tremblay et al. 1995, 1996). For instance, robust performance of beam–column joints is essential to achieve adequate seismic response. To achieve this performance, the ability of the beam–column joints to develop minimum interstorey drifts under cyclic loading must be demonstrated by physical testing (Fig. 4*b*). Appendix J of standard CSA-S16-01 references documents that provide design and detailing rules for connections satisfying the minimum specified drift limits. For ductile moment-resisting frames with an R_d of 5.0, the columns must also be stronger than the beams. Since the beams are the energy-dissipating elements, they must be class 1 sections, whereas class 2 sections are permitted for the stronger columns. If plastic hinges are expected at the base of the structure, then the columns must be class 1. For moderately ductile frames, the interstorey drift angle capacity is reduced for beam–column joints, and class 2 beams are permitted due to the lower expected inelastic demand. For frames with limited ductility (R_d = 2.0), the performance criteria for joints are reduced further and traditional joint detailing with special welding requirements is permitted. A strong column – weak beam design is not required for this system, but columns must be class 1 sections because inelastic action is more likely to develop in these elements. However, moment-resisting frames with limited ductility are allowed only for structures up to 12 storeys located in lower seismic zones.

Moderately ductile (R_d = 3.0) and limited-ductility (R_d = 2.0) concentrically braced steel frames can dissipate energy essentially through inelastic straining in bracing members. For both systems, bracing bents must be such that the storey shear resistance provided by the tension-acting braces is similar for storey shears acting in opposite directions and that the braces can develop their yield strength in tension. Figure 5*a* shows examples of frames that meet these requirements. Limited inelastic deformations are permitted in beams of four-storey and lower chevron braced frames, provided that the beams are class 1 and their connections can carry the forces associated with beam hinging. Otherwise, columns and beams must be capable of resisting forces that correspond to yielding and buckling in the braces. Because braced frames are prone to soft-storey response with localized energy dissipation, building height restrictions are imposed to reduce the likelihood of this phenomenon (Fig. 5*a*). The limits are more restrictive when higher inelastic demand is expected (R_d = 3.0) or when the frame has reduced energy-dissipation capacity (tension-only bracing). Several detailing requirements are also prescribed for these systems, some of which are indicated in Fig. 5*b*. For instance, brace slenderness is limited to 200 in most frames to ensure minimum energy dissipation. This limit is extended to 300 for low-rise, tension-only braced frames designed with R_d = 2.0. The brace cross section must also meet maximum width-to-thickness ratios to delay the occurrence of local buckling and prevent premature brace fracture. Less severe limits are prescribed for the width-to-thickness ratio when lower inelastic demand is anticipated, i.e., for more slender braces or when limited-ductility braced frames are used in low-seismic regions. To preserve the integrity of the energy-dissipating mechanism, brace connections must resist brace loads induced by brace yielding in tension and brace buckling in compression. In addition, brace connections must be detailed for ductile rotational behaviour in the plane of buckling of the braces if high inelastic response is expected.

Figure 5*c* illustrates some of the provisions for ductile eccentrically braced steel frames (R_d = 4.0). Beam segments created by intentionally introducing eccentricity at the brace connections are expected to dissipate energy through yielding in shear or bending (Koboevic and Redwood 1997). The yielding mechanism is selected by the designer by adjusting the length of the link with consideration of the link relative flexural and shear capacities. These ductile links must be class 1 sections and must be properly braced and stiffened to maintain their capacity under reversed cyclic loading. When a link beam frames directly into a column, the connection must be capable of developing the design interstorey drift

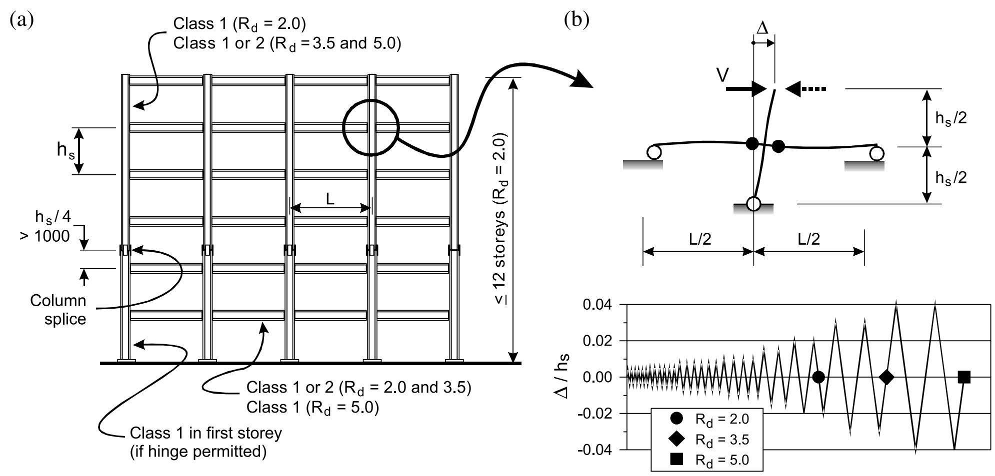
Table 1. Summary of design and detailing requirements for steel seismic force resisting systems (SFRSs).

Type of SFRS	R_d	R_o	Summary of design and detailing requirements in CSA standard CSA-S16-01
Ductile moment-resisting frames	5.0	1.5	Beams must be capable of plastic hinging without failure in connections Plastic hinging in columns permitted only at their bases, except for single-storey structures Axial load level limited to 30% of squash load in columns with plastic hinging Ductile members must be class 1 and capable of undergoing inelastic response without stability failures Limited inelastic deformations permitted in column joint panel zones if properly detailed Beam–column joints capable of developing an interstorey drift angle of 0.04 rad under cyclic loading
Moderately ductile moment-resisting frames	3.5	1.5	Same as for ductile moment-resisting frames except for the following Beams must be class 1 or 2 Axial load level limited to 50% of squash load in columns with plastic hinging Ductile elements must satisfy moderate bracing requirements Beam–column joints capable of developing an interstorey drift angle of 0.03 rad under cyclic loading
Limited-ductility moment-resisting frames	2.0	1.3	Height and seismic zone restrictions apply Beams must be class 1 or 2 Columns must be class 1 Limited inelastic deformations permitted in column joint panel zones if properly detailed Beam–column joints capable of developing an interstorey drift angle of 0.02 rad under cyclic loading or meeting minimum detailing requirements
Moderately ductile concentrically braced frames	3.0	1.5	Types of bracing limited to tension–compression, chevron, or tension-only bracing, with some configurations (e.g., knee-bracing and K-bracing) not permitted Frames with similar storey shear resistance provided by tension-acting braces in opposite directions Height restrictions apply depending on type of bracing Braces detailed to dissipate minimum energy in tension and compression, with local buckling delayed Beams, columns, and connections to resist forces induced by inelastic bracing members Brace connections designed to allow rotation from brace buckling or strengthened to develop hinging at brace ends Columns and their splices designed for secondary bending moment effects
Limited-ductility concentrically braced frames	2.0	1.3	Same as for moderately ductile concentrically braced frames except for the following Height restrictions are relaxed Braces must satisfy limited-ductility detailing for low-rise structures or low seismic zones Brace rotation capability at connections not required for slender braces and low seismic zones No minimum force level for splices in gravity columns for low seismic zones
Ductile eccentrically braced frames	4.0	1.5	Link beams must be class 1 and detailed and braced to yield in shear or flexure Link beam plastic rotational limits depend on yielding mode Beams outside of links, braces, and columns stronger than link beams Link beams to column connections must develop anticipated plastic rotation Columns and their splices designed for secondary bending moment effects
Ductile plate walls	5.0	1.6	Minimum detailing requirements for plate walls must be satisfied Beams and columns must be class 1 and capable of undergoing inelastic response without stability failures Column splices with minimum flexural and shear resistances Limited inelastic deformations permitted in column joint panel zones if properly detailed Beam-to-column connections must satisfy minimum detailing for limited-ductility moment-resisting frames

Table 1 (*concluded*).

Type of SFRS	R_d	R_o	Summary of design and detailing requirements in CSA standard CSA-S16-01
			Column bases must be stiffened and anchorage must be stronger than the columns
Limited-ductility plate walls	2.0	1.5	Minimum detailing requirements for plate walls must be satisfied Height restriction applies Walls must have factored shear and flexural resistances greater than or equal to corresponding factored loads
Conventional construction	1.5	1.3	Members and connections must have factored resistances greater than or equal to corresponding factored load effects In high seismic zones, connections must exhibit ductile failure modes or must be designed for increased seismic forces Must satisfy detailing requirements for conventional construction
Others	1.0	1.0	

Fig. 4. Steel moment-resisting frames: (*a*) summary of detailing requirements; (*b*) minimum interstorey drift requirements (CSA 2001*a*). h_s, storey height; *L*, span (centre-to-centre of column).



under cyclic loading or be reinforced so that it remains elastic. Columns, braces, and beams outside of the link segments must be stronger than the ductile links.

Limited-ductility steel plate walls (R_d = 2.0) need only meet the nonseismic provisions of standard CSA-S16-01. The web plates are designed to resist the factored storey shear forces. Beams and columns must be proportioned to resist the bending moments and axial forces induced by the factored seismic loads, including tension-field action in the web plates. Columns must be class 1 and have minimum stiffness to develop uniform tension fields in the web plates. This plate wall system is restricted, however, to buildings of 12 storeys and lower. Additional requirements are specified for ductile plate walls (R_d = 5.0), as illustrated in Fig. 6 (Kulak et al. 2001). Beams must be class 1 or 2 and must be rigidly connected to the columns. These connections must meet the provisions specified for beam–column joints in limited-ductility moment-resisting frames. The columns must be reinforced at their bases so that hinging develops at some distance above the base plates.

