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New Approach to Evaluate the Response Modification Factors for Steel Moment Resisting Frames

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Abstract

The design force levels currently specified by most seismic codes are calculated by dividing the base shear for elastic response by the response modification factor (R). This is based on the fact that the structures possess significant reserve strength, redundancy, damping and capacity to dissipate energy. This paper proposed the evaluation methodology and procedure of the response modification factors for steel moment resisting frames. The response modification factors are evaluated by multiplying ductility factor (R_μ) for SDOF systems, MDOF modification factor (R_M) and strength factor (R_S) together. The proposed rules were applied to existing steel moment resisting frames. The nonlinear static pushover analysis was performed to estimate the ductility (R_μ), MDOF modification (R_M) and strength factors (R_S). The results showed that the response modification factors (R) have different values with various design parameters such as design base shear coefficient (V/W), failure mechanism, framing system and number of stories.

Keywords: response modification factor, pushover analysis, MDOF modification factor, steel moment resisting frame

1. Introduction

Seismic codes have long relied upon the concept of inelastic spectrum for specifying design forces to be used for elastic analysis of structures which are expected to respond inelastically to the design earthquakes. Thus, the design base shear is calculated by dividing the base shear for elastic response by the response modification factor (R). The concept of response modification factor (R) was proposed based on the premise that well-detailed seismic framing systems could sustain large inelastic deformations without collapse and develop lateral strengths exceeding their design strength. The response modification factor (R) was assumed to represent the ratio of forces that would develop under the specified ground motion if the framing system were to behave entirely elastically with respect to the prescribed design forces at the strength level.

The response modification factor (R) specified in seismic design codes mostly depend on committee consensus based on the observed performance of buildings during past earthquake. The response modification factors (R) have been the subjects of investigations by various researchers. Balendra and Huang (2003) found that the response modification factors decreased when the number of stories increased. For 3-, 6- and 10-stories braced frame, they found that the response modification factors varied from 8.5 to 3.5. Maheri and Akbari (2003) investigated the response modification factors of steel braced reinforced concrete framed dual systems. The effects of some parameters influencing the value of R factor, including the height of the frame, share of bracing system from the applied load and the type of bracing system were investigated. They found that the addition of steel X and knee braces significantly increase the response modification factor; hence the number of stories appeared to be the predominant variable. Kim and Choi (2005) evaluated the overstrength, ductility and response modification factors of the chevron type concentric braced frames by performing pushover analysis of model structures with various stories and span lengths. Galíndez and Thomson (2007) estimated the response modification factors for Colombian code compliant steel moment frame buildings of different heights (3-, 7-, and 11 stories), located in a high-risk seismic region. The response modification factor was evaluated and found to be five times lower than that recommended by the code for a seismic design demand level. Asgarian and Shokrgazar

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(2009) evaluated the overstrength, ductility and response modification factor of buckling restrained braced frames with various stories and different bracing configurations. They performed static pushover analysis, nonlinear incremental dynamic analysis and linear dynamic analysis. They found that overstrength, ductility and response modification factors are decreased as the number of stories are increased. Mahmoudi and Zaree (2010) evaluated the response modification factors of conventional concentric braced frames and buckling restrained braced frames. They found that response modification factors have different values for brace configuration types, number of bracing bays and buildings height. According to the results of the above studies, the response modification factor appears to be a period and applied load-dependent factor. A single value of response modification for all buildings of a given framing type, irrespective of building height, plan geometry and framing layout, could not be justified. Despite the profound influence of response modification factors (R) on the seismic design process, and ultimately on the seismic performance of buildings, no sound technical basis exists for the value of response modification factors (R) tabulated in seismic design codes (ATC-19 and ATC-34, 1995). There is an obvious and pressing need to develop a rational technical basis for response modification factors (R) if equivalent lateral force design procedures are to be retained for seismic design.

The main objective of this work is to suggest the simplified evaluation methodology of the response modification factors (R) for steel moment resisting frames. Empirical expressions to evaluate the ductility factor (R_μ) for SDOF systems and MDOF modification factors (R_M) were previously developed by the authors (Kang and Choi, 2010). Based on this precedent studies, simplified expressions and procedures to evaluate the response modification factors (R) for steel moment resisting frames are suggested in this study. Several researchers performed the nonlinear incremental dynamic analysis or time history analysis to evaluate the ductility and strength factors (Kim *et al*., 2005; Galíndez *et al*., 2007; Asgarian *et al*., 2009; Kurban and Topkaya, 2009). It would be required a lot of effort and time to carry out the nonlinear dynamic or time history analysis. For practical purposes, a simplified methodology would be useful to estimate the response modification factor for existing or now structures. In this study, nonlinear static pushover analysis, relatively simple, was carried out to compute the ductility factor (R_μ) for SDOF systems, MDOF modification factor (R_M) and strength factor (R_S). The response modification factors (R) for the existing steel moment resisting frames were evaluated to verify the applicability of proposed procedures.

2. Response Modification Factor

In regard to response modification (R) factors, first introduced in ATC-3-06 (ATC, 1978), ATC-3-0-6 noted that:

- R factors were intended to reflect reduction in design force values that were justified on the basis of risk assessment, economics, and nonlinear behavior.
- The intent was to develop R factors could be used to reduce expected ground motions presented in the form of elastic response spectra to lower design levels by bringing modern structural dynamics into the design process.

Response modification factors (R) play a key, but controversial, role in the seismic design process. No other parameter in the design base shear equation impacts the design actions in a seismic framing system as does the value assigned to response modification factors (R). Several researchers have suggested the evaluation expressions of response modification factors (R) as shown in Table 1.

In Table 1, R_μ is the period dependent ductility factor, R_S is the strength factor, R_ξ is the damping factor, and R_R is the redundancy factor.

Figure 1 represents the base-shear versus roof displacement relation of a structure, which can be idealized by a bilinear elasto-perfectly plastic relation. In this figure, V_e correspond to the elastic response strength of the structure. The maximum base shear in an elasto perfectly plastic behavior is V_0. The ductility factor (R_μ) is defined as the ratio of elastic response strength V_e to maximum base shear in actual behavior V_0.

$$R_\mu = \frac{V_e}{V_0} \tag{1}$$

The strength factor (R_S) is defined as the ratio of maximum base shear in actual behavior V_0 to design base shear V_d.

$$R_s = \frac{V_0}{V_d} \tag{2}$$

Figure 1 showed the example of the evaluation of response modification factor for 8-story SCWB perimeter frame, where soil profile is Sc and seismic zone factor is 0.2.

