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# Response modification factors of chevron-braced frames

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**Abstract**

The overstrength, ductility, and the response modification factors of special concentric braced frames (SCBFs) and ordinary concentric braced frames (OCBFs) were evaluated by performing pushover analysis of model structures with various stories and span lengths. The results were compared with those from nonlinear incremental dynamic analyses. According to the analysis results, the response modification factors of model structures computed from pushover analysis were generally smaller than the values given in the design codes except in low-rise SCBFs. The results of incremental dynamic analysis generally matched well with those obtained from pushover analysis.


*Keywords:* Special concentrically braced frames; Ordinary concentrically braced frames; Overstrength factors; Ductility factors; Response modification factors

## 1. Introduction

Many seismic codes permit a reduction in design loads, taking advantage of the fact that the structures possess significant reserve strength (overstrength) and capacity to dissipate energy (ductility). The overstrength and the ductility are incorporated in structural design through a force reduction factor or a response modification factor. The factor represents the ratio of the forces that would develop under the specified ground motion if the structure were to behave elastically to the prescribed design forces at the strength limit state. Such a design concept is based on the assumption that well-detailed structures can develop lateral strength in excess of their design strength and sustain large inelastic deformation without collapse.

The role of the force reduction factor is essential in designing the earthquake load-resisting elements. The response modification factors, which were first proposed in the ATC 3-06 report [1], were in fact selected through committee consensus based on the observed performance of buildings during past earthquakes and on the estimates of system overstrength and damping, etc. [2].

In documents such as ATC-19 [2] and ATC-34 [3], the response modification factor was calculated as the product of three factors: overstrength factor, ductility factor, and redundancy factor. Osteraas [4] conducted a detailed study of reserve strength of three structural systems: distributed moment frames, perimeter moment frames, and concentric braced frames designed following the allowable stress design provisions with seismic loads per UBC seismic zone 4 and soil type S2. They observed that the overstrength factor of braced frames ranged between 2.8 and 2.2. Balendra and Huang [5] found that the overstrength factors and the ductility factors were almost the same for inverted V-braced and split X-braced frames. They also observed that the response modification factors decreased when the number of stories increased. For three-, six-, and ten-story braced frames, the response modification factor was found to vary from 8.5 to 3.5. Maheri and Akbari [6] investigated the response modification factors of steel-braced reinforced concrete framed dual systems. They found that the addition of steel X- and knee-braces increased the response modification factor significantly, and that the number of stories appeared to be the predominant variable. According to the results of the above studies, the modification factor appears to be a period and applied load-dependent factor contrary to seismic design codes prescribing a single value

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**Notation**

| | |
|---|---|
| $A_g$ | Gross sectional area of braces |
| $E_s$ | Elastic modulus of members |
| $F_{cr}$ | Buckling stress of braces |
| $F_y$ | Nominal yield stress |
| $KL$ | Effective length of the compression members such as columns and braces |
| $P_{cr}$ | Buckling strength of braces |
| $P_n$ | Nominal buckling strength of braces |
| $P_y$ | Nominal axial yield force of braces |
| $Q_b$ | Unbalanced load between the tensile yield and compressive buckling loads of braces |
| $R$ | Response modification factor |
| $R_o$ | Overstrength factor |
| $R_r$ | Redundancy factor |
| $R_y$ | Ratio of the expected yield stress and the nominal yield stress |
| $R_\mu$ | Ductility factor |
| $r$ | Radius of gyration |
| $V_e$ | Natural period of the structure |
| $V_d$ | Maximum seismic demand for elastic response |
| $V_y$ | Design base shear |
| $T$ | Base shear corresponding to the maximum inelastic displacement |
| $\Delta_{1.5\%}$ | Axial deformation of the brace at the maximum interstory drift of 1.5% of story height |
| $\Delta_{2.0\%}$ | Axial deformation of the brace at the maximum interstory drift of 2.0% of story height |
| $\Delta_{by}$ | Yield deformation of brace |
| $\Delta_{cb}$ | Buckling deformation of brace |
| $\Delta_e$ | Displacement of a corresponding elastic structure |
| $\Delta_{\max}$ | Maximum displacement of a structure |
| $\Delta_y$ | Yield displacement of a structure |
| $\mu$ | Displacement ductility ratio of a structure |
| $\Phi$ | Coefficient reflecting a soil condition |
| $\Phi_c$ | Strength reduction factor of braces |
| $\theta$ | Slope of braces |

for all buildings with a specific structural system regardless of building height and earthquake load level.

IBC 2000 [7] divides concentric braced frames into ordinary concentric braced frames (OCBFs) and special concentric braced frames (SCBFs). The 1997 edition of the Seismic Provisions for Structural Steel Buildings [8] of AISC introduced the SCBF based on experimental and analytical works which showed that the post-buckling behavior of concentrically braced frames could be greatly improved by limiting width/thickness ratios of structural members and by maintaining closer spacing of stitches and special design and detailing of end connections. The AISC Seismic Provisions stipulate that a floor beam of an SCBF, which is intersected by braces, shall be designed to support all the loads assuming that bracing is not present. Also the beam needs to be designed considering the maximum unbalanced vertical load applied to the beam by the braces. Therefore the floor beams in an SCBF are generally stronger than those in an OCBF, and SCBFs are expected to have increased ductility over OCBFs due to lesser strength degradation when compression braces buckle. Reflecting the difference, the IBC 2000 suggests different response modification factors for SCBFs and OCBFs, which are 6 and 5, respectively, although it provides the same overstrength factor, which is 2.

Previously the braced frame, originally developed to resist wind load, was widely investigated for seismic application both experimentally [9–11] and analytically [12–15]. The focus of those studies was on the strength of individual members, such as slenderness ratio or width/thickness ratio, or on the elastic/inelastic behavior of braced frames for various geometries or locations of braces. The previous research for evaluation of response modification factors has been carried out with limited model structures and design variables: for example, braced frames in low seismic regions [5], low-story braced frames designed in accordance with allowable stress design procedure [4], and reinforced concrete structure strengthened by steel braces [6]. However no research has yet been conducted for comparison of behavior factors of SCBFs and OCBFs designed for the same loading conditions following the Load and Resistance Factor Design procedure.

The present study focuses on the evaluation of overstrength, ductility, and response modification factors of twenty one SCBFs and nine OCBFs, designed in accordance with IBC 2000 and AISC Seismic Provisions for Structural Steel Buildings. Nonlinear static pushover analyses were carried out to obtain such behavior factors, and the results of the six-story SCBF were compared with those of nonlinear incremental dynamic analyses to verify the results of nonlinear static analysis procedure.

## 2. Response modification factors

Mazzolani and Piluso [16] addressed various theoretical approaches to compute the response modification factor ($q$-factor), such as the maximum plastic deformation approach, the energy approach, and the low-cycle fatigue approach. ATC-19 proposed a simplified procedure to estimate the response modification factors, in which the response modification factor, $R$, is calculated as the product of the three parameters that profoundly influence the seismic response of structures:

$$R = R_o R_\mu R_r \tag{1}$$

where $R_o$ is the overstrength factor to account for the observation that the maximum lateral strength of a structure generally exceeds its design strength. Recently FEMA-369 [17]

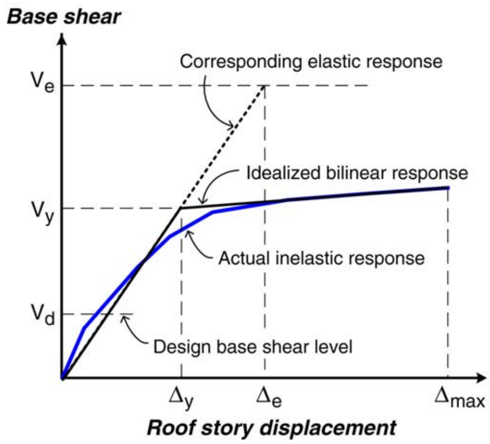



Fig. 1. Lateral load–roof displacement relationship of a structure.

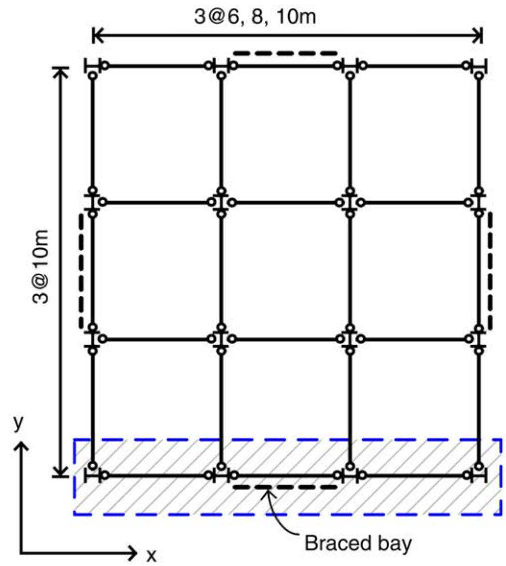
specified three components of overstrength factors in Table C5.2.7-1: design overstrength, material overstrength, and system overstrength. $R_\mu$ is a ductility factor which is a measure of the global nonlinear response of a structure, and $R_r$ is a redundancy factor to quantify the improved reliability of seismic framing systems constructed with multiple lines of strength. In this study it is assumed that there are plenty of vertical lines of seismic framing system, and the redundancy factor is equal to 1.0. In this case the response modification factor is determined as the product of the overstrength factor and the ductility factor. Fig. 1 represents the base-shear versus roof displacement relation of a structure, which can be developed by a nonlinear static analysis. The ductility factor $R_\mu$ and the overstrength factor $R_o$ are defined as follows:

$$R_\mu = \frac{V_e}{V_y}, \qquad R_o = \frac{V_y}{V_d} \tag{2}$$

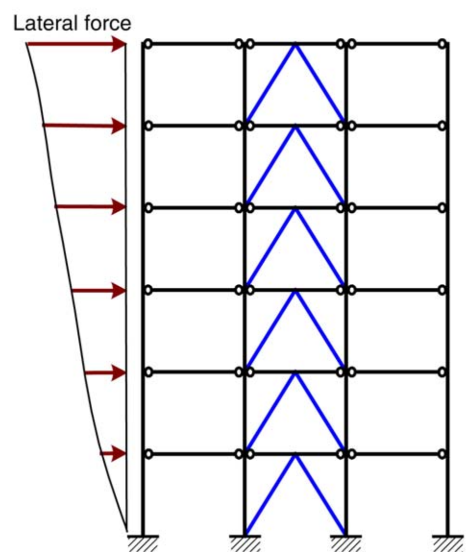
where $V_d$ is the design base shear, $V_e$ is the maximum seismic demand for elastic response, and $V_y$ is the base shear corresponding to the maximum inelastic displacement.