The derivations of the overstrength-related force modification factors, R_o, for steel structural systems are summarized in Table 2. R_{size} accounts for the fact that structural shapes or plate elements are selected by selecting the next (stronger) standard product available from the industry. It has also been shown that standard shapes have sectional properties that are typically somewhat higher than the nominal values (Schmidt and Bartlett 2002). This factor is taken as equal to 1.05 for structural shapes, based on a survey of typical structures. For the web plate of plate walls, a value of 1.10 has been chosen assuming that the plate thickness is rounded upwards to the next available plate thickness. The factor R_ϕ is taken as 1/0.9 = 1.11, as the resistance factor, ϕ, associated with ductile failure modes is equal to 0.9 in steel structures. A value of 1.10 has been adopted for R_{yield} that corresponds to the average ratio of the actual yield stress to the minimum specified yield for W shapes, as determined by Schmidt and Bartlett (2002).

The factor R_{sh}, which accounts for strain hardening, varies depending on the yielding and the level of inelastic deformation. This factor is approximately 1.3 for short links yielding in shear in eccentrically braced frames, 1.15 for plastic hinges in beams, and 1.05 in tension elements. A value of 1.15 was chosen for the ductile and moderately ductile moment-resisting frames, since both systems are designed and detailed to achieve large plastic deformations. A value of 1.05 is used for frames with limited ductility. For concentrically braced steel frames, strain hardening develops only

Fig. 5. Braced steel frames: (*a*) height limitations for concentrically braced steel frame systems; (*b*) summary of detailing requirements for concentrically braced steel frames; (*c*) summary of detailing requirements for ductile eccentrically braced steel frames (CSA 2001*a*). *K*, effective length factor; *r*, radius of gyration.

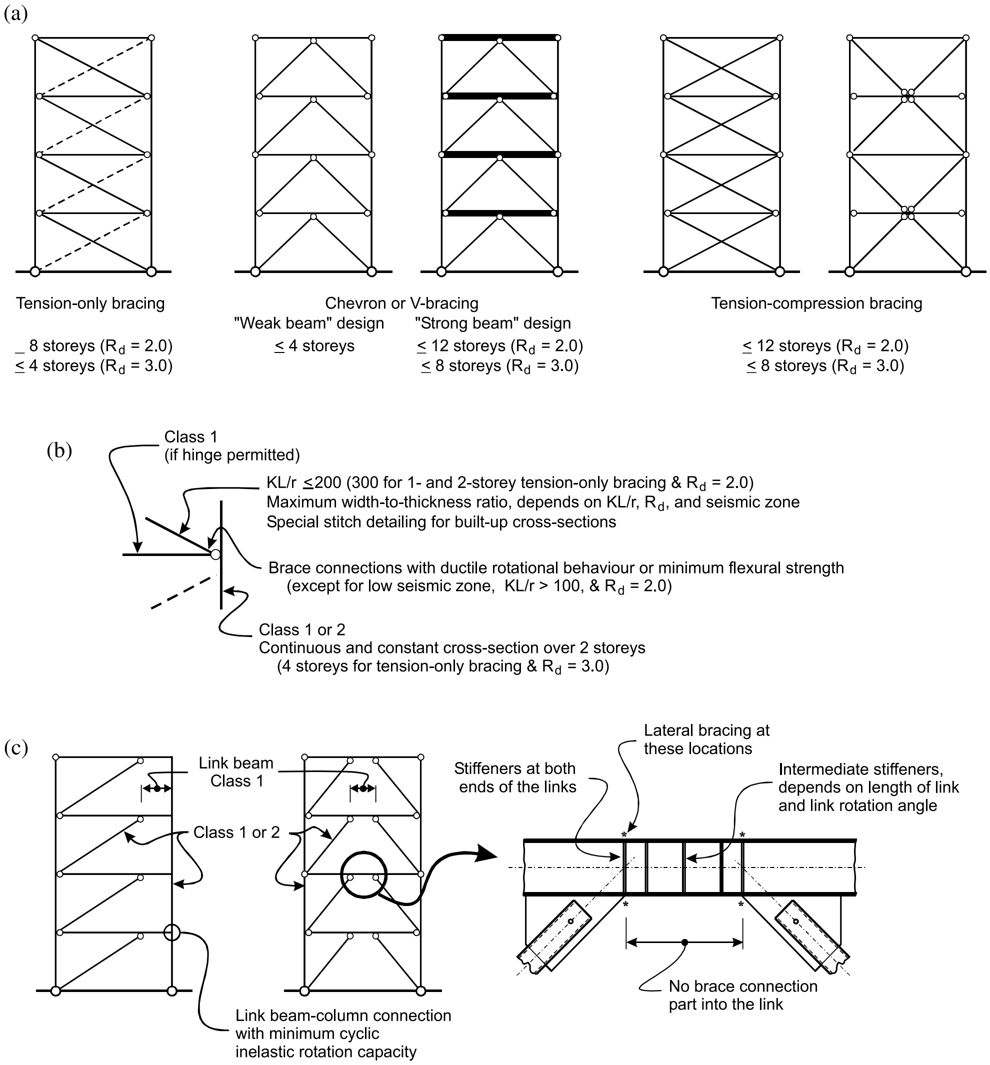
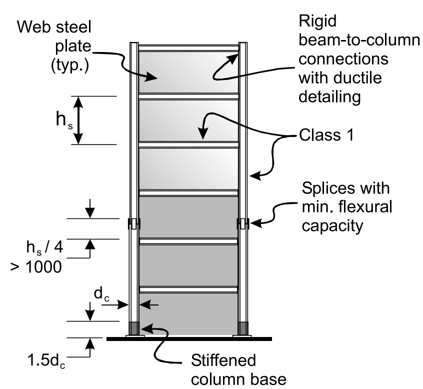


Fig. 6. Summary of detailing requirements for ductile steel plate walls (CSA 2001*a*). d_c, column dimension.



in braces yielding in tension, resulting in a value of R_{sh} equal to 1.05. For eccentrically braced frames, a conservative value of 1.15 was adopted assuming flexural yielding rather than shear yielding. In plate walls, strain hardening arises mainly from tension-field action in the plates, and a value of 1.05 was selected. For moment-resisting frames, the factor R_{mech} is greater than 1.00 when plastic hinges can form at the column bases after yielding in the beams. Since frames with pinned column bases are common in steel, the value of R_{mech} was conservatively set to 1.00. In concentrically braced steel frames, for which the braces are designed for compression forces, a reserve capacity is typically provided by the tension braces for tension–compression systems or by the compression braces for braced frames designed as tension-only systems. A conservative value of 1.00 was adopted for R_{mech}, however, to account for the strength degradation of the compression braces under reversed cyclic loading. For low-rise buildings with tension-only bracing, the use of very slender braces exhibiting negligible compression strength for limited-ductility braced frames is permitted. Therefore, R_{mech} is equal to 1.00 for that category. An R_{mech} value of 1.00 is also prescribed for eccentrically braced steel frames because a collapse mechanism is formed after yielding of the beam link segments. In plate walls, the compression strut that develops in the web plate and the ele-

Table 2. Derivation of overstrength-related force modification factors for steel seismic force resisting systems (SFRSs).

Type of SFRS	Calculation of R_o						NBCC R_o
	R_{size}	R_ϕ	R_{yield}	R_{sh}	R_{mech}	R_o	
Ductile moment-resisting frames	1.05	1.11	1.10	1.15	1.00	1.47	1.5
Moderately ductile moment-resisting frames	1.05	1.11	1.10	1.15	1.00	1.47	1.5
Limited-ductility moment-resisting frames	1.05	1.11	1.10	1.05	1.00	1.35	1.3
Moderately ductile concentrically braced frames	1.05	1.11	1.10	1.05	1.00	1.35	1.3
Limited-ductility concentrically braced frames	1.05	1.11	1.10	1.05	1.00	1.35	1.3
Ductile eccentrically braced frames	1.05	1.11	1.10	1.15	1.00	1.47	1.5
Ductile plate walls	1.10	1.11	1.10	1.10	1.10	1.63	1.6
Limited-ductility plate walls	1.10	1.11	1.10	1.05	1.05	1.48	1.5
Conventional construction	1.05	1.11	1.10	1.00	1.00	1.28	1.3

Table 3. Summary of design and detailing requirements for reinforced concrete seismic force resisting systems (SFRSs).