A large number of earthquake records are required to evaluate the response modification factors using the Eq. (1) and Table 1. The linear and nonlinear dynamic analysis should be carried out to calculate the ductility factors. The main shortcoming of evaluation of response modification factors listed in Table 1 is the absence of simplified rules, specific procedure and methodology. It

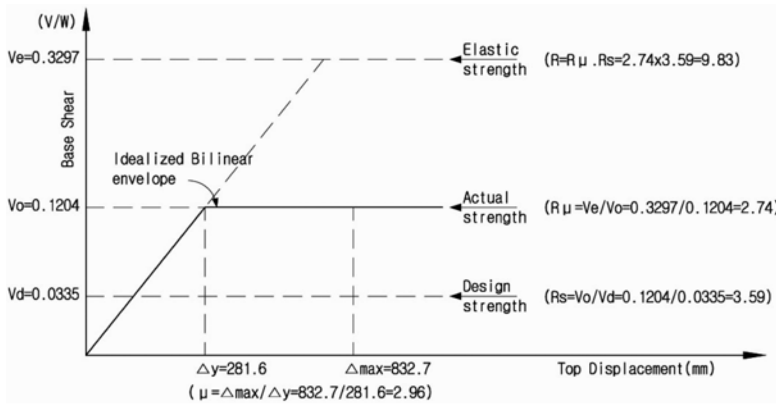


Figure 1. Example of the evaluation of response modification factor.

Table 1. Evaluation of response modification factor

	Component of R factor
Uang and Bertero (1986) and Whittaker *et al.* (1987)	$R=R_\mu R_S R_\zeta$
Freeman (1990) and Uang (1991)	$R=R_\mu R_S$
ATC-19 (1995)	$R=R_\mu R_S R_R$

would be desirable to suggest the simplified expressions and procedures to evaluate the response modification factors (R) for practical purpose.

3. New Approach to Evaluate the R Factor

3.1. Ductility factors for SDOF systems

The ductility factor (R_μ) accounts for the non-linear characteristics of structures and plays the most important role in the determination of response modification factor (R). The ductility factor (R_μ) for SDOF systems is defined as the ratio of the elastic strength demand to the inelastic strength demand, as shown in the following formula:

$$R_\mu = \frac{F_y(\mu=1)}{F_y(\mu=\mu_i)} \tag{3}$$

where F_y (μ=1) is the lateral strength required to avoid yielding in the system under a given ground motion, and F_y ($\mu=\mu_i$) is the lateral strength required to keep the displacement ductility ratio demand, μ, less than or equal to a pre-determined target ductility ratio, μ_i, subject to the same ground motion.

To compute the ductility factors (R_μ), earthquake ground motions recorded from 47 earthquakes were collected from the National Geology Data Center (NGDC) in the U.S. and the Pacific Earthquake Engineering Research Center (PEER) in the University of California in Berkeley. A complete list of earthquake ground motions used in this study can be found in Kang (2003). The ground motions were classified into four groups according to the average shear wave velocity at the recording station, $\overline{v}_s$, as follows:

(a) Site AB: $\overline{v}_s$ ≥750 m/s, (b) Site C: 360 m/s≤$\overline{v}_s$ ≤750 m/s, (c) Site D: 180 m/s≤$\overline{v}_s$ ≤360 m/s and (d) Site E: $\overline{v}_s$ <180 m/s. These site classifications are consistent with current building codes, especially the ASCE/SEI 7-05 (ASCE/SEI 7-05, 2007), the National Earthquake Hazard Reduction Programs (FEMA 302, 1997) and the International Building Code 2000. A non-linear time history analysis was carried out on SDOF systems. The target ductility values were selected the elastic, 2, 3, 4, 5, and 6. The inelastic response spectrum was computed for a set of 60 discrete periods that ranged from 0.05 to 3.0 seconds.

The results of preceding studies showed that the ductility factor depends strongly on the target displacement ductility ratio (μ), the period (T), and the site conditions (Kang and Choi, 2002; 2004). A simplified expression is desired to consider the ductility factor (R_μ)-displacement ductility ratio (μ)-period (T) relationship for each site condition. The approximate ductility factor for SDOF systems ($R_{\mu,SDOF}$) was suggested by

$$R_{\mu,SDOF} = 1 + \frac{T}{\phi} \tag{4}$$

where ϕ is a function of the displacement ductility ratio (μ), the period (T), and the site conditions (Kang and Choi, 2010). For each site condition, the functions ϕ that fit best mean ductility factors were given by Appendix.

3.2. Modification of the MDOF systems

The ductility factors (R_μ) previously discussed can be used for the design of structures that can be approximately modeled such as SDOF systems. Most structures, however, need to be modeled as multi-degree-of-freedom (MDOF) systems. They have a much more complex behavior than SDOF systems, particularly in the non-

Table 2. Some sources of strength factors given in the literature

1. Effects of discrete member sizes.
2. Effects of underestimating member strength capacities in the design process (e.g., conservative models for predicting member strength, actual vs. nominal material strength properties.
3. Effects of code minimum sizes and requirements.
4. Effects of stiffness requirements on member strength.
5. Effects of desired uniformity of members for constructability.
6. Architectural considerations.
7. Effects of non-structural elements that are not considered as part of the lateral load resisting system.
8. Code-calculated period and related base shear.
9. Importance of building.
10. Assumed lateral load distribution.
11. Design controlled by other loading case. (e.g., wind)
12. Effects of other loads in load combinations. (e.g., effects of brevity loads on the required member strength)
13. Redistribution of internal forces in the inelastic range.

linear range. Thus, the ductility factor of SDOF systems must be modified for the design of MDOF systems.

The modification factor (R_M) is defined as follows:

$$R_M=\frac{R_{\mu,MDOF}}{R_{\mu,SDOF}}=\frac{V_y,(\mu=1)/V_y,(\mu=\mu_i)}{F_y,(\mu=1)/F_y,(\mu=\mu_i)}=\frac{F_{y,SDOF}}{V_{y,MDOF}} \tag{5}$$

where $R_{\mu,MDOF}$ is the ratio of the elastic strength demand to the inelastic strength demand in the MDOF structure.

Nassar and Krawinkler (1991) and Miranda (1997) provided some of the answers to the assessment of the strength demands of inelastic MDOF systems for their comparison with their SDOF counterparts. The modification factor (R_M) was proposed to account for MDOF systems, based on these previous studies. The MDOF modification factors (R_M) were given by Appendix for the strong-column weak-beam (SCWB) and weak-column strong-beam (WCSB) models.