## 3. Design of model structures

To evaluate the overstrength factors, ductility factors, and the response modification factors of braced frames, 3, 6, 9, 12, 15, 18, and 21 story SCBFs and 3, 6, and 9 story OCBFs with the bay length varied as 6, 8, and 10 m were designed per the 'Load and Resistance Factor Design' [18] and the 'Seismic Provisions for Structural Steel Buildings' [8] of AISC. The story height of every model structure was fixed to 3.6 m. Fig. 2(a) shows the plan of the prototype structure, and the chevron braces are located in the mid-bay of the perimeter frames (Fig. 2(b)). The dead and live loads of 4.90 and 2.45 kN/m$^2$, respectively, were used for gravity load, and the earthquake design base shear was determined based on the IBC 2000 using the following parameters: spectral acceleration $S_{DS} = 0.5$ g and $S_{D1} = 0.3$ g, Seismic Use Group II, soil type B (rock site), and the response modification factors = 6.0 for SCBF and 5.0 for OCBF. A36 steel was used for every structural member. The braces were designed to resist all lateral seismic loads, and the beam–column joints were assumed to be pinned. The structural design was carried out using the program code MIDAS-Gen [19]. To take the conventional design practice into consideration, the same structural members were used in three consecutive stories. The structural members selected for the nine-story model structures are listed in Table 1. Only the external braced frames shown in Fig. 2(a) were used for analyses.




(a) Plan.




(b) Brace configuration.

Fig. 2. Configuration of model structures.

In the Seismic Provisions for Structural Steel Buildings the slenderness ratios of compression members (columns and braces) of inverted V-type SCBFs and OCBFs are limited as follows:

$$\frac{KL}{r} \leq 5.87\sqrt{\frac{E_s}{F_y}} \qquad \text{for SCBF} \tag{3a}$$

Table 1
Sectional properties of nine-story model structures

| Span length (m) | Story | Interior col. | Exterior col. | Interior beam | Exterior beam | Brace |
|---|---|---|---|---|---|---|
| (a) Special concentric braced frames | | | | | | |
| | 1–3 | W14 × 176 | W10 × 39 | W27 × 217 | W16 × 36 | W8 × 35 |
| 6 | 4–6 | W14 × 109 | W10 × 30 | W27 × 217 | W16 × 36 | W8 × 35 |
| | 7–9 | W14 × 38 | W10 × 22 | W27 × 161 | W16 × 36 | W8 × 24 |
| | 1–3 | W14 × 193 | W10 × 54 | W36 × 260 | W16 × 67 | W10 × 45 |
| 8 | 4–6 | W14 × 109 | W10 × 39 | W36 × 230 | W16 × 67 | W8 × 40 |
| | 7–9 | W14 × 43 | W10 × 22 | W36 × 210 | W16 × 67 | W8 × 35 |
| | 1–3 | W14 × 211 | W12 × 65 | W36 × 439 | W18 × 86 | W8 × 67 |
| 10 | 4–6 | W14 × 120 | W12 × 45 | W36 × 359 | W18 × 86 | W8 × 58 |
| | 7–9 | W14 × 48 | W12 × 30 | W36 × 300 | W18 × 86 | W8 × 48 |
| (b) Ordinary concentric braced frames | | | | | | |
| | 1–3 | W14 × 211 | W10 × 49 | W14 × 26 | W16 × 36 | W8 × 35 |
| 6 | 4–6 | W14 × 99 | W10 × 39 | W14 × 26 | W16 × 36 | W8 × 35 |
| | 7–9 | W14 × 43 | W10 × 26 | W14 × 26 | W16 × 36 | W8 × 28 |
| | 1–3 | W14 × 233 | W10 × 68 | W14 × 43 | W16 × 67 | W8 × 48 |
| 8 | 4–6 | W14 × 120 | W10 × 45 | W14 × 43 | W16 × 67 | W8 × 48 |
| | 7–9 | W14 × 48 | W10 × 30 | W14 × 43 | W16 × 67 | W8 × 31 |
| | 1–3 | W14 × 283 | W12 × 79 | W14 × 53 | W18 × 86 | W10 × 54 |
| 10 | 4–6 | W14 × 132 | W12 × 58 | W14 × 53 | W18 × 86 | W10 × 49 |
| | 7–9 | W14 × 61 | W10 × 33 | W14 × 53 | W18 × 86 | W10 × 45 |

$$\frac{KL}{r} \leq 4.23\sqrt{\frac{E_s}{F_y}} \quad \text{for OCBF} \tag{3b}$$

where $r$ is the radius of gyration, $KL$ is the effective length, $E_s$ is the elastic modulus, and $F_y$ is the yield stress of the members. The width-to-thickness ratio specified in the Seismic Provision of AISC Table I-8-1 was also applied in the design. For design of beams in SCBFs the unbalanced load ($Q_b$) between the tensile yield and compressive buckling loads of braces as well as the gravity loads was considered. As shown in Fig. 3, the unbalanced load was obtained as follows with the buckling load taken to be $0.3\Phi_c P_n$:

$$\begin{aligned} Q_b &= (R_y P_y - 0.3\Phi_c P_n) \times \sin\theta \\ &= (R_y A_g F_y - 0.3\Phi_c A_g F_{cr}) \times \sin\theta \end{aligned} \tag{4}$$

where $R_y$ is the ratio of the expected yield stress and the nominal yield stress ($F_y$), for which 1.5 is recommended in the Seismic Provision [8]. $P_y$ is the nominal axial yield force, $P_n$ is the nominal buckling strength, $\Phi_c$ is the strength reduction factor, $F_{cr}$ is the buckling stress of braces, and $\theta$ is the slope of the brace as described in Fig. 3.

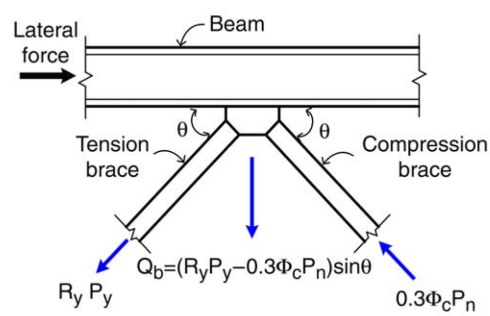


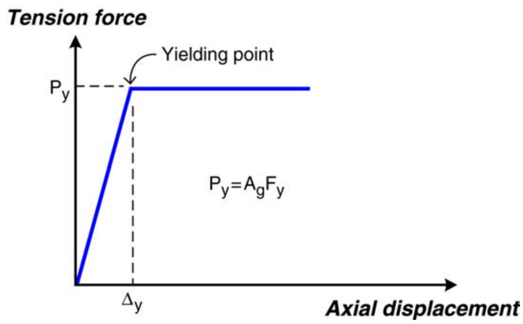

Fig. 3. Unbalanced force due to buckling of compression brace.

## 4. Nonlinear static analysis of model structures

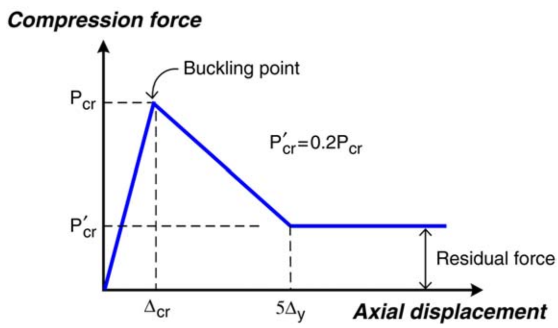
Mwafy and Elnashai [20] investigated the applicability of the inelastic static (pushover) analysis and the inelastic dynamic analysis on 12 reinforced concrete buildings with various characteristics. They concluded that the static pushover analysis is more appropriate for low-rise and short-period frame structures. For long-period or high-rise structures, however, the inelastic dynamic analysis is preferable due to the participation of higher modes. In this study the pushover analysis was employed to obtain the inelastic responses of model structures, and the results of the six-story SCBF were compared with those obtained from dynamic analyses to verify the applicability of the static procedure.

### 4.1. Pushover analysis

Eigenvalue analyses were conducted first using the program DRAIN-2DX [21] to determine the elastic natural periods and mode shapes of the model structures. Then pushover analyses were carried out to evaluate the global yield limit state and the structural capacity by progressively increasing the lateral story forces proportional to the fundamental mode shape. The post-yield stiffness of the beams and columns was assumed to be 2% of the initial stiffness, and that of the braces was assumed to be zero. The expected yield stress of structural members was assumed to




(a) Tension brace.




(b) Compression brace.

Fig. 4. Simplified analysis model for force–displacement relationship of brace.