Type of SFRS	R_d	R_o	Summary of design and detailing requirements in CSA standard CSA-A23.3-94
Ductile moment-resisting frames	4.0	1.7	Beams capable of flexural hinging with shear failure and bar buckling avoided Beams and columns must satisfy ductile detailing requirements Columns properly confined and stronger than beams Joints properly confined and capable of transmitting shears from beam hinging
Moderately ductile moment-resisting frames	2.5	1.4	Beams and columns must satisfy detailing requirements for moderate ductility Beams and columns to have minimum shear strengths Joints must satisfy moderate ductility detailing requirements and must be capable of transmitting shears from beam hinging
Moment-resisting frames with conventional construction	1.5	1.3	Beams and columns must have factored resistances greater than or equal to factored loads Beams and columns must satisfy design and detailing requirements for conventional construction Joints must have factored shear resistances greater than or equal to shears from factored loads
Ductile coupled walls	4.0	1.7	At least 66% of base overturning moment resisted by wall system must be carried by axial tension and compression in coupled walls Coupling beams to have ductile detailing and be capable of flexural hinging or ductile diagonal reinforcement (shear failure and bar buckling avoided) Walls to have minimum resistance to permit attainment of nominal strength in coupling beams and minimum ductility level
Ductile partially coupled walls	3.5	1.7	Coupling beams to have ductile detailing and be capable of flexural hinging or ductile diagonal reinforcement (shear failure and bar buckling avoided) Walls to have minimum resistance to permit attainment of nominal strength in coupling beams and minimum ductility level
Ductile shear walls	3.5	1.6	Walls capable of flexural hinging without local instability, shear failure, or bar buckling Walls must satisfy ductile detailing and ductility requirements
Moderately ductile shear walls	2.0	1.4	Walls must satisfy detailing and ductility requirements for moderate ductility Walls must have minimum shear strength
Shear walls with conventional construction	1.5	1.3	Walls must have factored shear and flexural resistances greater than or equal to corresponding factored loads Walls must satisfy detailing requirements for conventional construction
Others	1.0	1.0	

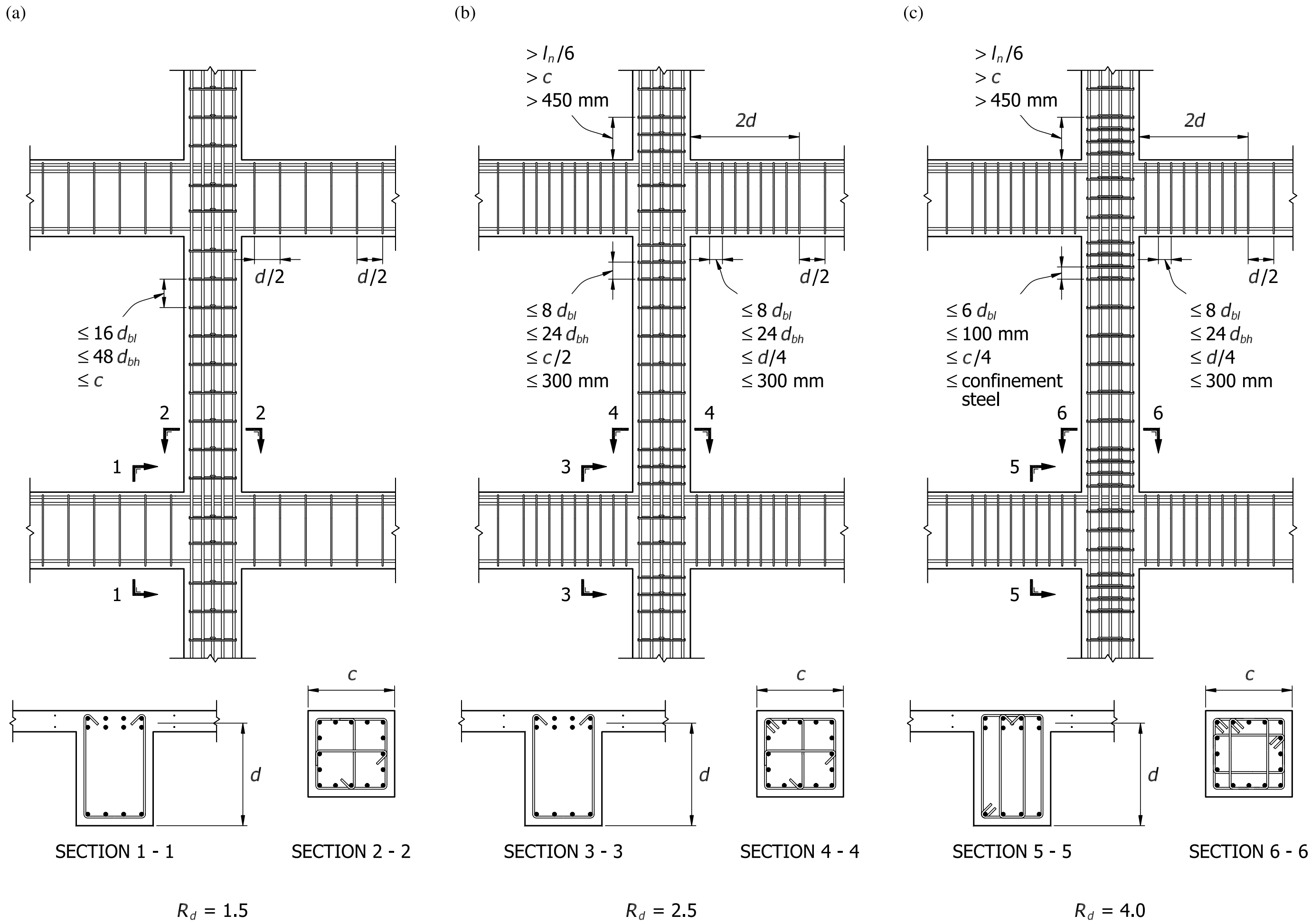
ments of the moment-resisting frame provide additional lateral resistance to the system. Values of 1.10 and 1.05 were adopted for R_{mech} in ductile walls and walls with limited ductility, respectively, to account for this behaviour.

Reinforced concrete structural systems

Table 3 gives the different types of reinforced concrete structural systems in the 2005 NBCC and the corresponding force modification factors, R_d and R_o. Table 3 also summarizes the corresponding design and detailing requirements of standard CSA-A23.3-94 (CSA 1994*a*) that must be satisfied for each system.

Figure 7 illustrates some of the detailing requirements for reinforced concrete frame systems. In Fig. 7, d_{b1} refers to

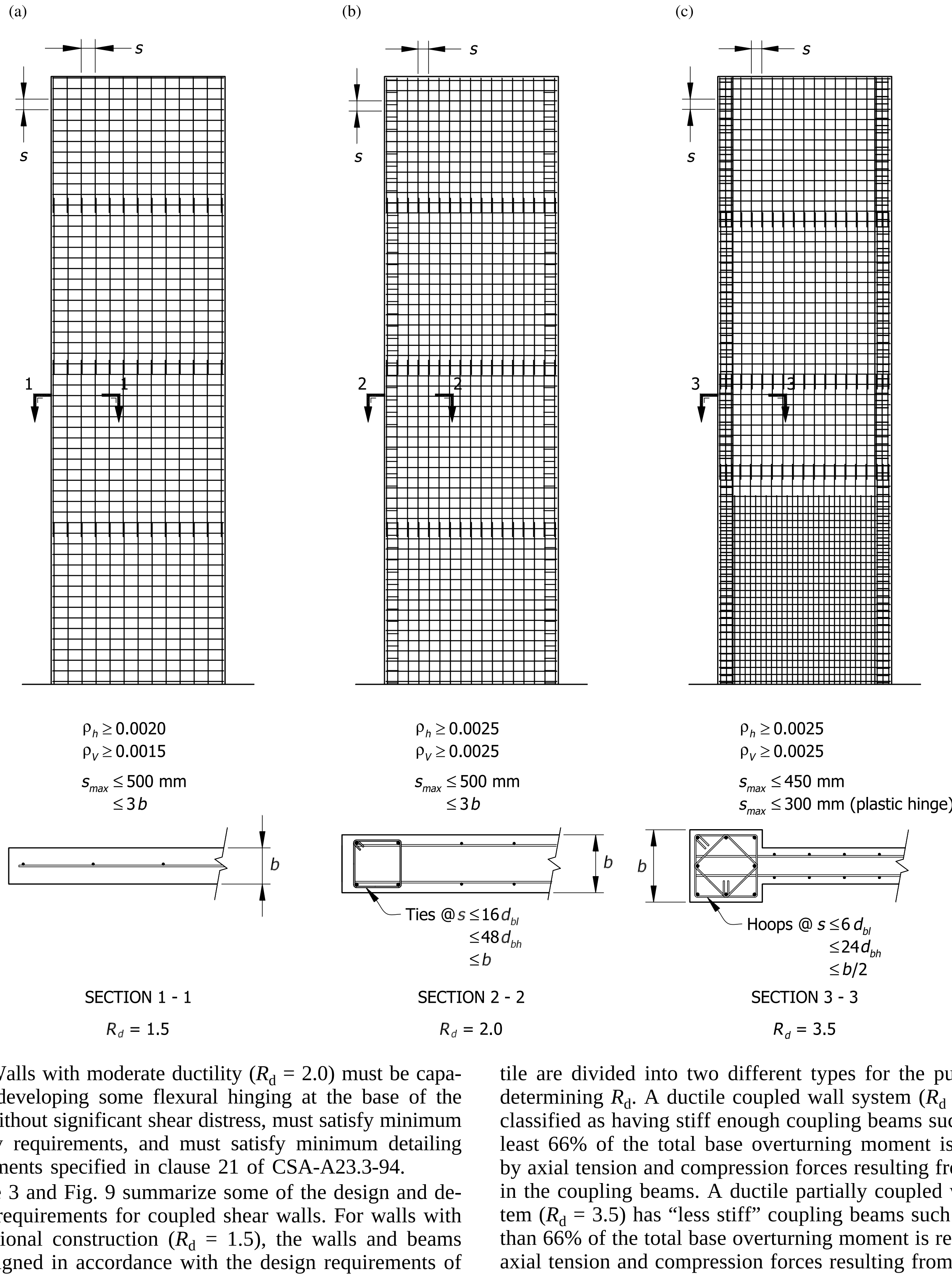
Fig. 7. Summary of detailing requirements for reinforced concrete moment-resisting frames (CSA 1994*a*): (*a*) R_d = 1.5; (*b*) R_d = 2.5; (*c*) R_d = 4.0. *c*, depth of the flexural compressive zone; *d*, effective depth of the wall; d_{bh}, diameter of the horizontal reinforcing bars; d_{bl}, diameter of the longitudinal reinforcing bars; l_n, clear height of column.