3.3. Strength factors

Observations of structural performance under many past earthquake have led to the conclusion that code designed buildings must possess significant strength factors in order for them to have survived without damage when earthquake forces are considerably larger than those considered in design (Hwang and Shinozuka, 1994; Uang and Maarouf, 1993). Therefore, it is necessary that strength factors be quantified and included in the seismic design process. Many researchers have attempted to identify the factors that may have contributed to the observed strength factors. Table 2 lists some of the sources of strength factors mentioned in the literature (Freeman 1990; Osteraas 1990; Uang and Maarouf, 1993; Sudhir and Navin, 1995).

Non-linear static analysis (also termed pushover analysis) can be used to estimate the strength of a building or framing system. The procedure used to estimate the strength of a building was straightforward, but required the analyst to select a limiting state of response. Typical limiting responses include maximum inter-story drift and maximum plastic hinge rotation. For the *Immediate Occupancy Performance Level*, a drift level less than 0.01 are desirable. This limit is selected because steel frames normally experience their significant yielding at an inter-story drift ratio of between 0.005 and 0.01. Steel is ductile material and no significant damage is expected at the 0.01 drift level. The practical drift limit for the *Life Safety* and *Collapse Prevention* performance might have been 0.02 and 0.04, respectively (FEMA 274, 1997).

The steps in the procedure are as follows:

- Using nonlinear static analysis, construct the base shear-roof displacement relation-ship for the building.
- At the roof displacement corresponding to the limiting state of response, calculate the base shear force (V_0) in the building. The reserve strength of the building is equal to the difference between the design base shear (V_d) and V_0.
- Calculate the strength factor using the equation (2).

3.4. New formula and procedure to evaluate the R factors

In this study, a new expression to evaluate the response modification factor was suggested in following manners.

$$R=R_\mu\times R_M\times R_S \tag{6}$$

In the above equation, ductility factor (R_μ) is calculated from Eq. (4) considering the soil conditions. The modification factor to account MDOF effects is calculated from Appendix. Nonlinear static pushover analysis was carried out to compute the ductility factor (R_μ), MDOF modification factor (R_M), and strength factor (R_S). Kim and Choi (2005) found that the results of incremental dynamic analysis generally matched well with those obtained from static pushover analysis.

The simplified procedures to estimate the response modification factors (R) were as follows.

- Calculate the period (T) from code expression or

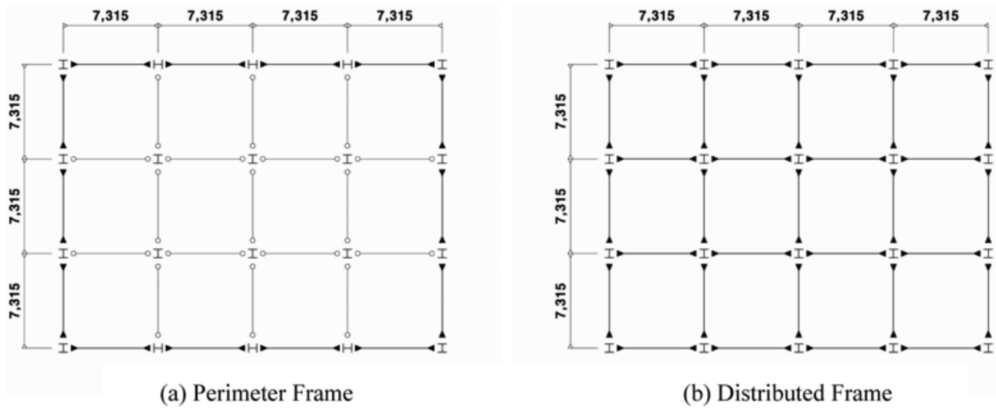


Figure 2. Perimeter and distributed frames (unit: mm).

eigenvalue analysis.

- Calculate the displacement ductility ratio (μ) using nonlinear static analysis.
- Evaluate the ductility factor (R_μ) for SDOF system and MDOF modification factor (R_M).
- Estimate the strength factor (R_S) from nonlinear static analysis.
- The response modification factors (R) are calculated by multiplying ductility factor (R_μ) for SDOF systems, MDOF modification factor (R_M) and strength factor (R_S) together.

4. Application to Special Steel Moment Resisting Frames

4.1. Model frames

The proposed expression to evaluate the response modification factor (R) was applied the existing special steel moment resisting frames. A complete list of member identification and size could be found in Kang's paper (Kang, 2003). Each frame was designed and detailed in accordance with UBC-1997 standards (ICBO, 1997) and AISC seismic provisions (AISC, 2002). The frames were 4, 8, and 16 stories tall and story height was 3, 658 mm for the typical stories and 5,486 mm for the bottom story. The frames were regular with the floor plan and geometry and had a 4-bay×3-bay plan, with bay dimensions of 7,315 mm×7,315 mm. The dead loads of all floors were 4.8 kN/m^2. The live loads of floor and roof were 2.4 kN/m^2 and 1.2 kN/m^2, respectively. Models were designed for two structural framing systems: the perimeter frame (*PF*) and the distributed frame (*DF*), as shown in Fig. 2.

For the PF model, the perimeter frames fully resisted the lateral loads, whereas the interior frames resisted the gravity loads. For the DF model, all the frames resisted the lateral loads and gravity loads. The structural design was conducted using the MIDAS-Gen program (2001).

Studies have historically focused on strong-column weak-beam (SCWB) steel frames, because they are more ductile than weak-column strong-beam (WCSB) frames. With regard to SCWB joints, one of the following relationships shall be satisfied for special steel-moment-resisting frames and are assured by UBC-97.

$$\frac{\Sigma Z_C(F_{yc}-P_{uc}/A_g)}{\Sigma Z_b F_{yb}} \geq 1.0 \tag{7}$$

$$\frac{\Sigma Z_C(F_{yc}-P_{uc}/A_g)}{V_n d_b H/(H-d_b)} \geq 1.0 \tag{8}$$

It is sometimes uneconomical or impractical, however, to implement the SCWB behavior at each joint. Consequently, UBC and NEHRP (FEMA 302, 1997) permit the use of WCSB joints under specific conditions. The strength of the joint need not satisfy Eq. (7) or (8) in any of the following cases: columns with $Puc<0.3F_{yc}A_g$, columns in any story that have a ratio of the design shear strength to the design force that is 50% greater than the story above, and any column not included in the design for the resistance of the required seismic shears, but included in the design for the resistance of axial overturning forces.