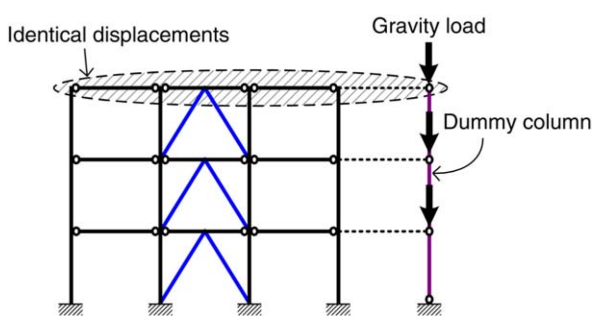
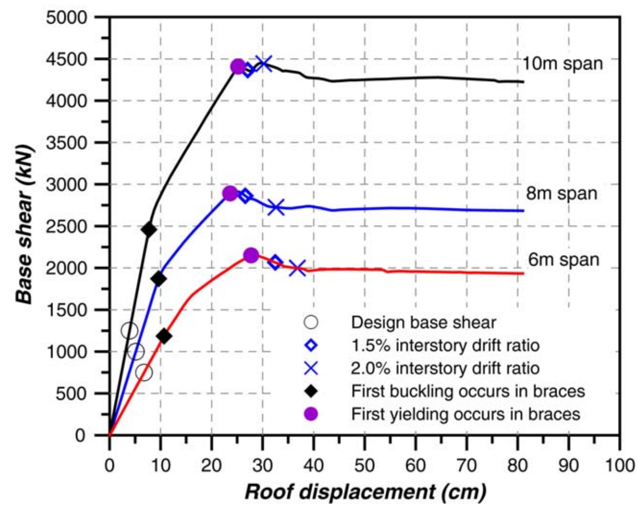


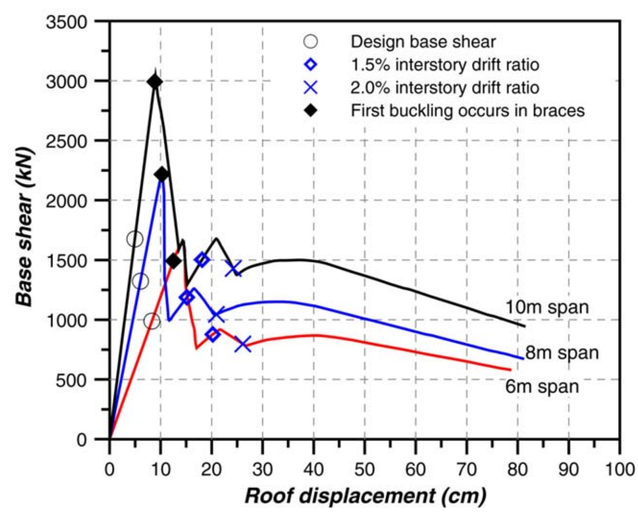

Fig. 5. Modeling of $P$–delta effect.

be 1.5 times the nominal yield stress as recommended by the Seismic Provisions for Structural Steel Buildings [8] for ASTM A36 steel. The phenomenological model proposed by Jain and Goel, which was also presented in FEMA-274 [22], was used for modeling nonlinear behavior of braces (Fig. 4). The post-buckling residual compression force is set to be 20% of the buckling load as given in Tables 5–8 of FEMA-273 [23]. The $P$–$\Delta$ effect was considered by employing the dummy column, shown in Fig. 5, which is subjected to the gravity load of interior frames not included in the analysis models.

Fig. 6(a) and (b) show the pushover curves of the nine-story SCBF and OCBF structures, respectively. In the base shear–roof displacement curves, the points corresponding to the design base shear, the first buckling and yielding of braces, maximum inter-story drift of 1.5% and 2.0% of story height are indicated. In the figures it can be observed



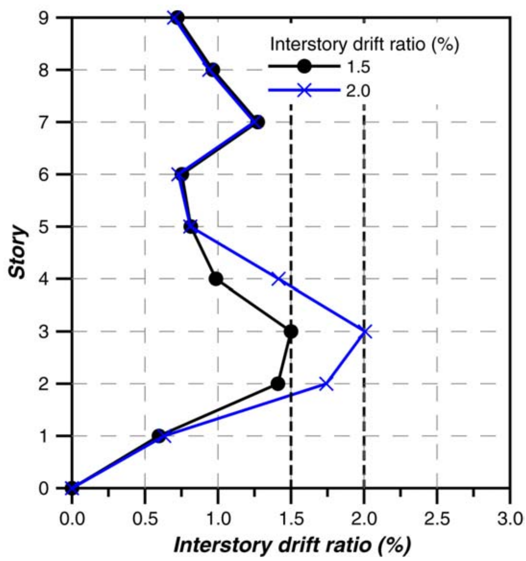

(a) SCBF.



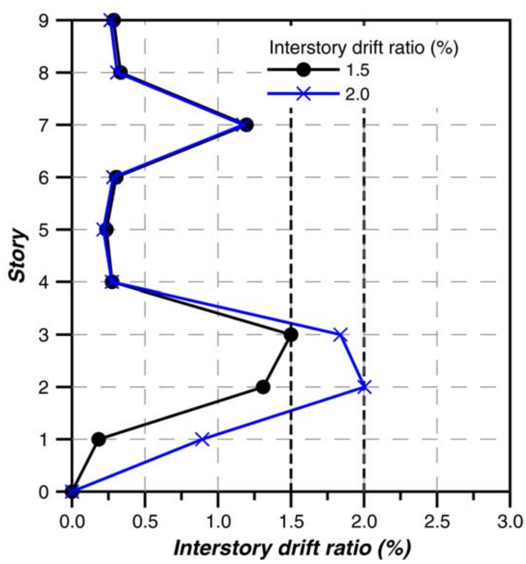

(b) OCBF.

Fig. 6. Pushover curves of the nine-story structures.

that the stiffness of the SCBF decreases slightly by the buckling of a compressive brace, and the maximum strength is reached slightly before the first yielding of a tensile brace. The maximum strength is about three times as high as the design base shear. However in the OCBF, the structure behaves elastically and then the strength drops sharply at the occurrence of the first buckling of the compression brace and the subsequent yielding of beams connected to the buckled brace. The lateral strength of the structure slightly increases with further redistribution of loads, but drops again at buckling of braces in other stories. It also can be observed both in the SCBF and OCBF that as the span length increases the stiffness and strength of the system increase. Fig. 7 shows the inter-story drift ratio of the nine-story structure with 6 m span length, where it can be observed that large drift occurs in lower stories where buckling occurs in braces. Fig. 8 depicts the state of damage in structural members and the ductility ratio in braces of the nine-story SCBFs. It can be observed that when the




(a) SCBF.




(b) OCBF.

Fig. 7. Inter-story drift ratio of the nine-story structures with 6 m span length.

maximum inter-story drift reaches 1.5% of the story height most of the braces under compression buckle. No plastic hinge can be observed in beams which were designed considering the vertical unbalanced force. The corresponding results for OCBFs with the same height and bay length are shown in Fig. 9, which suggests that damage is concentrated in braces and beams located in the lower three stories. Compared to the SCBF, the ductility demand in braces is much higher in the OCBF.

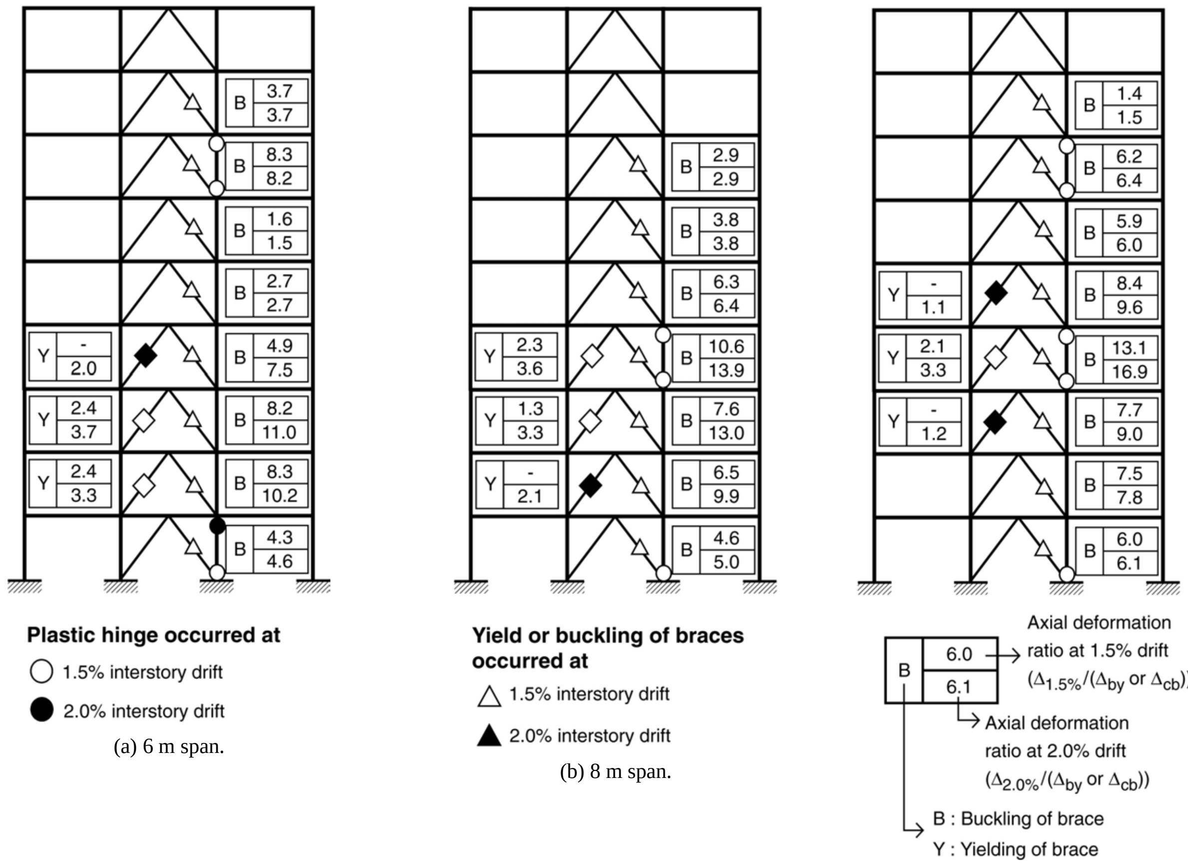
### *4.2. Failure state of braced frames*