the diameter of the longitudinal bars and d_{bh} refers to the diameter of the column ties or hoops. The system with conventional construction (R_d = 1.5) typically has lap splices in the vertical bars at the floor levels, with the beams and columns designed and detailed in accordance with the nonseismic provisions of clauses 1–18 of standard CSA-A23.3-94. In contrast, the members of ductile moment-resisting frames (R_d = 4.0) are designed using capacity design procedures (Table 3) to ensure that the columns are stronger than the beams and that no brittle shear or bond failures occur. In addition, very stringent detailing requirements must be satisfied (Fig. 7*c*), to provide the necessary levels of concrete confinement in the beams, columns, and joints and to delay the onset of buckling of the longitudinal reinforcing bars (diameter of d_{bl}). The requirements of the American Concrete Institute code (ACI 1983) were adopted for the design and detailing of ductile frame members (CSA 1984, 1994*a*). Moment-resisting frames with moderate ductility (R_d = 2.5) must be designed using capacity design and detailing requirements that are not as stringent as those for R_d = 4.0 (Table 3; Fig. 7). The suitability of the requirements for moderate ductility was confirmed by results from reversed cyclic loading tests on full-scale beam–slab–column subassemblages (Paultre et al. 1989). Both the moderately ductile and ductile moment-resisting frame systems must satisfy the more stringent design and detailing requirements of clause 21 of standard CSA-A23.3-94.

Table 3 summarizes some of the design requirements, and Fig. 8 illustrates some of the detailing requirements of the CSA standard for shear walls. The wall with conventional construction (R_d = 1.5) typically has lap splices in the vertical bars at the floor levels, with the uniformly distributed and concentrated reinforcement satisfying the design and detailing requirements of the nonseismic provisions of clauses 10, 11, and 14 of standard CSA-A23.3-94. In contrast, the ductile walls (R_d = 3.5) must satisfy the more stringent design and detailing requirements of clause 21 of CSA-A23.3-94, which are based on the requirements in the New Zealand standard (NZS 1982). These provisions include minimum reinforcement limits for the uniformly distributed reinforcement and concentrated reinforcement, minimum ductility requirements, and detailing requirements, particularly in the region of expected plastic hinging. The walls must be capable of developing plastic hinging at their bases without significant shear distress, without lateral buckling of the compression zone, and with limited bar buckling in the regions of concentrated reinforcement. No more than 50% of the vertical reinforcement may be lap spliced at any one

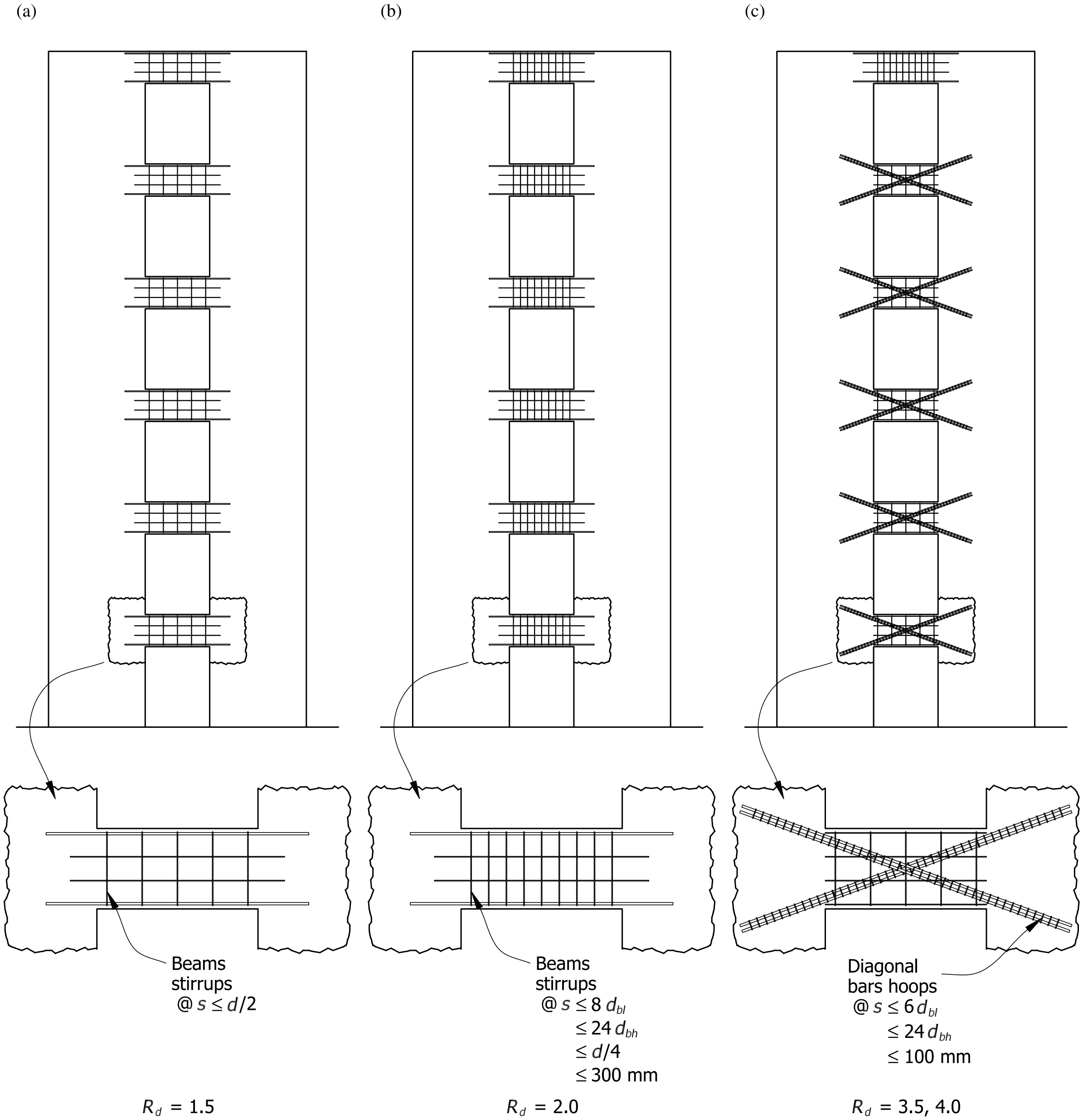
Fig. 8. Summary of detailing requirements for reinforced concrete shear walls: (*a*) R_d = 1.5; (*b*) R_d = 2.0; (*c*) R_d = 3.5 (CSA 1994*a*). *b*, wall thickness; *s*, bar spacing; s_{max}, maxumum bar spacing; ρ_h, reinforcement ratio (uniformly distributed horizontal bars); ρ_v, reinforcement ratio (uniformly distributed vertical bars).



level. Walls with moderate ductility (R_d = 2.0) must be capable of developing some flexural hinging at the base of the walls without significant shear distress, must satisfy minimum ductility requirements, and must satisfy minimum detailing requirements specified in clause 21 of CSA-A23.3-94.

Table 3 and Fig. 9 summarize some of the design and detailing requirements for coupled shear walls. For walls with conventional construction (R_d = 1.5), the walls and beams are designed in accordance with the design requirements of clauses 1–18 of standard CSA-A23.3-94. The coupled walls with moderate ductility (R_d = 2.0) must have the walls and beams designed in accordance with clause 21.9 for moderate ductility (Fig. 9*b*). Coupled walls that are classified as ductile are divided into two different types for the purpose of determining R_d. A ductile coupled wall system (R_d = 4.0) is classified as having stiff enough coupling beams such that at least 66% of the total base overturning moment is resisted by axial tension and compression forces resulting from shear in the coupling beams. A ductile partially coupled wall system (R_d = 3.5) has “less stiff” coupling beams such that less than 66% of the total base overturning moment is resisted by axial tension and compression forces resulting from shear in the coupling beams. These two systems must have the walls interconnected by ductile coupling beams. Coupling beams having significantly high shear stresses and relatively small ratios of beam span to beam depth must be reinforced with

Fig. 9. Summary of detailing requirements for reinforced concrete coupled walls: (*a*) R_d = 1.5, (*b*) R_d = 2.0; (*c*) R_d = 3.5 or 4.0 (CSA 1994*a*).



well-confined diagonal reinforcement (Fig. 9*c*). Other coupling beams may be reinforced with the ductile beam details for frame members (Fig. 7*c*). The coupling beams are the energy-dissipating elements, and hence care must be taken to ensure that significant ductility and energy dissipation can occur in these elements. The walls must satisfy minimum ductility levels and minimum detailing requirements for both the uniformly distributed reinforcement and the concentrated reinforcement. Examples of the seismic design of a ductile moment-resisting frame and a coupled wall structure are given by Mitchell et al. (1995).