The WCSB models were designed meeting the columns with $Puc<0.3F_{yc}A_g$. The columns with $Puc<0.3F_{yc}A_g$ for WCSB models were referred to *Exception criteria* in this research. For WCSB models, it would be expected that this systems have a lower response modification factor (R) because of limited ductility. Therefore, WCSB systems would not be designed as a special moment resisting frames in practice. The purpose of the WCSB model was to compare merely the results of SCWB model in this research.

In general, the size of structural member is controlled by *Strength criteria* or *Drift criteria* for SCWB models. In WCSB PF models, some sizes of the structural members were controlled by drift criteria in higher seismic design intensity ranges, but it were controlled by *Exception criteria*, that is, columns with $Puc<0.3F_{yc}A_g$, in lower seismic design intensity ranges. In WCSB DF models, most sizes of the structural members were

Table 3. Some of the stability coefficient and story drift ratio

Story	SCWB PF		SCWB DF		WCSB PF		WCSB DF	
	Stability coeff. (θ)	Story drift ratio	Stability coeff. (θ)	Story drift ratio	Stability coeff. (θ)	Story drift ratio	Stability coeff. (θ)	Story drift ratio
16	0.0032	0.0124	0.0113	0.0117	0.0031	0.0118	0.0101	0.0108
15	0.0057	0.0140	0.0205	0.0135	0.0053	0.0131	0.0177	0.0120
14	0.0071	0.0140	0.0251	0.0135	0.0066	0.0129	0.0214	0.0118
13	0.0076	0.0134	0.0266	0.0127	0.0079	0.0137	0.0245	0.0120
12	0.0079	0.0127	0.0271	0.0118	0.0089	0.0142	0.0264	0.0118
11	0.0089	0.0133	0.0305	0.0124	0.0101	0.0147	0.0260	0.0108
10	0.0099	0.0138	0.0339	0.0129	0.0111	0.0149	0.0251	0.0098
9	0.0106	0.0137	0.0363	0.0131	0.0117	0.0148	0.0250	0.0092
8	0.0110	0.0135	0.0385	0.0131	0.0123	0.0149	0.0246	0.0085
7	0.0117	0.0136	0.0412	0.0133	0.0128	0.0147	0.0237	0.0078
6	0.0123	0.0136	0.0436	0.0134	0.0133	0.0143	0.0227	0.0071
5	0.0128	0.0133	0.0447	0.0130	0.0136	0.0140	0.0221	0.0065
4	0.0132	0.0130	0.0456	0.0126	0.0138	0.0134	0.0214	0.0060
3	0.0137	0.0128	0.0476	0.0125	0.0135	0.0124	0.0209	0.0056
2	0.0143	0.0126	0.0508	0.0126	0.0136	0.0117	0.0211	0.0053
1	0.0130	0.0107	0.0520	0.0122	0.0181	0.0146	0.0263	0.0062

controlled by *Exception criteria* except for extreme lower seismic design intensity ranges. These *Exception criteria* caused the some extreme values of the strength factors (R_S) in WCSB models, especially in lower seismic design intensity ranges.

A total of 108 frames were designed for the following permutations:

- Structures with 4, 8 and 16 stories;
- SCWB and WCSB failure mechanisms;
- Perimeter frame (PF) and distributed frame (DF);
- Site categories with S_A, S_C, and S_E; and
- Seismic zone factor Z=0.075 (Z1), 0.2 (Z2B), and 0.4 (Z4).

P-delta effects on story shears and moments, the resulting member forces and moments, and the story drifts induced by these effects are not required to be considered when the stability coefficient, θ, is equal to or less than 0.1 (ASCE/SEI 7-05 and ICC, 2000).

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d} \tag{9}$$

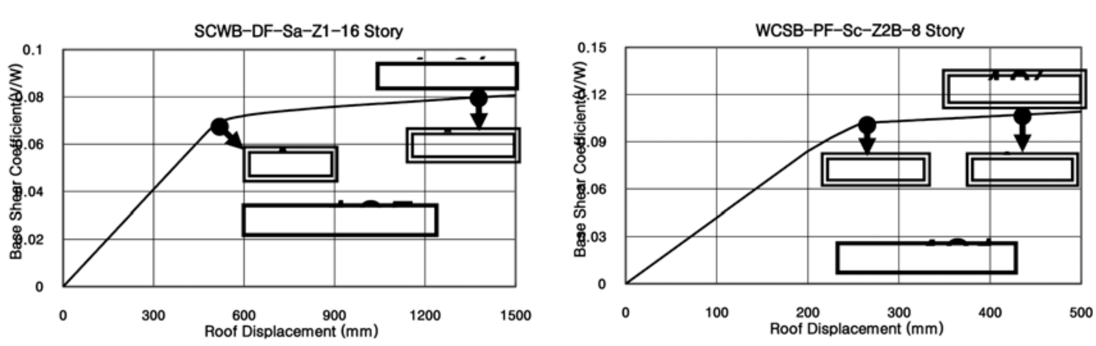
The stability coefficient, θ, shall not exceed $\theta_{\max}$ determined as follows.

$$\theta_{\max} = \frac{0.5}{\beta C_d} \tag{10}$$

The value of $\theta_{\max}$ was calculated as 0.909, taking the value of β=1.0. Table 3 shows the stability coefficient (θ) and story drift ratio for 16-stories, where soil profile is S_E and seismic zone factor is 0.4.

4.2. Nonlinear static analysis

The non-linear static analysis of the frames was performed by subjecting a structure to monotonically



(a) 16 story SCWB distributed frame (b) 8 story WCSB perimeter frame

Figure 3. Some of the displacement ductility ratio for prototype frames.

increasing lateral forces. For that purpose, DRAIN-2D+ computer program was used (Tsai and Li, 1994). In the non-linear static analysis, selecting an appropriate lateral load distribution is an important step. To determine the elastic natural periods and mode shapes of the model frames, eigenvalue analyses were carried out before the pushover analysis. Then pushover analyses were carried out to evaluate the global yield limit state and the frame capacity by progressively increasing the lateral story forces proportional to the fundamental mode shape. For the modeling of frames, the DRAIN-2D+ beam-column element (element 2), with 1% strain hardening, is the primary element used in these analyses. The strength, stiffness and shear distortions of panel zones were not considered. The contribution of the floor slab was not included. The drift limit to estimate the displacement ductility ratio is assumed 0.04 inter-story drift at any story.