The failure criteria of a structure are generally defined in two levels: local and global levels. Appendix I: Tentative guidelines for performance-based seismic engineering of the SEAOC-Blue Book [24] regulates the limit state of SCBFs and OCBFs for the collapse prevention stage as 2.2% and 1.5% of the maximum story drift or as the maximum story ductility ratio of 5.0. In FEMA-356 [25] the maximum inter-story drift ratio of a braced frame is limited to 2.0% for the collapse prevention performance level, which is significantly smaller than the 5.0% story drift ratio recommended for moment frames. In an element level, FEMA-356 regulates the acceptance criteria of tensile and compressive braces classified as primary components in the collapse prevention stage as nine times the yield deformation ($\Delta_{by}$) and seven times the buckling deformation ($\Delta_{cb}$), respectively. In this study the global limit state of 2.0% maximum inter-story drift ratio is used to define the collapse state of an SCBF, and 1.5% of the maximum inter-story drift ratios for an OCBF. At the global limit state, the fracture limit state of each member is also checked. The local failure criteria are also checked at the 2.0% of the maximum drift ratio in OCBF for comparison.

Figs. 8 and 9 demonstrate that some of the braces under compression reach a limiting state when the maximum inter-story drift reaches 1.5% of the story height. Although not one tension brace reached a limiting state, it would be reasonable to consider some of them already failed because of the bi-directional nature of earthquakes. However it should be noted that a global failure may not occur by failure of few braces. Therefore to consider 2.0% of maximum inter-story drift ratio for the SCBF and 1.5% for the OCBF as a global limiting state of a braced frame appears to be reasonable. For comparison the behavior factors at 2.0% drift ratio were also obtained in the OCBF.

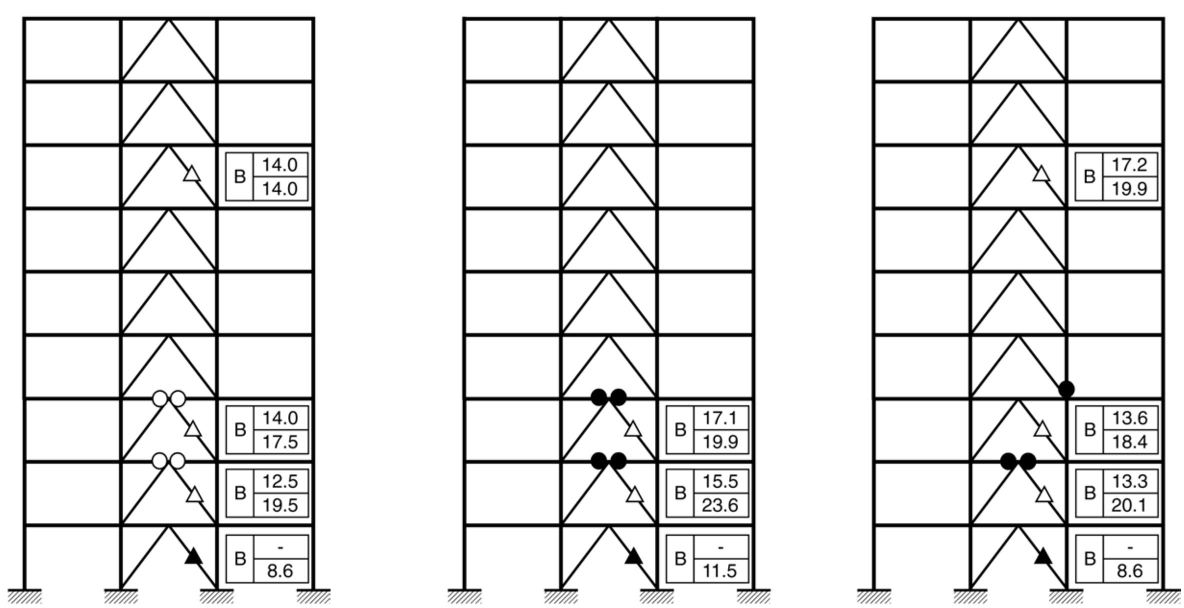
### *4.3. Overstrength factors*

The capacity envelopes obtained from pushover analysis were utilized to evaluate overstrength factors. To find out the yield point, a straight line was drawn in such a way that the area under the original curve is equal to that of the idealized one as recommended in FEMA-356 [25] for structures with negative post-yield stiffness (Fig. 10). It is recommended that the yield strength of the idealized force–displacement curve should not be taken as greater than the maximum base shear force of the actual pushover curve. The base shear at yield and the maximum strength of all the analysis model structures are presented in Table 2, and the overstrength factors are plotted in Fig. 11. It can be observed that the overstrength factors of SCBF structures increase as the span length increases and the number of stories decreases. However those of OCBFs are not much affected by the change in span length. As the span length increases, the gravity load and the design base shear increase if other design conditions remain the same. Also increased are: (i) the bending moment in beams and columns; (ii) axial load in columns; and consequently (iii) the nominal strength of structural members. The increase in base shear in SCBFs results in increase in brace size, the unbalanced force acting




(a) 6 m span. (b) 8 m span. (c) 10 m span.

Fig. 8. Locations of inelastic deformation and brace ductility ratios of the nine-story SCBFs.



(a) 6 m bay length. (b) 8 m bay length. (c) 10 m bay length.

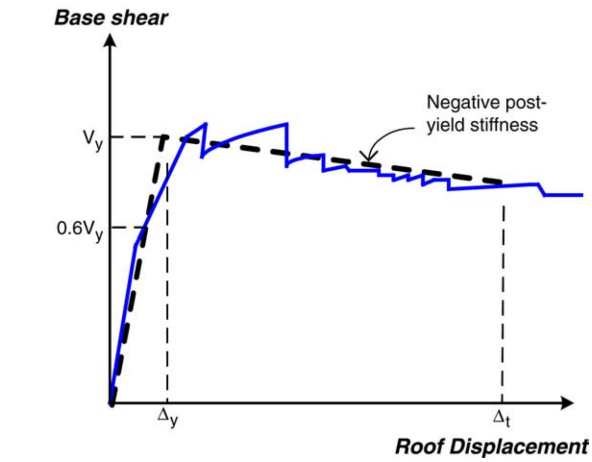
Fig. 9. Location of inelastic deformation and brace ductility ratios of the nine-story OCBFs.

on the floor beams, the size of beams, and consequently the overstrength factors. However in OCBFs, in which the unbalanced force is not considered, the increase in cross-sectional area of structural members is not as significant as in SCBFs. For example, when the span lengths of the three-story SCBF and OCBF increase from 6 to 8 m, the increase in design base shear is almost the same (33.3% and 33.6%, respectively). However the increase in yield strength in SCBF, which is 72.2%, is significantly larger than that of the OCBF, which is 43.6%. This explains the observation that the increase in overstrength factors in SCBFs is larger than that in OCBFs when the span length increases.

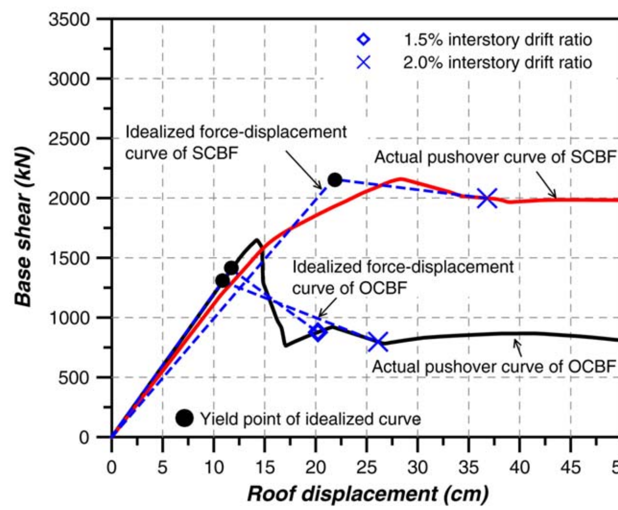
In SCBFs, the overstrength factors are generally larger than the value specified in IBC-2000, which is 2, except for the 21-story structure with span length of 6 m. In OCBF structures, however, the overstrength factors turned out to be smaller than the specified value of 2. These results are compatible with the FEMA-369 report [17] which states that overstrength factors for braced frames vary from 1.5 to 2.0. It also can be observed that the overstrength factors of OCBFs, estimated at the 1.5% inter-story drift ratio, are somewhat larger than those estimated at the 2.0% inter-story drift. The analysis results show that the overstrength factors of SCBFs are about two to three times larger than those of OCBFs. As mentioned earlier, the larger overstrength factors result from the use of larger beams designed considering the unbalanced force in braces. This also contributes to the distribution of plastic hinges in almost all stories in SCBFs, whereas in OCBFs the plastic hinges are concentrated in a few lower stories as seen in Figs. 8 and 9. This implies that soon after the first buckling of a brace and consequent formation of plastic hinges in the beam, the lateral-load resisting capacity of the OCBF structure decreases rapidly, resulting in lower overstrength factors.

The overstrength factors of OCBFs obtained in this study are somewhat smaller than obtained by Uang and Bertero [10] from the experiments of the 30%-scaled model with slabs, which is 2.4. The larger value for overstrength factor seems to be contributed from the participation of the slab in resisting the unbalanced force of braces. It is reported that the composite action of a composite beam increases the bending stiffness about 10–15% and the bending strength about 5–10% [13]. Therefore the results of the current study for overstrength factors, which were obtained from analytical study using bare frames without a slab, may be considered as a lower bound.