The values of the overstrength-related force modification factors, R_o, for concrete structural systems are given in Table 4. The component R_{size} accounts for the fact that designers choose reinforcing bars that are available and hence often provide an excess of steel. In addition, bar spacings are usually rounded downwards and member sizes are rounded upwards. To account for these factors R_{size} has been assumed to be 1.05. The factor R_ϕ is taken as $1/\phi_s$, since for many members the strength is governed by yielding of the reinforcement. The resistance factor for reinforcing bars, ϕ_s, is 0.85, and hence R_ϕ is 1.18. Although the actual average reinforcing bar yield is somewhat above the specified value (Mirza and MacGregor 1979), a conservative value of 1.05 was assumed for R_{yield}, since this effect seems to be less pronounced for the larger bar sizes. The component R_{sh} accounts for the development of strains well into strain hardening, resulting in stresses above the yield stress. This effect is significant for ductile elements that have excellent confinement of the concrete and prevention of premature buckling of the longitudinal bars. Also, the reinforcement for systems designed with R_d greater than 2.0 must be con-

Table 4. Derivation of overstrength-related force modification factors for reinforced concrete seismic force resisting systems (SFRSs).

Type of SFRS	Calculation of R_o						NBCC R_o
	R_{size}	R_ϕ	R_{yield}	R_{sh}	R_{mech}	R_o	
Ductile moment-resisting frames	1.05	1.18	1.05	1.25	1.05	1.71	1.7
Moderately ductile moment-resisting frames	1.05	1.18	1.05	1.10	1.00	1.43	1.4
Moment-resisting frames with conventional construction	1.05	1.18	1.05	1.00	1.00	1.30	1.3
Ductile coupled walls	1.05	1.18	1.05	1.25	1.05	1.71	1.7
Ductile partially coupled walls	1.05	1.18	1.05	1.25	1.05	1.71	1.7
Ductile shear walls	1.05	1.18	1.05	1.25	1.00	1.63	1.6
Moderately ductile shear walls	1.05	1.18	1.05	1.10	1.00	1.43	1.4
Shear walls with conventional construction	1.05	1.18	1.05	1.00	1.00	1.30	1.3

Table 5. Summary of design and detailing requirements for timber seismic force resisting systems (SFRSs).

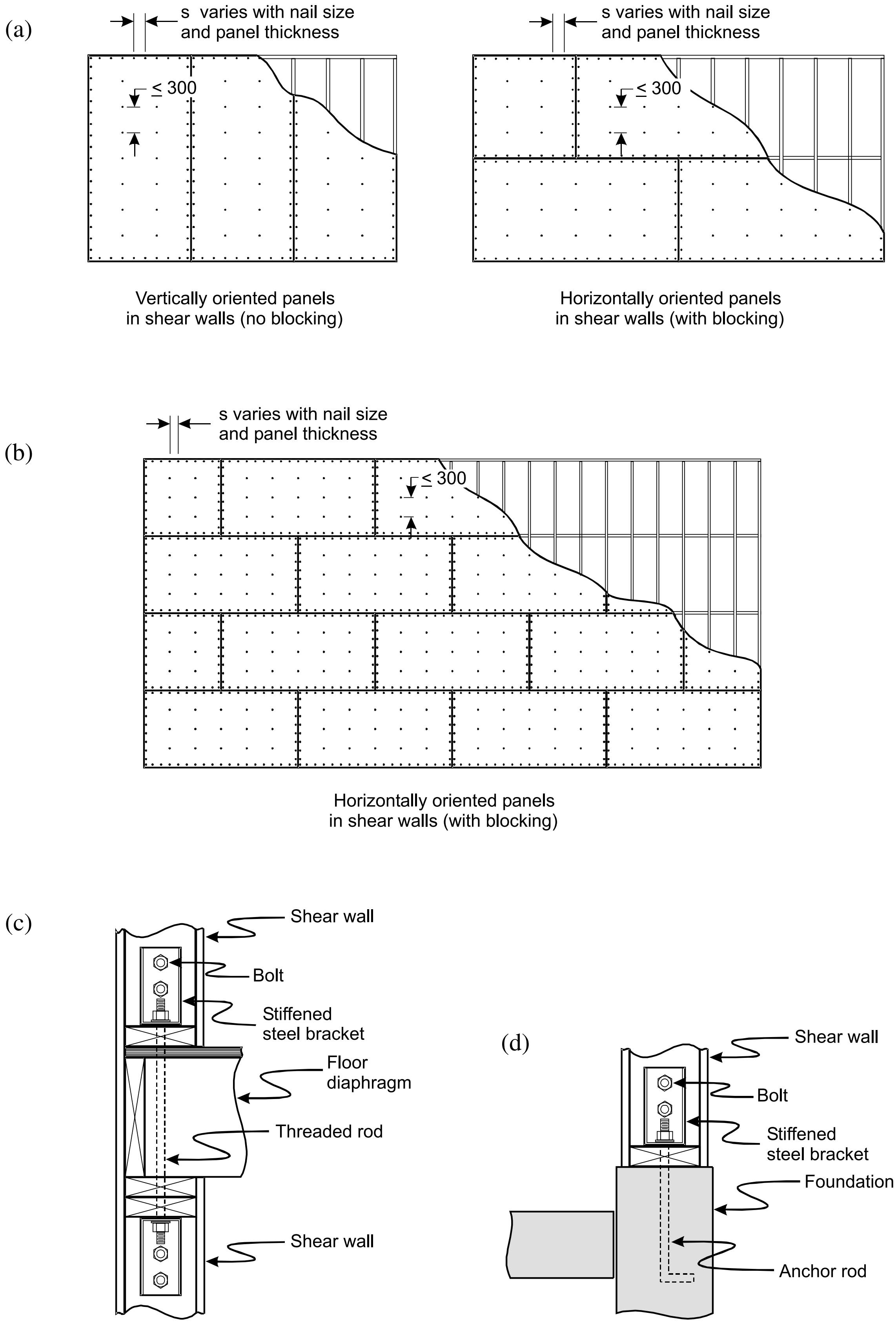
Type of SFRS	R_d	R_o	Summary of design and detailing requirements in CSA standard CSA-O86-01
Nailed shear walls with wood-based panels	3.0	1.7	Nailed wood based panels such as plywood, oriented strand board (OSB), and waferboard must be sized and fastened to provide a factored shear resistance equal to or greater than the factored shear force Minimum size and maximum spacing must be satisfied for framing members Nails must be used with maximum spacings at panel edges and at intermediate framing members, and minimum edge distance must be provided Perimeter members must resist axial forces and be adequately connected and spliced
Shear walls with wood-based and gypsum panels in combination	2.0	1.7	A spatially balanced combination of nailed wood based panels mixed with nailed or screwed gypsum wallboard panels must be sized and fastened to provide a factored shear resistance equal to or greater than the factored shear The amount of wood-based panels provided in each storey must be such that they resist a minimum percentage of the total storey shear The storey height is limited to 3.6 m Gypsum wallboard must conform to type X (fire-rated)
Moderately ductile braced or moment-resisting frames	2.0	1.5	Members and connections to be sized and detailed such that the factored resistance equals or exceeds the factored load Concentrically braced frames or moment-resisting frames must have ductile connections such as connections made with timber (glulam) rivets designed in rivet yielding mode
Limited-ductility braced or moment-resisting frames	1.5	1.5	Members and connections to be sized and detailed such that the factored resistance equals or exceeds the factored load Connections with limited ductility such as bolted connections with a small ratio of wood member thickness to bolt diameter
Other wood- or gypsum-based SFRSs	1.0	1.0	

structed with weldable-grade reinforcement. Since the weldable-grade steel has a tensile strength of at least 1.25 times the actual yield stress, R_{sh} is taken as 1.25 only for the ductile cases and 1.10 for the moderately ductile cases. As discussed earlier, the component R_{mech} accounts for the beneficial effects of the hierarchy of yielding in assessing the collapse mechanism that could form. This factor is dependant on the ratio, β, of the strength of the columns to the strength of the interconnecting beams and on the number of storeys. For ductile moment-resisting frame and coupled wall structures, the factor β is 1.38, and hence R_{mech} is taken as 1.05 for structures greater than four storeys, as shown in Fig. 3*b*.

Timber structural systems

Table 5 gives the different types of timber systems in the 2005 NBCC and the corresponding force modification factors R_d and R_o. Table 5 also summarizes the corresponding design and detailing requirements of standard CSA-O86-01 (CSA 2001*b*) that must be satisfied for each system. Experimental research and experience from past earthquakes have demonstrated that properly connected wood-based shear walls exhibit good ductility and energy dissipation (Rainer and Karacabeyli 2000). Inelastic deformations arise from both bending of the nails and local bearing deformations in the timber around the nails. In addition, the light weight of wood structures results in smaller inertia forces. Figure 10

Fig. 10. Summary of detailing requirements for nailed wood-based construction (CSA 2001*b*): (*a*) shear walls; (*b*) example of diaphragm with blocking; (*c*) hold down between floors; (*d*) hold down with anchor bolts.



illustrates some of the detailing requirements for wood-based panels. The nail spacings and sizes are chosen to provide the required shear strength for the panel. Additional nail spacings are also prescribed. In addition, the floor and roof diaphragms must be designed to resist the required diaphragm forces, and adequate connections between the diaphragms and the wall panels must be provided. Figures 10*c* and 10*d* illustrate typical detailing for providing tension resistance between two storeys and at the foundation level.