It is well-known that the ductility ratio (μ) can be computed at the element, story and system levels. At the element level, ductility ratio can be expressed in terms of strain ductility ratio, curvature ductility ratio and rotation ductility ratio. At the story and system levels, ductility ratio (μ) is normally expressed in terms of the displacement ductility ratio. For the purpose of this study, the displacement ductility ratio at the system level is used to determine the ductility factor (R_μ). It must be recognized that the ductility factor is a measure of the nonlinear response of the complete framing system and not component of the framing system, regardless of which ductility parameter is used. Some of the calculations of displacement ductility ratio for the prototype structures are demonstrated by Fig. 3.

4.3. Evaluation of the R factors for prototype frames

The response modification factor (R) for 16-story SCWB perimeter frame, for example, is estimated as follows, where soil profile is S_C and seismic zone factor is 0.4. From nonlinear static analysis, the displacement ductility ratio (μ) is 3.23 and period (T) is 1.847 sec. The ductility factor (R_μ) for SDOF system is calculated by Eq. (4) as follows.

Table 4. R_m, R_M, R_S and R factors for SCWB models

No	Notation	Perimeter frames				Distributed frames			
		R_μ	R_M	R_S	R	R_μ	R_M	R_S	R
1	4-Sa-0.075	2.38	1.00	6.96	16.59	2.84	1.00	28.40	80.79
2	8-Sa-0.075	2.66	0.89	5.36	12.65	2.81	0.88	21.67	53.76
3	16-Sa-0.075	2.28	0.74	5.56	9.38	2.76	0.71	12.08	23.78
4	4-Sa-0.2	3.08	1.00	4.46	13.75	2.85	1.00	10.59	30.22
5	8-Sa-0.2	2.84	0.88	4.14	10.36	2.81	0.88	8.13	20.17
6	16-Sa-0.2	2.56	0.72	5.20	9.62	2.78	0.71	4.76	9.43
7	4-Sa-0.4	2.57	1.00	3.66	9.39	2.85	1.00	5.30	15.12
8	8-Sa-0.4	3.48	0.86	3.73	11.16	2.88	0.88	3.95	10.01
9	16-Sa-0.4	3.67	0.68	3.76	9.33	2.59	0.72	4.35	8.13
10	4-Sc-0.075	2.64	1.00	5.60	14.79	2.97	1.00	13.06	38.82
11	8-Sc-0.075	2.81	0.89	4.57	11.39	2.97	0.88	10.53	27.58
12	16-Sc-0.075	2.64	0.73	4.77	9.15	2.93	0.71	8.06	16.85
13	4-Sc-0.2	2.68	1.00	3.66	9.81	2.97	1.00	5.30	15.75
14	8-Sc-0.2	3.11	0.88	3.59	9.83	3.03	0.88	4.33	11.55
15	16-Sc-0.2	3.40	0.69	4.05	9.54	2.67	0.73	5.32	10.31
16	4-Sc-0.04	2.56	1.00	3.02	7.72	2.59	1.00	4.49	11.65
17	8-Sc-0.4	2.91	0.88	3.20	8.23	2.74	0.89	4.07	9.92
18	16-Sc-0.4	3.49	0.69	3.84	9.25	2.79	0.72	4.43	8.90
19	4-Se-0.075	2.12	1.00	3.55	7.51	2.54	1.00	6.51	16.55
20	8-Se-0.075	2.57	0.89	4.08	9.29	2.67	0.88	5.26	12.39
21	16-Se-0.075	2.54	0.72	4.40	8.09	2.70	0.72	4.38	8.46
22	4-Se-0.2	2.46	1.00	3.41	8.39	2.21	1.00	4.25	9.41
23	8-Se-0.2	2.49	0.89	3.03	6.73	2.62	0.88	3.91	9.07
24	16-Se-0.2	3.55	0.68	3.75	9.00	2.39	0.73	4.09	7.18
25	4-Se-0.4	2.52	1.00	3.22	8.10	2.15	1.00	4.17	8.97
26	8-Se-0.4	2.24	0.90	2.66	5.39	2.54	0.89	3.33	7.50
27	16-Se-0.4	3.78	0.67	3.48	8.78	3.23	0.69	4.07	9.07

Table 5. R_m, R_M, R_S, and R factors for WCSB models

No	Notation	Perimeter frames				Distributed frames			
		R_μ	R_M	R_S	R	R_μ	R_M	R_S	R
1	4-Sa-0.075	1.95	0.74	5.06	7.30	1.97	0.74	29.29	42.60
2	8-Sa-0.075	1.97	0.66	15.57	20.45	1.54	0.70	52.51	56.84
3	16-Sa-0.075	1.49	0.64	13.80	13.80	1.34	0.66	41.44	41.44
4	4-Sa-0.2	2.06	0.73	3.73	5.63	1.98	0.74	10.94	15.97
5	8-Sa-0.2	1.89	0.67	5.54	6.97	1.54	0.70	19.67	21.29
6	16-Sa-0.2	1.35	0.66	4.16	4.16	1.33	0.67	15.57	15.57
7	4-Sa-0.4	2.09	0.73	3.51	5.35	1.98	0.74	4.32	6.31
8	8-Sa-0.4	1.77	0.68	3.02	3.62	1.61	0.69	7.64	8.55
9	16-Sa-0.4	1.92	0.59	3.49	3.95	1.58	0.63	6.76	6.76
10	4-Sc-0.075	2.11	0.74	4.03	6.26	2.06	0.74	13.48	20.56
11	8-Sc-0.075	1.83	0.68	7.32	9.09	1.60	0.70	25.46	28.60
12	16-Sc-0.075	1.38	0.66	7.55	7.55	1.35	0.67	27.67	27.67
13	4-Sc-0.2	2.43	0.71	2.98	5.18	2.06	0.74	4.32	6.59
14	8-Sc-0.2	1.76	0.69	3.19	3.86	1.72	0.69	8.19	9.72
15	16-Sc-0.2	1.52	0.64	3.97	3.97	1.38	0.66	10.38	10.38
16	4-Sc-0.04	2.34	0.72	3.00	5.05	2.20	0.73	3.70	5.94
17	8-Sc-0.4	2.11	0.65	2.44	3.37	1.65	0.70	4.91	5.66
18	16-Sc-0.4	1.51	0.65	3.63	3.63	1.64	0.63	5.78	5.97
19	4-Se-0.075	1.94	0.73	3.60	5.13	1.88	0.74	5.31	7.39
20	8-Se-0.075	1.77	0.68	3.83	4.58	1.59	0.70	13.28	14.72
21	16-Se-0.075	1.46	0.65	4.33	4.33	1.37	0.66	13.10	13.10
22	4-Se-0.2	2.05	0.72	3.13	4.64	1.91	0.74	3.97	5.59
23	8-Se-0.2	1.90	0.66	2.52	3.17	1.63	0.69	4.29	4.84
24	16-Se-0.2	1.74	0.61	3.65	3.88	1.62	0.63	6.23	6.33
25	4-Se-0.4	2.12	0.72	3.16	4.80	1.97	0.73	3.75	5.39
26	8-Se-0.4	1.93	0.66	2.34	2.98	1.69	0.69	3.31	3.84
27	16-Se-0.4	1.52	0.64	3.29	3.29	1.43	0.65	5.09	5.09