In IBC 2000 some special elements and components are required to be designed for the special seismic load combinations in which the earthquake load effect is multiplied by the system overstrength factor. This is to ensure that the elements have enough strength to resist the maximum force transferred from the other elements of the lateral force resisting system. If the overstrength factor is underestimated, the transferred force is also underestimated; and this may lead to unsafe design. Therefore precise estimation of the overstrength factor is essential to guarantee seismic safety of structures.




(a) Idealized curve for negative post-yield slope.




(b) Bilinear representation of nine-story OCBF (6 m span).

Fig. 10. Idealized force–displacement curve for braced frames.

### 4.4. Ductility factors

The ductility factor $R_\mu$ was obtained using the system ductility factor $\mu$ by the procedure proposed by Newmark and Hall [26] and Miranda and Bertero [27]. Newmark and Hall proposed the following equations for the system ductility factors:

$$\begin{aligned} R_\mu &= 1.0 \qquad (T < 0.03\ \text{s}) \\ R_\mu &= \sqrt{2\mu - 1} \qquad (0.12 < T < 0.03\ \text{s}) \\ R_\mu &= \mu \qquad (T > 1.0\ \text{s}) \end{aligned} \tag{5}$$

where $T$ is the natural period of the structure. Miranda and Bertero developed general $R_\mu$–$\mu$–$T$ relationships using 124 ground motions recorded on a wide range of soil conditions.

Table 2
Results of pushover analysis for model structures (Unit: kN, cm, s)

(a) Special concentric braced frames

| Span length (m) | | # of stories | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | 3 | 6 | 9 | 12 | 15 | 18 | 21 |
| 6 | Period | 0.46 | 0.88 | 1.44 | 2.07 | 2.79 | 3.54 | 4.40 |
| | $V_d$ | 413.64 | 677.37 | 749.66 | 805.57 | 851.79 | 891.51 | 926.52 |
| | $V_y$ | 1312.35 | 1905.64 | 2156.55 | 2052.35 | 1956.65 | 1859.12 | 1763.84 |
| | $\Delta_y$ | 3.76 | 10.14 | 21.74 | 32.02 | 46.42 | 62.97 | 79.59 |
| | $\Delta$ | 10.52 | 17.60 | 36.80 | 46.64 | 64.69 | 86.03 | 104.00 |
| 8 | Period | 0.40 | 0.78 | 1.27 | 1.79 | 2.38 | 2.99 | 3.64 |
| | $V_d$ | 551.49 | 903.17 | 999.52 | 1074.07 | 1135.68 | 1188.65 | 1235.36 |
| | $V_y$ | 2260.00 | 2922.22 | 2908.47 | 3113.54 | 2962.59 | 3142.05 | 2997.25 |
| | $\Delta_y$ | 3.37 | 9.74 | 17.49 | 28.21 | 39.02 | 54.88 | 70.45 |
| | $\Delta$ | 9.96 | 18.24 | 32.61 | 45.58 | 60.56 | 76.10 | 89.81 |
| 10 | Period | 0.37 | 0.69 | 1.15 | 1.57 | 2.16 | 2.69 | 3.23 |
| | $V_d$ | 689.39 | 1128.96 | 1249.42 | 1342.61 | 1419.60 | 1485.84 | 1544.20 |
| | $V_y$ | 3220.49 | 4086.61 | 4228.91 | 4438.15 | 4109.37 | 3909.55 | 3837.53 |
| | $\Delta_y$ | 3.97 | 9.08 | 16.99 | 26.65 | 35.04 | 43.82 | 52.80 |
| | $\Delta$ | 10.26 | 17.80 | 30.21 | 45.32 | 57.62 | 68.93 | 80.10 |

(b) Ordinary concentric braced frames

| Span length (m) | | # of stories | | | | | |
|---|---|---|---|---|---|---|---|
| | | 3 | | 6 | | 9 | |
| | | 1.5% Drift | 2.0% Drift | 1.5% Drift | 2.0% Drift | 1.5% Drift | 2.0% Drift |
| 6 | Period | 0.43 | | 0.85 | | 1.39 | |
| | $V_d$ | 537.75 | | 886.70 | | 988.53 | |
| | $V_y$ | 739.58 | 707.57 | 1406.47 | 1180.93 | 1397.30 | 1303.77 |
| | $\Delta_y$ | 1.29 | 1.24 | 5.95 | 4.99 | 11.70 | 10.92 |
| | $\Delta$ | 6.32 | 8.18 | 12.19 | 16.13 | 20.22 | 26.11 |
| 8 | Period | 0.42 | | 0.73 | | 1.20 | |
| | $V_d$ | 719.77 | | 1187.84 | | 1322.50 | |
| | $V_y$ | 1062.35 | 981.99 | 1825.24 | 1493.26 | 1743.65 | 1645.02 |
| | $\Delta_y$ | 1.25 | 1.16 | 4.19 | 3.43 | 7.95 | 7.49 |
| | $\Delta$ | 6.07 | 7.90 | 11.28 | 15.16 | 15.14 | 20.93 |
| 10 | Period | 0.37 | | 0.66 | | 1.12 | |
| | $V_d$ | 910.68 | | 1502.87 | | 1675.10 | |
| | $V_y$ | 1428.47 | 1261.42 | 2318.14 | 1981.09 | 2551.42 | 2339.41 |
| | $\Delta_y$ | 1.03 | 0.92 | 3.35 | 2.86 | 7.53 | 6.90 |
| | $\Delta$ | 5.94 | 7.80 | 11.15 | 14.88 | 18.14 | 24.23 |

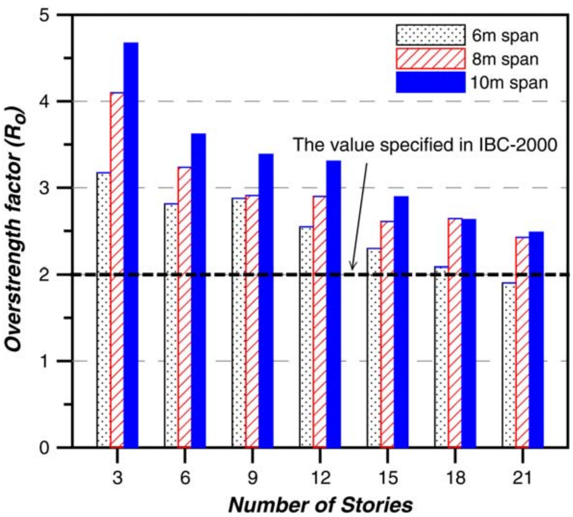
The following equation is for a rock site:

$$R_\mu = \frac{\mu - 1}{\Phi} + 1$$
$$\Phi = 1 + \frac{1}{10T - \mu T} - \frac{1}{2T} e^{-1.5(\ln(T) - 0.6)^2} \qquad (6)$$

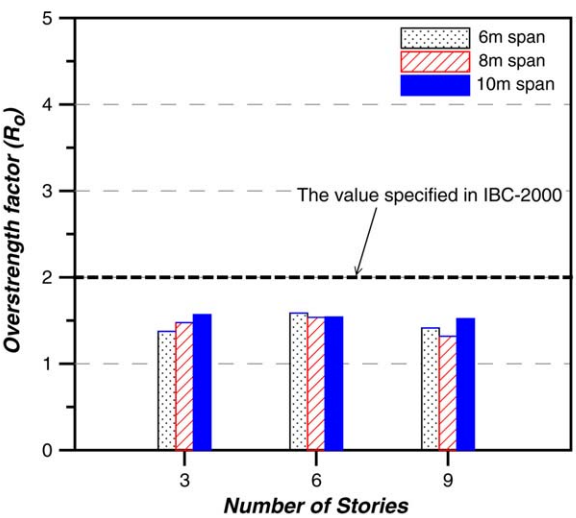
where $\Phi$ is a coefficient reflecting a soil condition. The system ductility ratio $\mu$ is obtained by dividing the roof displacement at the limit state by the system yield displacement.

Fig. 12 plots the inter-story drift and story shear force relationship in each story of the nine-story SCBF and OCBF structures with span length of 6 m. Based on the figures the story ductility ratios were computed and presented in Fig. 13. It can be observed in Fig. 13 that the maximum story ductility ratio becomes 3.55 and 7.04 for SCBF and OCBF, respectively, when the maximum inter-story drift reaches 2% of the story height. In the case of OCBF, the story ductility ratio exceeds 5.0 specified in SEAOC [24] for the limit state of the collapse prevention performance level. This is contributed not only from the buckling of the braces but also from the plastic deformation in beams caused by the unbalanced force between the tensile yielding and compressive buckling force of braces.

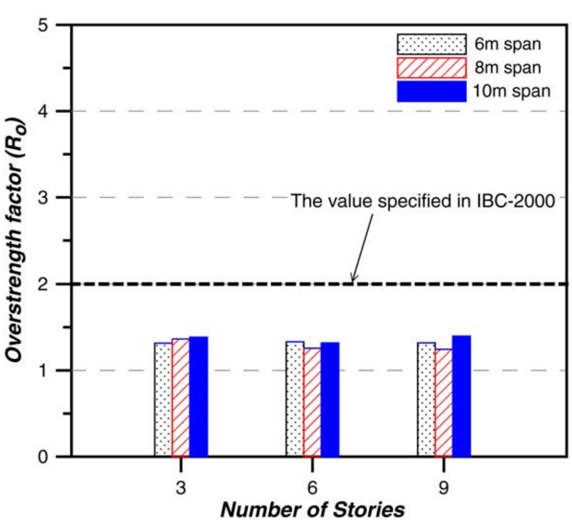
Figs. 14 and 15 show the system ductility factor $R_\mu$ of braced frames when the maximum story drift ratio reaches 2.0% for the SCBF and 1.5 and 2.0% for the OCBF. In most cases the factors computed by Miranda and Bertero turned out to be larger than those by Newmark and Hall.