Connections made with nails or screws in gypsum wallboard shear walls are not as ductile as connections in wood-based panels because of the local distress of the gypsum in the vicinity of the fasteners when a panel is subjected to shear. Testing and analyses have shown (Ceccotti and Karacabeyli 2002), however, that a mix of gypsum wallboard and wood-based panels can be used in structures to attain a minimum ductility-related force modification factor R_d of 2.0, provided that the wood-based panels resist a minimum percentage of shear in each storey (Fig. 11). In addition to the design provisions related to the use of gypsum wallboard, the 2001 version of standard CSA-O86-01 contains alternate design procedures for determining the lateral load capacity of shear wall segments with and without hold-down connectors (Ni and Karacabeyli 2000). It also contains strength adjustment factors for unblocked shear walls, revised species factors for framing material, increased capacities for anchor bolts, and a conversion formula for power-driven nails (Karacabeyli and Ni 2001).

The inelastic response of concentrically braced frames and moment-resisting frames made of timber depends almost entirely on the ductility of the connections. Hence, there are two categories for these structural systems. The first category includes frames with connections that have moderate ductility, such as connections with timber (glulam) rivets designed in rivet yielding mode (Popovski et al. 2002). The second category includes connections with limited ductility, such as bolted connections with a small ratio of wood member thickness to bolt diameter (Popovski et al. 1999).

Table 6 gives the overstrength-related force modification factors R_o for timber structural systems. For wood-based panels, considerable inelastic deformations arise from the high local bearing stresses in the wood surrounding the nails. Therefore, the factor R_ϕ is taken as $1/\phi$, where the resistance factor for wood is 0.7. This results in an R_ϕ value of 1.43. For braced or moment-resisting frames with moderate ductility, ductile connections must be provided. Glulam rivets and lag screws are designed with a value of ϕ_{wood} equal to 0.6, resulting in an R_ϕ value of 1.66. Bolted connections and drift pins, however, are designed with a ϕ_{wood} value of 0.7, and hence a minimum or dependable value for R_ϕ is 1.43 for these connections. Since these different types of connections are used in braced frames, a conservative value of 1.43 was chosen for R_ϕ. For all of the timber structural systems, a conservative value of R_{yield} of 1.0 was assumed. For the case of nailed connections of wood-based panels, a factor of 1.05 was used for R_{sh}, based on evidence from full-scale panel tests under reversed cyclic loading (Rainer and Karacabeyli 1999). For the other cases, no strain-hardening effect was included, since large inelastic deformations may not develop in all connections. The component, R_{size}, accounts for the fact that designers choose practical connector

Fig. 11. Minimum percentage of storey shear resisted by wood-based panels in shear walls with wood-based and gypsum panels in combination (CSA 2001*b*).

spacings and must choose connectors from available products (CWC 2001). This results in some overdesign of the connections. This is particularly important for connections of wood-based panels that have prescribed spacings and nail sizes to achieve the desired shear force levels and in addition have maximum spacing limits for the connectors. A factor of 1.15 was considered to be reasonable for these cases, whereas a factor of 1.05 was chosen for other connection types. Because capacity design procedures have not yet been implemented for the design of timber structures, a value 1.00 was chosen for R_{mech}.

Masonry structural systems

Table 7 gives the different types of masonry structural systems in the 2005 NBCC and the corresponding force modification factors R_d and R_o. Table 7 also summarizes the design and detailing requirements of standard CSA-S304.1-94, *Masonry design for buildings* (*limit states design*) (CSA 1994*b*). Further information on the requirements and application of the CSA standard is given by Glanville et al. (1996). The seismic behaviour and design of masonry structures are presented by Paulay and Priestley (1992).

Figure 12 illustrates some of the detailing requirements for masonry shear walls. The unreinforced masonry walls have a ductility-related force modification factor R_d of 1.00 and an overstrength-related force modification factor R_o of 1.00. These low factors signify no ductility and no dependable overstrength and were chosen because of the poor performance of unreinforced masonry walls in actual earthquakes and the fact that many such walls fail in the direction perpendicular to the plane of the walls due to the weak joints. Unreinforced masonry construction has not been permitted for use in structures situated in moderate to high seismic zones in Canada since 1980 (NBCC 1980).

Reinforced masonry shear walls with limited ductility must contain minimum amounts of both horizontal and vertical uniformly distributed reinforcement (Fig. 12*b*). The total amount of vertical reinforcement in the walls is also limited to 2% of the gross area of the wall. Reinforcement equivalent to at least one No. 15 bar must be provided around each panel and each opening. The walls must be designed for flexure and shear, including sliding shear.

The details of the uniformly distributed reinforcement in reinforced masonry shear walls with moderate ductility (re-

Table 6. Derivation of overstrength-related force modification factors for timber seismic force resisting systems (SFRSs).

Type of SFRS	Calculation of R_o						NBCC R_o
	R_{size}	R_ϕ	R_{yield}	R_{sh}	R_{mech}	R_o	
Nailed shear walls with wood-based panel	1.15	1.43	1.00	1.05	1.00	1.73	1.7
Shear walls with wood-based and gypsum panels in combination	1.15	1.43	1.00	1.05	1.00	1.73	1.7
Moderately ductile braced or moment-resisting frames	1.05	1.43	1.00	1.00	1.00	1.50	1.5
Limited-ductility braced or moment-resisting frames	1.05	1.43	1.00	1.00	1.00	1.50	1.5

Table 7. Summary of design and detailing requirements for masonry seismic force resisting systems (SFRSs).

Type of SFRS	R_d	R_o	Summary of design and detailing requirements in CSA standard CSA-S304.1-94
Moderately ductile shear walls	2.0	1.5	Walls to be designed to resist factored moment resistance and exhibit minimum plastic hinging without shear failure and local buckling Sliding shear failure at joints to be avoided Minimum ductility level required Seismic detailing requirements for moderate ductility must be satisfied In plastic hinge region, only 50% of vertical bars to be lapped and all voids to be filled
Limited-ductility shear walls	1.5	1.5	Same as shear walls with moderate ductility except with relaxation of reinforcement detailing
Shear walls with conventional construction	1.5	1.5	Walls must have factored shear and flexural resistances greater than or equal to corresponding factored loads Detailing requirements for minimum seismic reinforcement must be satisfied
Moment-resisting frames with conventional construction	1.5	1.5	Columns and beams must have factored shear and flexural resistances greater than or equal to corresponding factored loads Columns to satisfy minimum detailing requirements for vertical reinforcement and lateral ties Beams to satisfy minimum detailing requirements for longitudinal reinforcement
Unreinforced masonry	1.0	1.0	Unreinforced walls and columns must have factored shear and flexural resistances greater than or equal to corresponding factored loads
Others	1.0	1.0	

ferred to as nominally ductile in the 1994 CSA standard) are similar to those in shear walls with limited ductility, but the moderate-ductility shear walls have an additional requirement for the maximum spacing of the vertical bars of $d/4$, where d is the effective depth of the wall. In the region of expected plastic hinging (Fig. 12*c*) the voids in the masonry must be grouted, and open-ended blocks must be used if the masonry is laid in stack pattern. The slenderness ratio of the wall is limited in the region of the compression zone to prevent local instability. The shear resistances contributed by the masonry and arising from the axial compressive load are reduced by one half in the plastic hinge region. The sliding shear resistance is also reduced in the plastic hinge region. A minimum level of flexural ductility is prescribed by limiting the depth, c, of the flexural compressive zone to 0.2 times the wall length. The horizontal reinforcement must be effectively continuous (restrictions on lap locations) to the ends of the walls and must be anchored around vertical bars at the ends of the walls with 180° hooks.

Figure 12*c* also shows the detailing required for the proposed new case for limited-ductility shear walls with R_d = 1.5. It is noted that there is relaxation in the requirements for the length of the plastic hinge, lapping of the vertical reinforcement, and anchorage of the horizontal reinforcement. Unlike the case for moderate ductility, there is no reduction in the shear carried by the masonry in the plastic hinge region.

Figure 13 illustrates the required detailing of the reinforcement in masonry frame construction with limited ductility. In columns there are minimum and maximum limits for the amount of vertical reinforcement and maximum spacing limits for the lateral ties. The beam steel has maximum and minimum limits and spacing limits for the uniformly distributed reinforcement in deeper beams.

Table 8 gives the components of overstrength contributing to R_o. The unreinforced masonry structures systems are assigned an R_o value of 1.00. For the reinforced masonry systems the factor R_ϕ is taken as $1/\phi_s$, since for many members the strength is governed by yielding of the reinforcement. The resistance factor for the reinforcement, ϕ_s, is 0.85, and hence R_ϕ is 1.18. Because the principal reinforcement consists of smaller bar sizes than in conventional reinforced concrete structures, it was assumed that the actual average reinforcing bar yield is 1.1 times the minimum specified yield strength. This results in an R_{yield} value of 1.1. Because of the limited ductility of reinforced masonry, an R_{sh} value

Fig. 12. Summary of detailing requirements for masonry shear walls (CSA 1994*b*): (*a*) R_d = 1.0; (*b*) R_d = 1.5; (*c*) R_d = 2.0. s_h, spacing of horizontal bars; s_v, spacing of vertical bars; α, reinforcement distribution factor.