$$R_\mu = 1+\frac{T}{\phi} = 1+\frac{1.847}{0.741} = 3.49 \tag{11}$$

where the function ϕ is computed by Appendix for site C as follows.

$$\phi = \frac{1}{8+9(\ln\mu)}+\frac{0.83T}{\mu-1} = \frac{1}{8+9(\ln 3.23)}+\frac{0.83\times 1.847}{3.23-1} = 0.741 \tag{12}$$

The modification factor (R_M) to account for the MDOF effects is calculated as follows.

$$R_M = 1.24\times e^{\{-0.1[\ln(\mu)+2]T\}} = 1.24\times e^{\{-0.1[\ln(3.23)+2]\times 1.847\}} = 0.69 \tag{13}$$

From nonlinear static analysis, the strength factor is 3.84 for this example frames, then the response modification factor(R) is evaluated as follows.

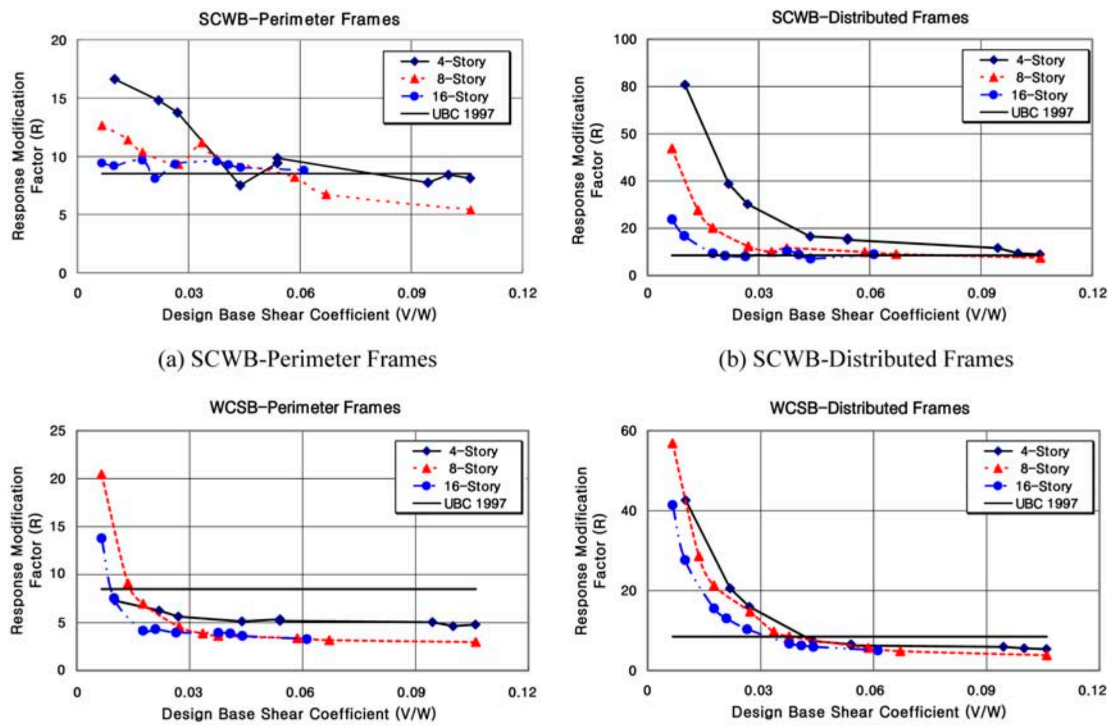
$R = R_\mu \times R_M \times R_s = 3.49\times 0.69\times 3.84 = 9.25$

The ductility for SDOF (R_μ), MDOF modification (R_M), strength (R_S) and response modification factors (R) for all of the prototype frames are provided in Table 4 and 5 for SCWB and WCSB models, respectively. The first number in the notations refers to number of story. The second letter refers to site category, and last number refers to seismic zone factor. The member sizes were controlled by gravity loads where the seismic design intensity were minor region such as seismic zone factor of Z=0.075 or soil profile of S_A. In these regions, the frame had a great value of strength factors (R_S) because the seismic design base shear (V_d) had a relatively small value compared to the actual maximum base shear (V_0). These great values of strength factors (R_S) caused the some extreme values of response modification factors (R), as shown in Table 4 and 5.

5. Results and Discussions

5.1. Effects of seismic design intensity

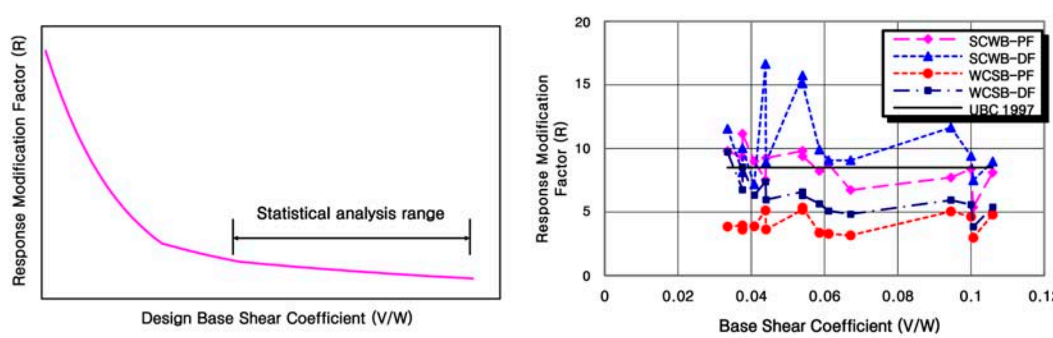
The variations of response modification factors with design base shear coefficient (V/W), which represent the seismic design intensity, are shown in Fig. 4 for SCWB



(a) SCWB-Perimeter Frames (b) SCWB-Distributed Frames

(c) WCSB-Perimeter Frames (d) WCSB-Distributed Frames

Figure 4. Variation of response modification factors with V/W



(a) Statistical analysis ranges of R factors (b) Variations of R factors with V/W

Figure 5. Statistical analysis of R factors with V/W.

and WCSB models, respectively. As shown in these figures, the seismic design intensity has great influence on response modification factors, regardless of SCWB and WCSB models. The following observations are made from Fig. 4.