(a) SCBFs.

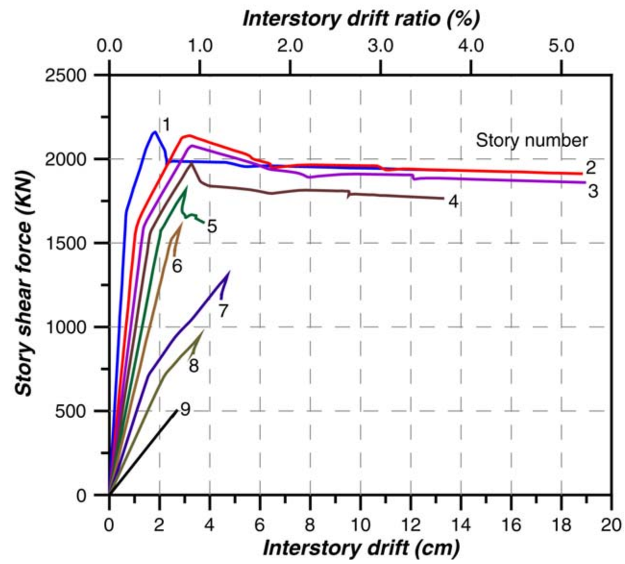



(b) OCBFs (1.5% inter-story drift).

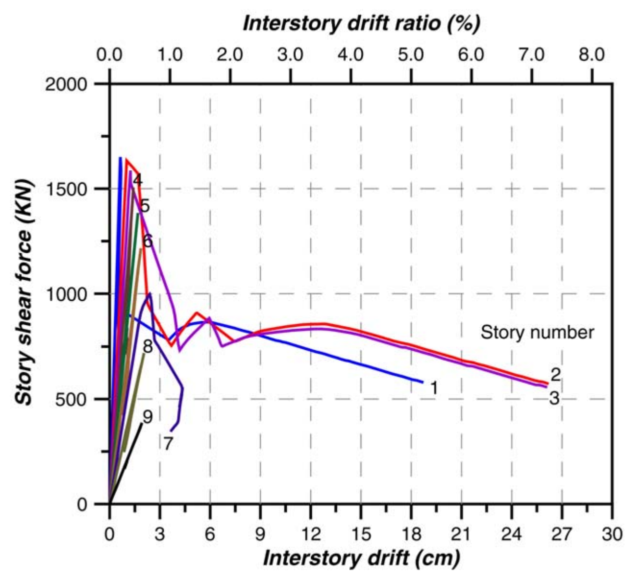



(c) OCBF (2.0% inter-story drift).

Fig. 11. Overstrength factors of model structures.



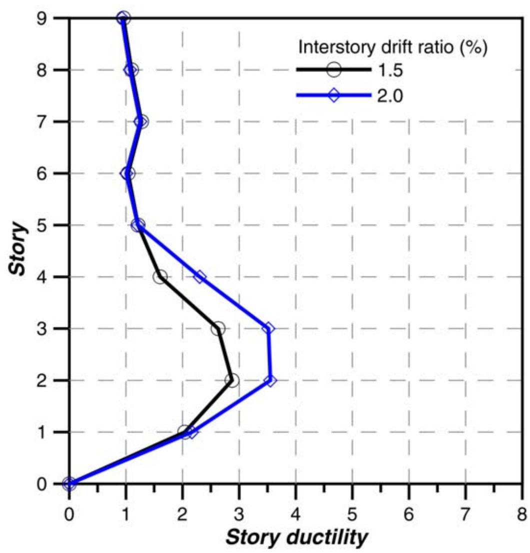

(a) SCBF.



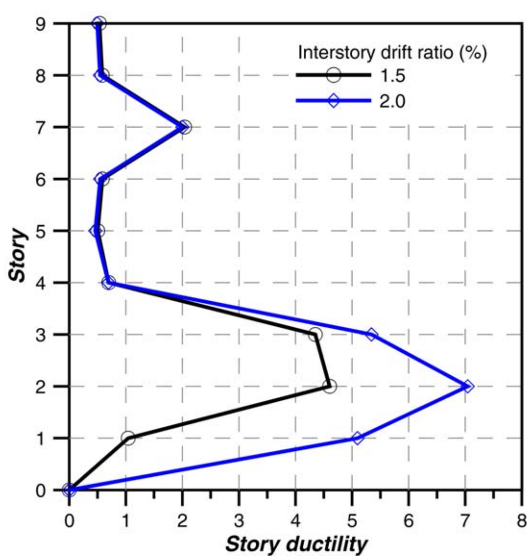

(b) OCBF.

Fig. 12. Inter-story drift and story shear force relationship of nine-story structures (6 m span).

It can be observed that in both SCBF and OCBF the ductility factors decrease as the span length decreases and the height of building increases except for the three-story OCBF computed using Miranda and Bertero's equation. This can be understood in reference to Fig. 16, which shows that the ductility factor computed by Miranda and Bertero increases as the ductility ratio increases up to 6 (up to 8 for longer period structure), but then decreases in further increase of ductility ratio. As the ductility ratios of the three-story OCBF are as large as 6.8 and 8.4 for the bay length of 8 and 10 m, respectively, the ductility factor $R_\mu$ decreases to 4.4 and 3.8, respectively.

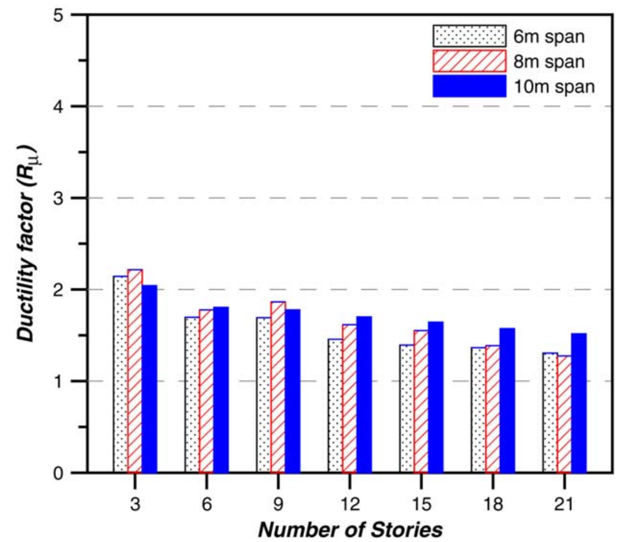



(a) SCBF.

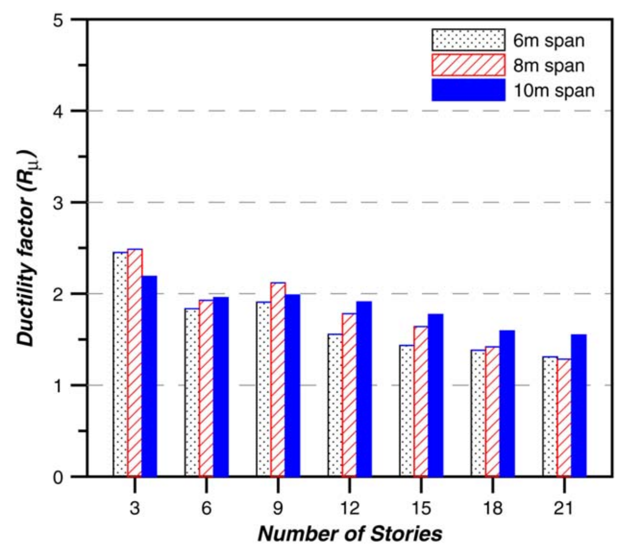



(b) OCBF.

Fig. 13. Story ductility ratios of the nine-story structures with 6 m span length.




(a) Newmark and Hall.



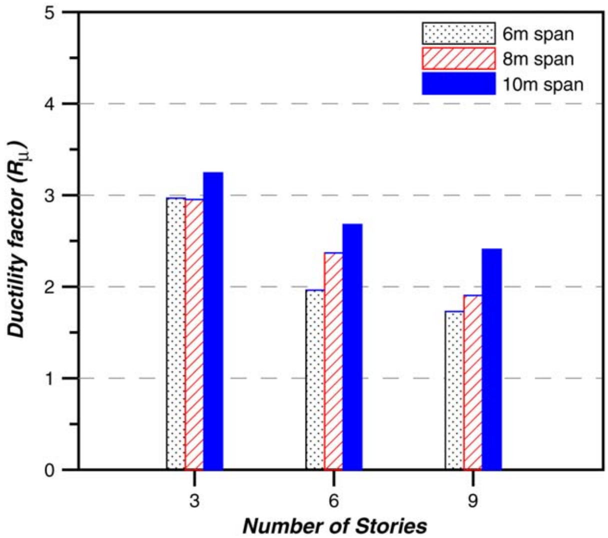

(b) Miranda and Bertero.

Fig. 14. System ductility factors of SCBFs.