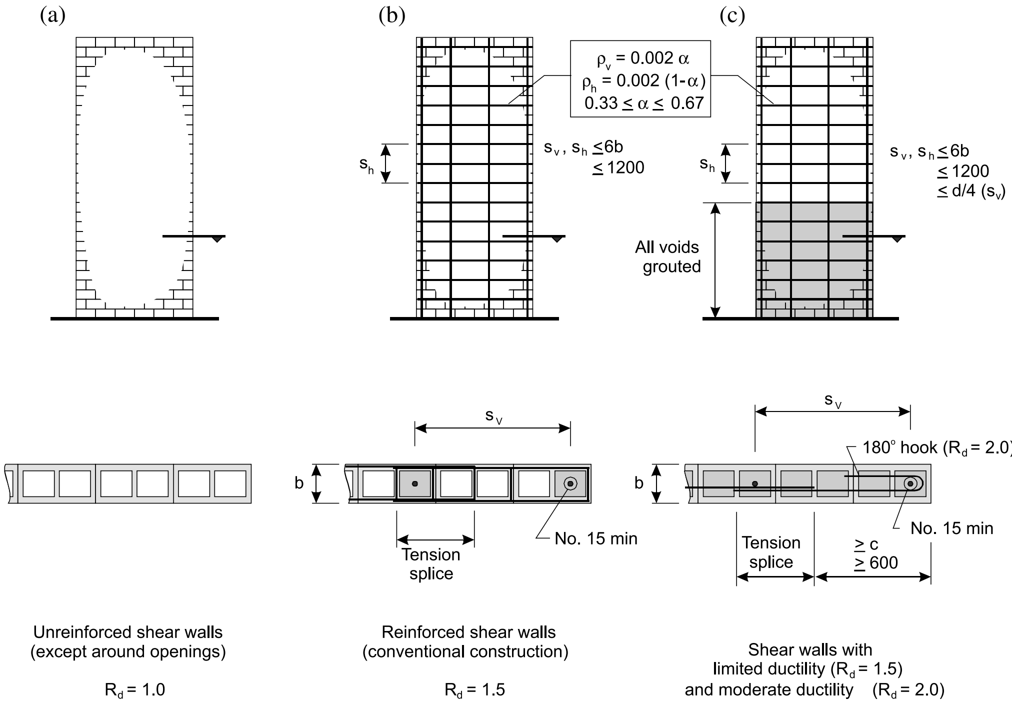
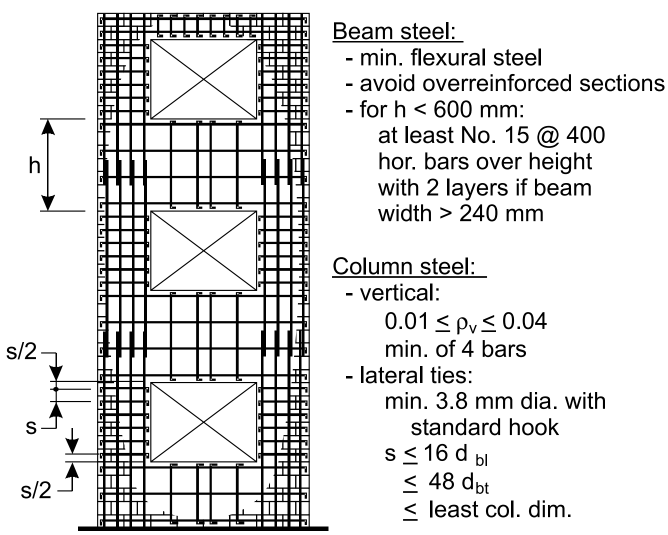


Fig. 13. Summary of detailing requirements for masonry moment-resisting frames (CSA 1994*b*). d_{bt}, diameter of lateral ties.



of 1.00 was chosen because no significant strain hardening of the reinforcement is expected. The component R_{size} is due to the fact that the masonry blocks and reinforcing bars are available in standard sizes, often leading to greater resistance than that required. Also, the minimum reinforcement details, maximum spacing limits, and limited locations for placing the reinforcing bars (in grouted cells) often lead to capacities above the required values. Hence a value of 1.15 was chosen for these reinforced masonry cases. Since there is no hierarchy of yielding, the R_{mech} value was chosen as 1.00.

Restrictions on structural systems

Table 9 gives the restrictions on the use of different structural systems as a function of the magnitude of $I_E F_a S_a(0.2)$ and $I_E F_v S_a(1.0)$, where F_a and F_v are the acceleration- and velocity-based site coefficients, respectively; and $S_a(T)$ is the 5% damped spectral response acceleration expressed as a ratio to gravitational acceleration for a period *T*. The most ductile systems have no limit (NL) on the building height, and some structural systems with moderate and limited ductility have limits on the height of the building. Structural systems that have demonstrated poor performance in major earthquakes are not permitted (NP) in significant seismic regions.

Future changes to the CSA standards

It is noted that the design and detailing requirements given in this paper correspond to those in the CSA standards at the time this paper was prepared. Once the 2005 NBCC is finalized, it is expected that some of the CSA standards may be revised. Designers are cautioned that, although this paper provides some guidelines for design and detailing requirements, the latest CSA standards must always be used.

Conclusions

The methodology and background for selecting the proposed values for the ductility- and overstrength-related seismic force modification factors for the different seismic force resisting systems (SFRSs) proposed in the 2005 NBCC are described. A major change from the 1995 NBCC is the introduction of an overstrength-related force modification factor and the elimination of the calibration factor *U*.

Table 8. Derivation of overstrength-related force modification factors for masonry seismic force resisting systems (SFRSs).

Type of SFRS	Calculation of R_o						NBCC R_o
	R_{size}	R_{ϕ}	R_{yield}	R_{sh}	R_{mech}	R_o	
Moderately ductile shear walls	1.15	1.18	1.10	1.00	1.00	1.49	1.5
Limited-ductility shear walls	1.15	1.18	1.10	1.00	1.00	1.49	1.5
Shear walls with conventional construction	1.15	1.18	1.10	1.00	1.00	1.49	1.5
Moment-resisting frames with conventional construction	1.15	1.18	1.10	1.00	1.00	1.49	1.5
Unreinforced masonry	1.00	1.00	1.00	1.00	1.00	1.00	1.0

Table 9. SFRS ductility-related force modification factors (R_d), overstrength-related force modification factors (R_o), and general restrictions.

Type of SFRS	R_d	R_o	Restrictions				
			$I_EF_aS_a(0.2)$				
			<0.20	≥0.20 to <0.35	≥0.35 to ≤0.75	>0.75	$I_EF_vS_a(1.0)$ > 0.30
Steel structures designed and detailed according to CSA standard CSA-S16-01							
Ductile moment-resisting frames	5.0	1.5	NL	NL	NL	NL	NL
Moderately ductile moment-resisting frames	3.5	1.5	NL	NL	NL	NL	NL
Limited-ductility moment-resisting frames	2.0	1.3	NL	NL	60	NP	NP
Moderately ductile concentrically braced frames							
Tension–compression bracing	3.0	1.3	NL	NL	40	40	40
Tension-only bracing	3.0	1.3	NL	NL	20	20	20
Limited-ductility concentrically braced frames							
Tension–compression bracing	2.0	1.3	NL	NL	60	60	60
Tension-only bracing	2.0	1.3	NL	NL	60	60	60
Chevron bracing	2.0	1.3	NL	NL	40	40	40
Ductile eccentrically braced frames	4.0	1.5	NL	NL	NL	NL	NL
Ductile plate walls	5.0	1.6	NL	NL	NL	NL	NL
Moderately ductile plate walls	2.0	1.5	NL	NL	60	60	60
Conventional construction	1.5	1.3	NL	NL	15	15	15
Other steel SFRS(s) not defined previously	1.0	1.0	15	15	NP	NP	NP
Concrete structures designed and detailed according to CSA standard CSA-A23.3-94 (2004 edition under preparation)							
Ductile moment-resisting frames	4.0	1.7	NL	NL	NL	NL	NL
Moderately ductile moment-resisting frames	2.5	1.4	NL	NL	60	40	40
Ductile coupled walls	4.0	1.7	NL	NL	NL	NL	NL
Ductile partially coupled walls	3.5	1.7	NL	NL	NL	NL	NL
Ductile shear walls	3.5	1.6	NL	NL	NL	NL	NL
Moderately ductile shear walls	2.0	1.4	NL	NL	NL	60	60
Conventional construction							
Moment-resisting frames	1.5	1.3	NL	NL	15	NP	NP
Shear walls	1.5	1.3	NL	NL	40	30	30
Other concrete SFRS(s) not listed previously	1.0	1.0	15	15	NP	NP	NP
Timber structures designed and detailed according to CSA standard CSA-O86-01							
Shear walls							
Nailed shear walls with wood-based panels	3.0	1.7	NL	NL	30	20	20
Shear walls with wood-based and gypsum panels in combination	2.0	1.7	NL	NL	20	20	20
Braced or moment-resisting frames with ductile connections							
Moderately ductile frames	2.0	1.5	NL	NL	20	20	20
Limited-ductility frames	1.5	1.5	NL	NL	15	15	15
Other wood- or gypsum-based SFRS(s) not listed previously	1.0	1.0	15	15	NP	NP	NP

Table 9 (*concluded*).

			Restrictions				
			$I_E F_a S_a(0.2)$				
Type of SFRS	R_d	R_o	<0.20	≥0.20 to <0.35	≥0.35 to ≤0.75	>0.75	$I_E F_v S_a(1.0) > 0.30$
Masonry structures designed and detailed according to CSA standard CSA-S304.1-94 (under preparation)							
Moderately ductile shear walls	2.0	1.5	NL	NL	60	40	40
Limited-ductility shear walls	1.5	1.5	NL	NL	40	30	30
Conventional construction							
Shear walls	1.5	1.5	NL	60	30	15	15
Moment-resisting frames	1.5	1.5	NL	30	NP	NP	NP
Unreinforced masonry	1.0	1.0	30	15	NP	NP	NP
Other masonry SFRS(s) not listed previously	1.0	1.0	15	NP	NP	NP	NP

Note: The values in the restrictions columns are maximum height limits in metres. The most stringent requirement governs. NL, system is permitted and not limited in height as an SFRS, and height may be limited elsewhere in other parts; NP, system is not permitted.