- In lower seismic design intensity ranges, the more seismic design intensity increases, the more response modification factor decreases dramatically. On the other hand, in higher seismic design intensity ranges, regardless of SCWB and WCSB models, the response modification factors are approximately constant.
- The variations of response modification factors with the changes of seismic design intensity for distributed frames are more significant as compared to perimeter frames, regardless of SCWB and WCSB models.
- The variations of response modification factors with the changes of seismic design intensity for WCSB models are more noticeable as compared to SCWB models, regardless of perimeter and distributed frames.

5.2. Analysis of response modification factor

Statistical analysis was carried out to investigate the response modification factors for prototype frames. Some of the extreme values in lower base shear coefficient (V/W) ranges were excluded from statistical analysis. As shown in Fig. 5(a), the statistical analysis was carried out for the values which are approximately constant in higher base shear coefficient (V/W) ranges. The following conclusions can be made from Table 6 and Fig. 5.

Table 6. Statistical analysis of response modification factors for prototype frames

Soil profile	Zone factor	Story	Design base shear (V/W)	SCWB PF	SCWB DF	WCSB PF	WCSB DF
Sa	Z=0.4	4	0.0540	9.39	15.12	5.35	6.31
		8	0.0376	11.16	10.01	3.62	8.55
		16	0.0540	9.33	8.13	3.95	6.76
Sc	Z=0.2	4	0.0540	9.81	15.75	5.18	6.59
		8	0.0335	9.83	11.55	3.86	9.72
	Z=0.4	4	0.0945	7.72	11.65	5.05	5.94
		8	0.0586	8.23	9.92	3.37	5.66
		16	0.0440	9.25	8.90	3.63	5.97
Se	Z=0.075	4	0.0439	7.51	16.65	5.13	7.39
	Z=0.2	4	0.1000	8.39	9.41	4.64	5.59
		8	0.0671	6.73	9.07	3.17	4.84
		16	0.0408	9.00	7.18	3.88	6.33
	Z=0.4	4	0.1059	8.10	8.97	4.80	5.39
		8	0.1006	5.39	7.50	2.98	3.84
		16	0.0611	8.78	9.07	3.29	5.09
Mean value of *R* factor				8.57	10.59	4.13	6.26
Standard deviation				1.40	2.99	0.82	1.46
Coefficient of variation				0.16	0.28	0.20	0.23

- The values of the response modification factors for SCWB perimeter frames are ranged from 5.39 to 11.16, which are 63.4% to 131.3% of the assigned *R* value 8.5, in UBC 1997. For SCWB distributed frames, the values of the response modification factors are ranged from 7.50 to 16.65, and these values are 88.23% to 195.9% of the assigned value, that is 8.5.
- For WCSB perimeter frames, the values of the response modification factors are ranged from 2.98 to 5.35, which are 35.1% to 62.9% of the assigned R value 8.5. For WCSB distributed frames, the values of the response modification factors are ranged from 3.85 to 9.72, and these values are 45.2% to 114.3% of the assigned value.
- The coefficients of variation of the response modification factors for prototype structures are evaluated as 0.16, 0.28, 0.20, and 0.23, for SCWB perimeter frames, SCWB distributed frames, WCSB perimeter frames and WCSB distributed frames, respectively.

6. Conclusions

This research suggested simplified rules of an evaluation methodology for the response modification factors (R) for steel moment-resisting frames. The proposed rules can be applied to practical application by nonlinear static pushover analysis instead of nonlinear dynamic analysis. The proposed rules were applied to existing 108 steel moment resisting frames to verify the proposed rules. The results of this study can be summarized as follows:

- The variation of response modification factors with the increasing the seismic design intensity was decreased dramatically in lower seismic design intensity ranges. However, the response modification factors approached the constant values in higher seismic design intensity ranges.
- For SCWB models, the mean value of response modification factors, in higher seismic design intensity ranges, are evaluated as 8.57 and 10.59, for perimeter and distributed frames, respectively. These values are 100.8% and 124.6% of the recommended value of 8.5.
- For WCSB models, the mean value of response modification factors are evaluated as 4.13 and 6.26, for perimeter and distributed frames, respectively. These values are 48.6% and 73.6% of the recommended value of 8.5.

It was confirmed that the proposed rules can be used to evaluate response modification factors of existing or new structures. Based on the proposed rules, it is necessary to reassess the response modification factors to guarantee the desired performance or uniform level of safety for all seismic framing systems.

Notations

A_g: Gross area of a column
P_{uc}: Required axial strength in the column (in compression)≥0
F_{yb}: Specified minimum yield strength of a beam
F_{yc}: Specified minimum yield strength of a column

Z_b: Plastic section modulus of a beam
Z_c: Plastic section modulus of a column
V_n: Nominal strength of the panel zone
d_b: Average overall depth of the beams framed into the connection
H: Average of the story heights above and below the joint
P_x: The total unfactored vertical design load at and above Level x
Δ: The design story drift occurring simultaneously with V_x
V_x: The seismic shear force acting between Level x and x-1
h_{sx}: The story height below Level x
C_d: The deflection amplification factor
β: The ratio of shear demand to shear capacity for the story between Level x and x-1

Appendix

1. The functions ϕ

- For site AB, $\phi = \dfrac{1}{4+16(\ln\mu)} + \dfrac{0.92T}{\mu-1}$
- For site C, $\phi = \dfrac{1}{8+9(\ln\mu)} + \dfrac{0.83T}{\mu-1}$
- For site D, $\phi = \dfrac{1}{3+7(\ln\mu)} + \dfrac{0.79T}{\mu-1}$
- For site E, $\phi = \dfrac{1}{2+4(\ln\mu)} + \dfrac{0.82T}{\mu-1}$

2. MDOF modification factor (R_M)

- SCWB Models

(1) For $T \le 0.075$ sec, $R_M = 1$
(2) For $T > 0.075$ sec, $R_M = 1.24 \times e^{\{-0.1[\ln(\mu)+2]T\}}$

- WCSB Models

(1) For $T \le 0.2$ sec, $R_M = 1$
(2) For $T > 0.2$ sec, $R_M = \dfrac{0.8\mu^{-0.25}}{T^{0.15[\ln(\mu)+1]}}$

References

AISC (2002). *Seismic provisions for structural steel buildings*. American Institute of Steel Construction, Chicago, Illinois.