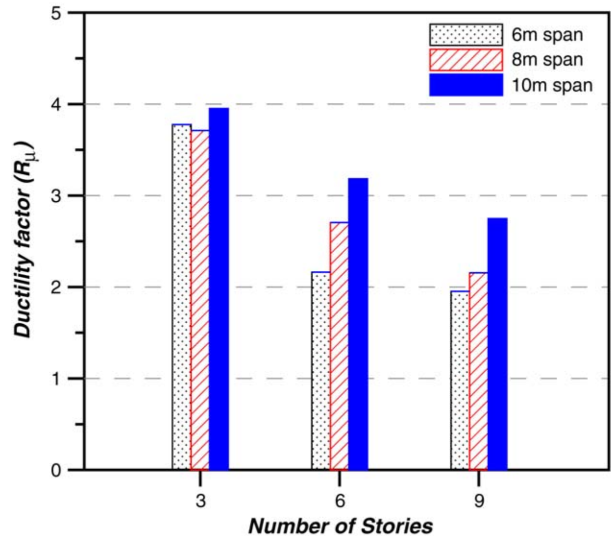
### 4.5. Response modification factors

The response modification factors, presented in Figs. 17 and 18, are computed by multiplying the overstrength and the ductility factors obtained in the previous sections. In SCBF the response modification factors are obtained when the maximum inter-story drift ratio reaches 2.0%, while the factors at 1.5% and 2.0% inter-story drifts are obtained in OCBF. It can be observed that the response modification factors decrease as the span length decreases and the height of the building increases. In the three- and six-story SCBF structures the response modification factors turns out to be larger than 6 which is prescribed in IBC 2000, and in higher structures the factors are generally less than 6. For OCBF only the three-story structure with 10 m bay length has response modification factors larger than the IBC 2000 specified value of 5. The response modification factors obtained at 2.0% maximum inter-story drift are slightly larger than those obtained at 1.5% inter-story drift. Considering that most OCBFs reach the limit state of the collapse prevention stage, as observed in Figs. 9 and 13, it would be reasonable to compute the response modification factor at 1.5% maximum inter-story drift.

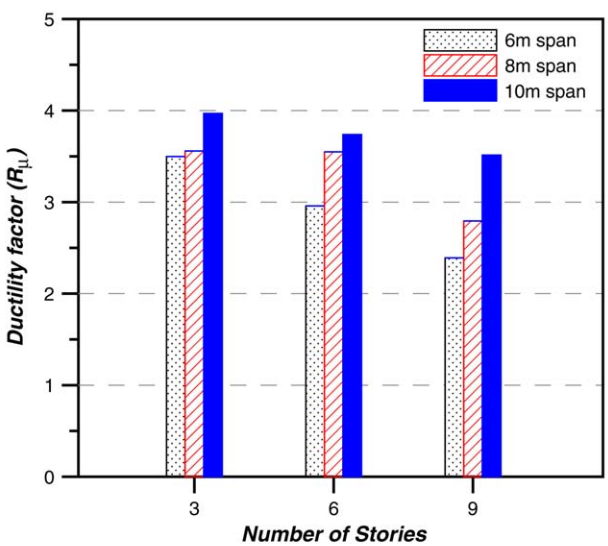
The response modification factors for OCBFs obtained in this study are somewhat smaller than those obtained by Balendra and Huang [5] with 3-, 6-, and 10-story inverted-V-braced frames, which are 8.52, 5.23, and 3.74, respectively. The ductility factors are quite similar in both studies; however the overstrength factors of their study, which range from 2.48 to 5.57, are larger than those of the current study.



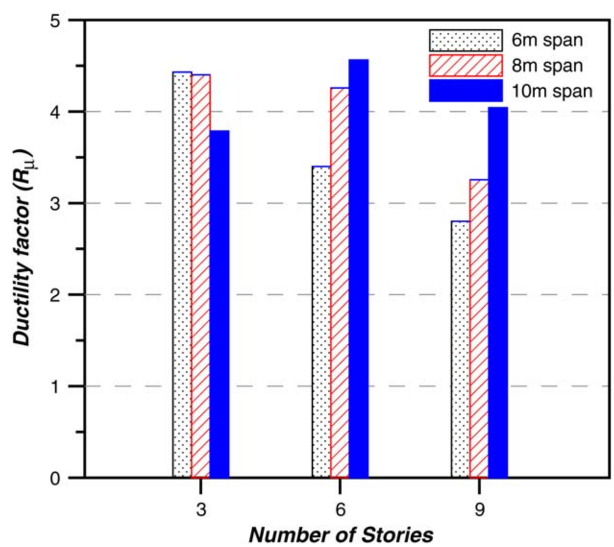

(a) Newmark and Hall (1.5% drift).




(b) Miranda and Bertero (1.5% drift).




(c) Newmark and Hall (2.0% drift).




(d) Miranda and Bertero (2.0% drift).

Fig. 15. System ductility factors of OCBFs.

These differences can be explained by the difference in seismic load used in the structural design. The ratios of base shear to seismic gravity load, $V_b/W_g$, of their analysis models are 1.4%, 1.4%, and 2.2% for the 3-, 6-, and 10-story structures, respectively; while those of our models are 12.5%, 10.2%, and 7.6% for the 3-, 6-, and 9-story OCBF structures, respectively. This implies that their models are assumed to be located in a low-seismic region, while the study models of the current study are designed for higher seismic loads. These results match with findings of Jain and Navin [28] that the overstrength factors of structures in a low-seismic region are five times as large as those in high-seismic region. Therefore it can be concluded that the structure designed for relatively low seismic load tends to have higher residual strength for seismic load.

## 5. Comparison with incremental dynamic analysis results

A series of incremental dynamic analyses were performed until all the predefined limit states were exceeded in order to verify the results of static analyses. Among the time history records developed for the SAC project [29], six records which match well with the design spectrum ($S_{DS} = 0.5$ g, $S_{D1} = 0.3$ g) were selected for dynamic analyses. The response spectra and the design spectrum are depicted in Fig. 19. Inelastic time-history analyses were carried out using the six-story SCBF model structure with 8 m span length using the program SNAP-2DX [30], and the dynamic pushover envelopes were obtained by plotting the point corresponding to the maximum base shear and the

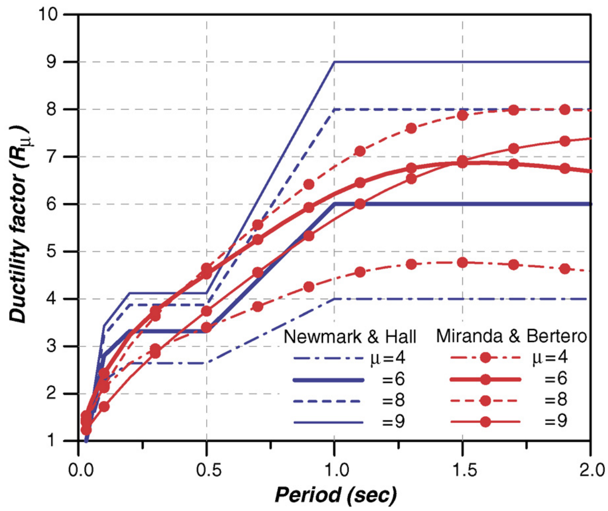
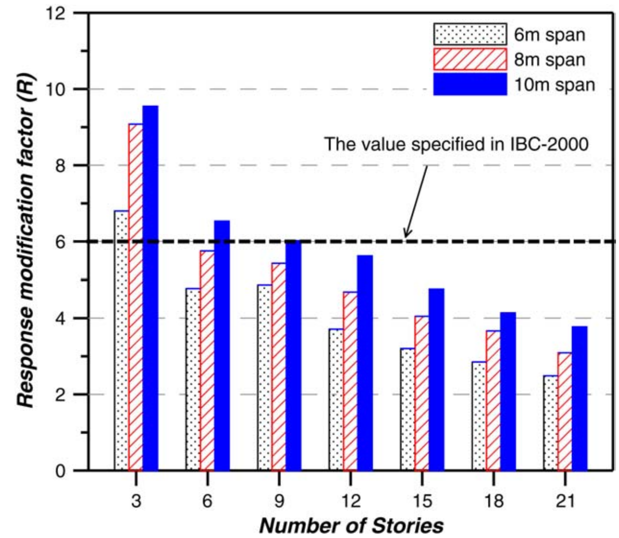
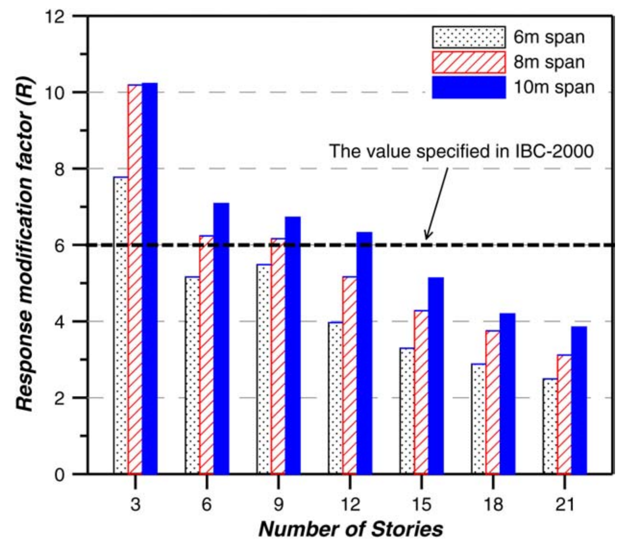



Fig. 16. Comparison of ductility factors computed from Newmark and Hall and Miranda and Bertero's equations.




(a) Newmark and Hall.




(b) Miranda and Bertero.

Fig. 17. Response modification factors of SCBFs.

maximum top-story displacement computed for each scaled record. The intensities of the time history records were varied by multiplying appropriate scaling factors. The dynamic pushover envelopes were compared with the static pushover curve in Fig. 20, which shows that the dynamic envelopes form upper bound for displacement larger than the yield point.