The ductility-related force modification factor, R_d, essentially corresponds to the R factor used in the 1995 NBCC. In the proposed 2005 NBCC provisions, this factor ranges from 1.00 for brittle systems such as unreinforced masonry to 5.00 for the most ductile systems such as ductile steel moment-resisting frames. The proposed overstrength-related force modification factor, R_o, which varies between 1.00 and 1.70, is introduced to account for the reserve of strength in the SFRS. In lieu of increasing the factored resistance to account for overstrength, the design force level is reduced by including the R_o factor in the denominator of the base shear equation. This approach is more in line with the usual design procedures where the factored resistance is compared with the factored load effects. The impact of these proposed changes and comparisons of design force levels with those in the 1995 NBCC are discussed by Heidebrecht (2003).

Acknowledgements

The authors wish to acknowledge the members of the Canadian National Committee on Earthquake Engineering and members of the different technical committees of the CSA standards for the many discussions during the development of the proposed provisions for seismic design.

References

ACI. 1983. Building code requirements for reinforced concrete (ACI318-83). American Concrete Institute, Detroit, Mich.

Adams, J., and Atkinson, G. 2003. Development of seismic hazard maps for the proposed 2005 edition of the National Building Code of Canada. Canadian Journal of Civil Engineering, **30**: 255–271.

ATC. 1995*a*. Structural response modification factors. ATC-19, Applied Technology Council (ATC), Redwood City, Calif.

ATC. 1995*b*. A critical review of current approaches to earthquake-resistant design. ATC-34, Applied Technology Council (ATC), Redwood City, Calif.

ATC. 1997. NEHRP guidelines for the seismic rehabilitation of buildings (FEMA-273). ATC-33, Applied Technology Council (ATC), Redwood City, Calif.

Bruneau, M., Uang, C.-M., and Whittaker, A. 1998. Ductile design of steel structures. McGraw-Hill, New York.

Ceccotti, A., and Karacabeyli, E. 2002. Validation of seismic design parameters for wood-frame shearwall systems. Canadian Journal of Civil Engineering, **29**: 484–498.

CSA. 1984. Design of concrete structures for buildings. Standard CAN3-A23.3-M84, Canadian Standards Association, Rexdale, Ont.

CSA. 1994*a*. Design of concrete structures. Standard CSA-A23.3-94, Canadian Standards Association, Rexdale, Ont.

CSA. 1994*b*. Masonry design for buildings (limit states design). Standard CSA-S304.1-94, Canadian Standards Association, Rexdale, Ont.

CSA. 2001*a*. Limit states design of steel structures. Standard CSA-S16-01, Canadian Standards Association, Rexdale, Ont.

CSA. 2001*b*. Engineering design in wood. Standard CSA-O86-01, Canadian Standards Association, Rexdale, Ont.

CWC. 2001. Wood design manual 2001. Canadian Wood Council (CWC), Ottawa, Ont.

ECS. 1998. Eurocode 8: design of structures for earthquake resistance. May 2001 Draft, European Committee for Standardization (ECS), Brussels, Belgium.

Fajfar, P., and Fischinger, M. 1990. On the response modification factor for reinforced concrete buildings. *In* Proceedings of the 4th U.S. National Conference on Earthquake Engineering, 20–24 May 1990, Palm Springs, Calif. Vol. 2, pp. 249–258.

Glanville, J.I., Hatzinikolas, M.A., and Ben-Omran, H.A. 1996. Engineered masonry design: limit states design. Winston House Enterprises, Winnipeg, Man.

Heidebrecht, A.C. 2003. Overview of seismic provisions of the proposed 2005 edition of the National Building Code of Canada. Canadian Journal of Civil Engineering, **30**: 241–254.

Heidebrecht, A.C., and Tso, W.K. 1985. Seismic loading provisions changes in the National Building Code of Canada. Canadian Journal of Civil Engineering, **12**: 653–660.

Humar, J., and Mahgoub, M.A. 2003. Determination of seismic design forces by equivalent static load method. Canadian Journal of Civil Engineering, **30**: 287–307.

Instituto de Ingeniería. 1987. Normas tecnicas complementarias para diseño por sismo. Departamento del Distrito Federal (Code for seismic design in the Federal District). Universidad Nacional Autónoma de México, Mexico City.

Karacabeyli, E., and Ni, C. 2001. Shearwalls and diaphragms: commentary of CSA Standard O86.1. Canadian Standards Association, Rexdale, Ont.

Koboevic, S., and Redwood, R.G. 1997. Design and seismic response of shear critical eccentrically braced frames. Canadian Journal of Civil Engineering, **23**: 761–771.

Kulak, G.L., Kennedy, D.J.L., Driver, R.G., and Medhekar, M. 2001. Steel plate shear walls: an overview. Engineering Journal, American Institute of Steel Construction, **38**(1): 50–62.

Mirza, S.A., and MacGregor, J.G. 1979. Variability of mechanical properties of reinforcing bars. ASCE Journal of the Structural Division, **105**(ST5): 921–937.

Mitchell, D., and Paultre, P. 1994. Ductility and overstrength in seismic design of reinforced concrete structures. Canadian Journal of Civil Engineering, **21**: 1049–1060.

Mitchell, D., Paultre, P., and Collins, M.P. 1995. Seismic design. *In* Concrete design handbook. 2nd ed. Canadian Portland Cement Association, Ottawa, Ont. Chapt. 11, pp. 11.1–11.33.

Nassar, A., and Krawinkler, H. 1991. Seismic demands for SDOF and MDOF systems. Report 95, John Blume Earthquake Engineering Research Center, Department of Civil Engineering, Stanford University, Stanford, Calif.

NBCC. 1980. National Building Code of Canada 1980. National Research Council of Canada, Ottawa, Ont.

NBCC. 1995. National Building Code of Canada. Institute for Research in Construction, National Research Council of Canada, Ottawa, Ont.

NEHRP. 1997. NEHRP recommended provisions for seismic regulations for new buildings and other structures. Part 2: commentary. FEMA-303, National Earthquake Hazard Reduction Program, Building Seismic Safety Council, Federal Emergency Management Agency, Washington, D.C.

Ni, C., and Karacabeyli, E. 2000. Effect of overturning restraint on performance of shearwalls. *In* Proceedings of the World Conference on Timber Engineering, Whistler, B.C., 31 July – 3 August. Department of Wood Science, The University of British Columbia, Vancouver, B.C. CD-Rom Paper No. 2.3.1: 1–9.

NZS. 1982. New Zealand standard code of practice for the design of concrete structures. NZS 3101: Part 1, Standard Association of New Zealand, Wellington, New Zealand.

Osteraas, G.D., and Krawinkler, H. 1990. Strength and ductility considerations in seismic design. Report 90, John Blume Earthquake Engineering Research Center, Department of Civil Engineering, Stanford University, Stanford, Calif.

Park, R., and Paulay, T. 1975. Reinforced concrete structures. John Wiley & Sons, New York.

Paulay, T., and Priestley, M.J.N. 1992. Seismic design of reinforced concrete and masonry buildings. John Wiley and Sons, New York.

Paultre, P., Castele, D., Rattray, S., and Mitchell, D. 1989. Seismic response of reinforced concrete frame sub assemblages — a Canadian code perspective. Canadian Journal of Civil Engineering, **16**: 627–649.

Popovski, M., Prion, H.G.L., and Karacabeyli, E. 1999. Braced timber frames as a load resistant system. *In* Proceedings of the Pacific Timber Engineering Conference, Rotorua, New Zealand, 15–20 March. New Zealand Forest Research Institute Limited, Rotorua, New Zealand. Vol. 3, pp. 391–398.

Popovski, M., Prion, H.G.L., and Karacabeyli, E. 2002. Seismic performance of connections in heavy timber construction. Canadian Journal of Civil Engineering, **29**: 389–399.

Rahgozar, M.A., and Humar, J.L. 1998. Accounting for overstrength in seismic design of steel structures. Canadian Journal of Civil Engineering, **25**: 1–15.

Rainer, J.H., and Karacabeyli, E. 2000. Performance of woodframe construction in earthquakes. *In* Proceedings of the 12th World Conference on Earthquake Engineering, Auckland, New Zealand, 30 January – 4 February. Society for Earthquake Engineering, Silverstream, New Zealand. CD-Rom Paper No. 2454: 1–8.

SAC. 2000. Recommended seismic design criteria for new steel moment-frame buildings. FEMA Report 350, SAC Joint Venture for the Federal Emergency Management Agency, Washington, D.C.

Schmidt, B.J., and Bartlett, F.M. 2002. Review of resistance factor for steel: data collection. Canadian Journal of Civil Engineering, **29**: 98–108.

Tremblay, R. 2001. Seismic behaviour and design of concentrically braced steel frames. Engineering Journal, American Institute of Steel Construction, **38**(3): 148–166.

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

Tremblay, R., Bruneau, M., Nakashima, M., Prion, H.G.L., Filiatrault, A., and DeVall, R. 1996. Seismic design of steel buildings: lessons from the 1995 Hyogoken-Nanbu earthquake. Canadian Journal of Civil Engineering, **23**: 727–756.

Tso, W.K. 1992. Overview of seismic provision changes in National Building Code of Canada. Canadian Journal of Civil Engineering, **19**: 383–388.