ASCE/SEI 7-05 (2007). *Minimum Design Loads for Buildings and Other Structures*. American Society of Civil Engineers, USA.

Asgarian, B. and Shokrgozar, H. R. (2009). "BRBF response modification factor." *Journal of Constructional Steel Research*, 65(2), pp. 290-298.

ATC (1978). Tentative Provisions for the Development of Seismic Regulations for buildings. *ATC Report 3-06*, Applied Technology Council, Redwood City, California.

ATC-19 (1995). Structural Response Modification Factors. *ATC Report 19*, Applied Technology Council, Redwood City, California.

ATC-34 (1995). A Critical Review of Current Approaches to Earthquake-resistant Design. *ATC Report 34*, Applied Technology Council, Redwood City, California.

Balendra T. and Huang, X. (2003). "Overstrength and ductility factors for steel frames designed according to BS 5950." *Journal of Structural Engineering*, ASCE, 120, pp. 1019-1035.

FEMA 302 (1997). *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*. Building Seismic Safety Council, Washington, DC.

FEMA 274 (1997). *NEHRP Commentary on the Guidelines for the Rehabilitation of Buildings*. Building Seismic Safety Council, Washington, DC.

Freeman, S. A. (1990). "On the correlation of code forces to earthquake demands." *Proc. 4th U.S.-Japan Workshop on Improvement of Building Structural Design and Construction Practices*, ATC-15-3 Report, Applied Technology Council, Redwood City, California.

Galíndez, N. and Thomson, P. (2007). "Performance of steel moment-frame buildings designed according to the Colombian code NSR-98." *Engineering Structures*, 29(9), pp. 2274-2281.

Hwang, H. and Shinozuka, M. (1994). "Effects of large earthquake on the design of buildings in eastern united states." *Proc. Fifth U.S. National Conference on Earthquake Engineering*, Chicago, Illinois.

ICBO (1997). *Uniform Building Code*. International Conference of Building Officials, Whittier, California.

ICC (2000). *International Building Code 2000*. International Code Council, Falls Church, Virginia, USA.

Kang, C. K. (2003). Evaluation of Response Modification Factors for Special Steel Moment Resisting Frames. *Ph. D. Dissertation*, Kyonggi University, Seoul, Korea (in Korean).

Kang, C. K. and Choi, B. J. (2002). "Empirical estimation of ductility factors for elasto-plastic SDOF systems in alluvium sites." *Proc. 2nd International Symposium on Steel Structures*, Korean Society of Steel Construction, Korea, pp. 533-542.

Kang, C. K. and Choi, B. J. (2004). "Statistical study and evaluation of ductility factors for elastic perfectly plastic SDOF systems in stiff soils." *International Journal of Steel Structures*, 4(1), pp. 15-24.

Kang, C. K. and Choi, B. J. (2010). "Empirical evaluation of ductility factors for the special steel moment-resisting frames in view of soil condition." *The Structural Design of Tall and Special Buildings*, 19(5). pp. 551-572.

Kim, J. K. and Choi, H. H. (2005). "Response modification factors of chevron-braced frames." *Engineering Structures*, 27, pp. 285-300.

Kurban, C. O. and Topkaya, C. (2009). "A numerical study on response modification, overstrength, and displacement amplification factors for steel plate shear wall systems." *Earthquake Engineering and Structural Dynamics*, 38, pp. 497-516.

Maheri, M. R. and Akbari, R. (2003). "Seismic behavior factor, R, for steel X-braced and knee-braced RC buildings." *Engineering Structures*, 25, pp. 1505-1513.

Mahmoudi, M. and Zaree, M. (2010). "Evaluating response modification factors of concentrically braced steel frames." *Journal of Constructional Steel Research*, 66(10), pp. 1196-1204.

MIDAS/Gen program (2001). *MIDAS/Gen-General structural design system.* Analysis and design manual, MIDAS Information Technology, Seoul, Korea.

Miranda, E. (1997). "Strength Reduction Factors in Performance-based Design." *Proc. EERC-CURE Symposium in Honor of V. V. Bertero*, Report No. UCB/EERC 97/05, University of California, Berkeley, California, pp. 125-132.

Nassar, A. A. and Krawinkler, H. (1991). Seismic Demands of SDOF and MDOF Systems. *Report No. 95*, John A. Blume Earthquake Engineering Center, Stanford University, California.

Osteraas, J. D. (1990). Strength and Ductility Considerations in Seismic Design. *Ph.D. Dissertation*, Stanford University.

Sudhir, K. J. and Navin, R. (1995). "Seismic Overstrength in Reinforced Concrete Frames." *Journal of Structural Engineering*, ASCE, 121(3), pp. 580-585.

Tsai, K. C. and Li, J. W. (1994). DRAIN 2D +: A General-purpose Computer Program for Static and Dynamic Analyses of Inelastic 2D Structures Supplemented with a Graphic Processor. *Report No. CEER/R83 03*, National Taiwan University.

Uang, C. M. and Maarouf, A. (1993). "Safety and Economy Considerations of UBC Seismic Force Reduction Factors." *Proc. the 1993 National Earthquake Conference*, Memphis, Tennessee.

Uang, C. M. and Bertero, V. V. (1986). Earthquake Simulation Tests and Associated Studies of a 0.3-Scale Model of Six-story Concentrically Braced Steel Structures. Earthquake Engineering Research Center, *Report No. UCB/EERC 86/10*, University of California, Berkeley, California.

Uang, C. M. (1991). "Establishing R(or Rw) and Cd factor for Building Seismic provisions." *Journal of Structural Engineering,* ASCE, 117(1), pp. 19-28.

Whittaker, A. S., Uang, C. M., and Bertero, V. V. (1987). Earthquake Simulation Tests and Associated Studies of a 0.3-Scale Model of Six-story Eccentrically Braced Steel Structures. Earthquake Engineering Research Center, *Report No. UCB/EERC 87/02*, University of California, Berkeley, California.