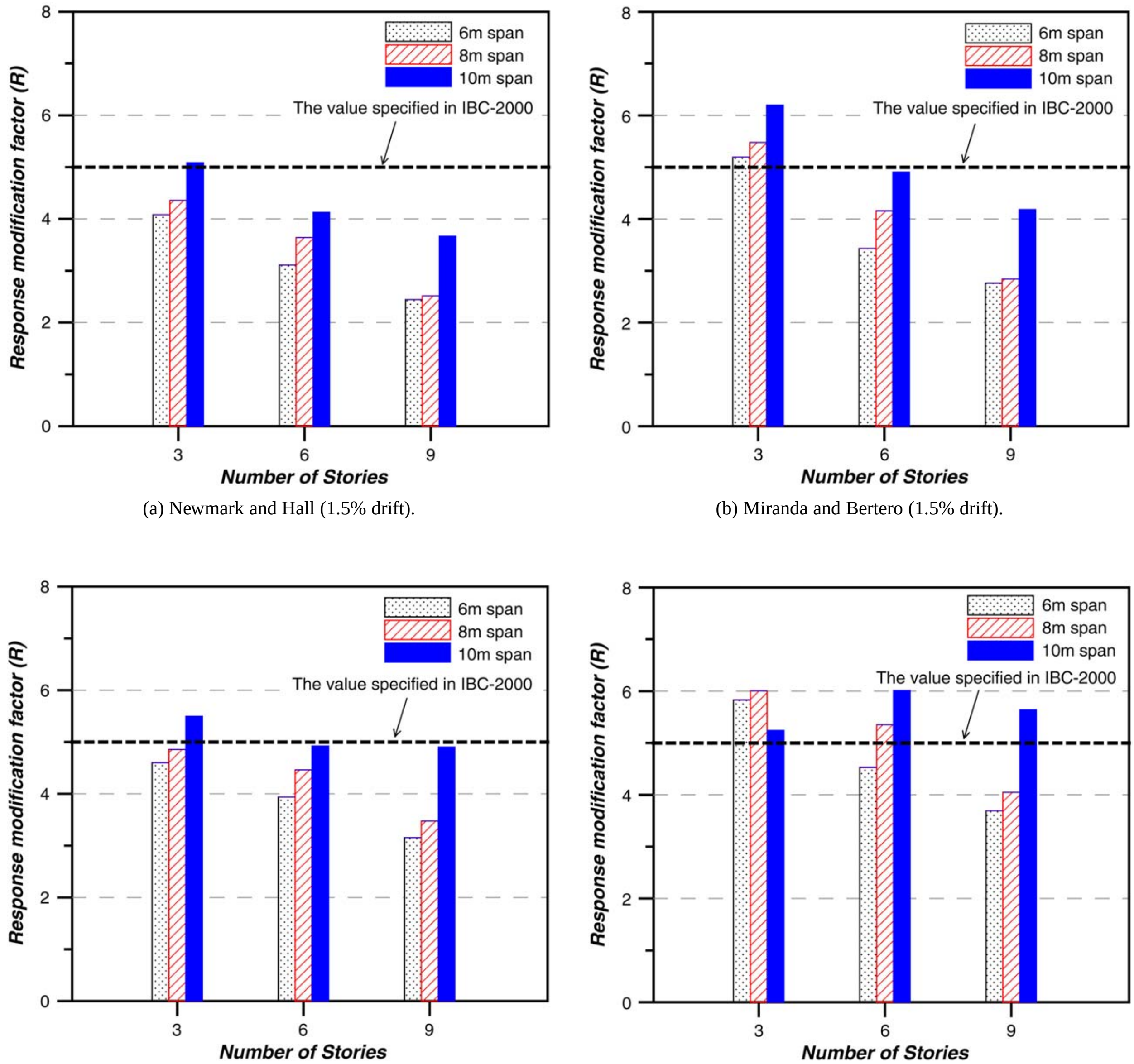
To obtain behavior factors, the six dynamic pushover envelopes were averaged and the average curve was fitted into a bi-linear curve. The overstrength factor obtained in this way is 2.88 which is 11% smaller than the factor obtained from the static pushover curve. However the ductility factor computed for each record using the Newmark and Hall procedure ranges from 1.65 to 2.13 with the mean value of 1.99, which is larger than 1.78 obtained from the static pushover curve. Consequently the response modification factor results in 5.76–6.14 with the mean value of 5.75, which is almost identical to the value obtained from static pushover analysis (which is 5.74).

## 6. Conclusions

The overstrength, ductility, and the response modification factors of the 21 special concentric braced frames and 9 ordinary concentric braced frames with various stories and span lengths were evaluated by performing pushover analyses. Some of the results were compared with those from nonlinear incremental dynamic analyses. The model structures were designed for relatively large seismic load and the beam–column connections were assumed to be pinned so that the seismic load was resisted mainly by braces. Such design conditions are expected to produce somewhat conservative results for response modification factors. The results of this study can be summarized as follows:

(1) The overstrength factors increased as the structure height decreased and the span length increased. In SCBFs, the factors turned out to be 1.9–3.17 for 6 m span, 2.43–4.10 for 8 m span, and 2.49–4.67 for 10 m span, which are generally larger than the IBC 2000 specified value of 2.0. In OCBFs, however, the factors ranged between 1.32 to 1.59 (1.5% maximum inter-story drift ratio) and 1.24 to 1.40 (2.0% drift ratio), which are significantly smaller than 2.0. The underestimation of overstrength factors in design codes may lead to unsafe design by underestimating the seismic force transferred to a critical element from the other elements of the lateral force resisting system.

(2) The ductility factors were obtained as 1.28–2.2 (Newmark and Hall procedure) and 1.29–2.49 (Miranda and Bertero) for SCBFs with limit state of 2% maximum inter-story drift ratio, and as 1.73–3.24 (Newmark and Hall) and 1.95–3.95 (Miranda and Bertero) for OCBFs with 1.5% maximum drift ratio. As in the case of overstrength factors, the ductility factors increased




(a) Newmark and Hall (1.5% drift).

(b) Miranda and Bertero (1.5% drift).

(c) Newmark and Hall (2.0% drift).

(d) Miranda and Bertero (2.0% drift).

Fig. 18. Response modification factors of OCBFs.

as the structure height decreased and the bay length increased.

(3) The response modification factors were in the ranges 2.49–6.8 (6 m span), 3.01–9.08 (8 m span), and 3.77–9.55 (10 m span) for SCBFs, and 2.44–5.09 for OCBFs when the Newmark and Hall procedure was applied to compute ductility factors. As the response modification factors were computed by multiplying overstrength and ductility factors, they also increased as the structure height decreased and the span length increased. In SCBFs the response modification factors turned out to be smaller than the code-specified value of 6.0 in most model structures except the three-story structures. The response modification factors were less than the code value of 5.0 in all OCBF model structures.

(4) The maximum base shear envelopes obtained by incremental dynamic analyses generally formed an upper bound to the static pushover curve. The response modification factors obtained from the two different procedures turned out to be similar.

It turned out that the earthquake-resisting capacity of braced frames, especially OCBFs, was generally less than the level specified in a design code such as IBC 2000. However, considering the fact that braced frames have superior load-resisting capacity as long as the compression braces do not buckle, it would be reasonable to design brace frames as rather more strength based by reducing the response modification factor. In fact, the response modification factors for the intermediate steel moment

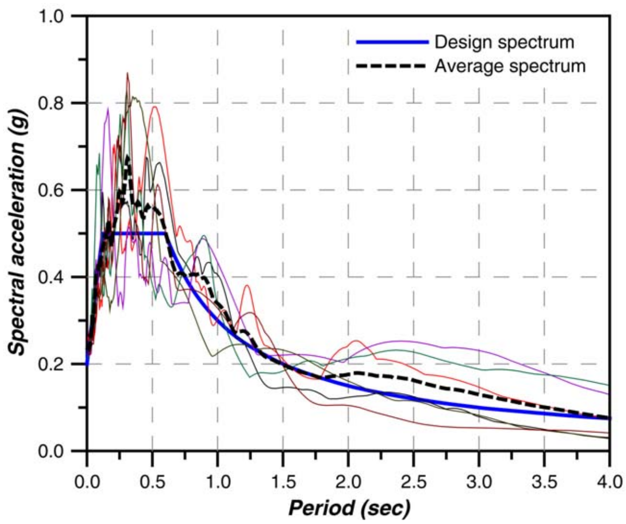



Fig. 19. Design spectrum and response spectra of selected earthquake records.

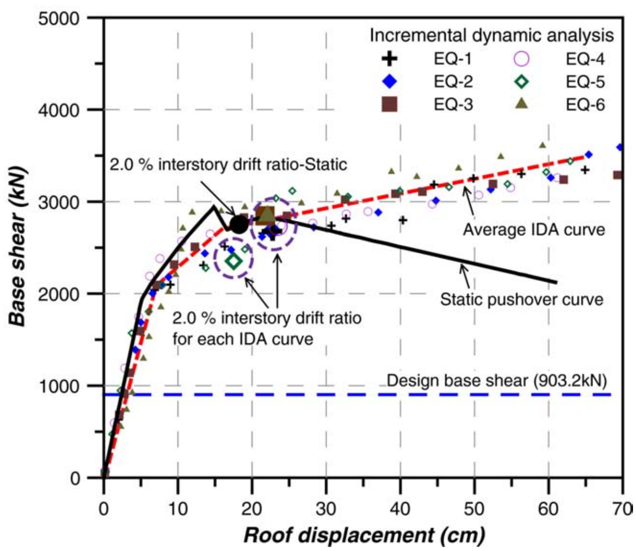



Fig. 20. Static and dynamic pushover curves of the six-story SCBF.

frames and the ordinary steel moment frames specified in FEMA-302 [31] were 6 and 4, respectively; however, based on new findings from recent research, they were reduced to 4.5 and 3.5, respectively, in FEMA-368 [32]. Also the response modification factors need to be defined in various performance levels considering seismic hazard levels, number of stories, target ductility ratios, etc. In this regard further research still needs to be performed considering various design variables to propose more reasonable behavior factors for concentric braced frames.

## Acknowledgement

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