STEEL STRUCTURES
www.kssc.or.kr

Steel Structures 7 (2007) 253-261

Elastic Seismic Design of Steel High-rise Buildings in Regions of Strong Wind and Moderate Seismicity

Cheol-Ho Lee* and Seon-Woong Kim

Department of Architecture, Seoul National University, San 56-1, Shinlim-Dong, Kwanak-Gu, Seoul 151-742, South Korea

Abstract

Lateral loading due to wind or earthquake is a major factor that affects the design of high-rise buildings. This paper highlights the problems associated with the seismic design of high-rise buildings in regions of strong wind and moderate seismicity. Seismic response analysis and performance evaluation were conducted for wind-designed concentrically braced steel high-rise buildings in order to check the feasibility of designing them per elastic seismic design criterion (or strength and stiffness solution) in such regions. Review of wind design and pushover analysis results indicated that wind-designed high-rise buildings possess significantly increased elastic seismic capacity due to the overstrength resulting from the wind serviceability criterion. The strength demand-to-capacity study showed that, due to the wind design overstrength, high-rise buildings with a slenderness ratio of larger than four or five can elastically withstand even the maximum considered earthquake (MCE) with the seismic performance level of immediate occupancy under the limited conditions of this study. A step-by-step seismic design procedure per the elastic criterion that is directly usable for practicing design engineers is also recommended.

Keywords: concentrically braced steel frames, high-rise buildings, moderate seismicity, seismic, wind

1. Introduction

Some parts of the world belong to the region of strong wind and moderate seismicity. For example, damaging typhoons frequently strike the Korean peninsular, but the regional seismicity of Korea is moderate to low such that the level of design peak ground acceleration is about 0.15 g in average return period of 476 years. It is well-known that the seismic spectral acceleration for typical high-rise buildings is drastically reduced due to their long fundamental period; it normally falls into the displacement-sensitive region of the seismic response spectrum. As a result, the apparent magnitude of wind base shear far exceeds that of design seismic base shear (or the elastic base shear demand divided by the seismic response modification factor). When faced with this rather unique lateral loading situation, it appears that different procedures are used among different engineers. Some conduct wind design only and omit seismic design, thinking that seismic requirements will automatically be satisfied since wind loading is apparently much larger. Others sometimes try to conduct costly and time-consuming inelastic dynamic analysis, often by using input motions not rationally scaled, to evaluate the seismic performance of wind-designed structures. But these approaches have some undesirable aspects. First, the structural system of high-rise buildings should be viewed as a special system (or undefined system) whose design is not well covered by current seismic codes; the seismic response modification factor of high-rise buildings assumed by the engineer when comparing the design seismic force with wind loading has no code-basis and is difficult to justify. Second, considering the critical importance and monumental nature of high-rise buildings, a designer of a tall building of today, even in moderate to low seismic regions, should evaluate the probable seismic impact on the selected structural system and be able to rationally present the results to the client or the public. Third, effort-demanding inelastic dynamic analysis for high-rise building design may not be needed at all in regions of strong wind and moderate seismicity as will be discussed in this paper.

On the other hand, concentrically braced frames (CBFs) that resist the lateral load through axial load paths are among the most cost-effective systems in providing the stiffness and strength requirements for wind design. However, CBFs have not been considered as the best choice for resisting earthquake load in the inelastic range due to limited energy dissipation capacity, low redundancy, and the propensity to soft story response (Tremblay. 2002). Thus it is desirable, if economically acceptable, to limit the behavior of steel high-rise CBFs in the elastic range even under strong ground motion excitation. It was speculated that the seismic design of steel high-rise CBFs per elastic criterion (or strength and stiffness solution) is economically feasible in regions of strong wind and moderate seismicity because of the system overstrength

*Corresponding author
Tel: +82-2-880-8735, Fax: +82-2-871-5518
E-mail: ceholee@snu.ac.kr

Table 1. Basic data for wind load calculation

Parameter	Value	Remarks
Basic wind speed	30 (m/sec)	Seoul (exposure B)
Topographic factor	1.0	No wind speed-up effect
Importance factor	1.1	Category (special)
Damping ratio	0.02	2% of critical damping

*Dead load: 5.5 kN/m^2
**Live load: 2.5 kN/m^2 (office building)

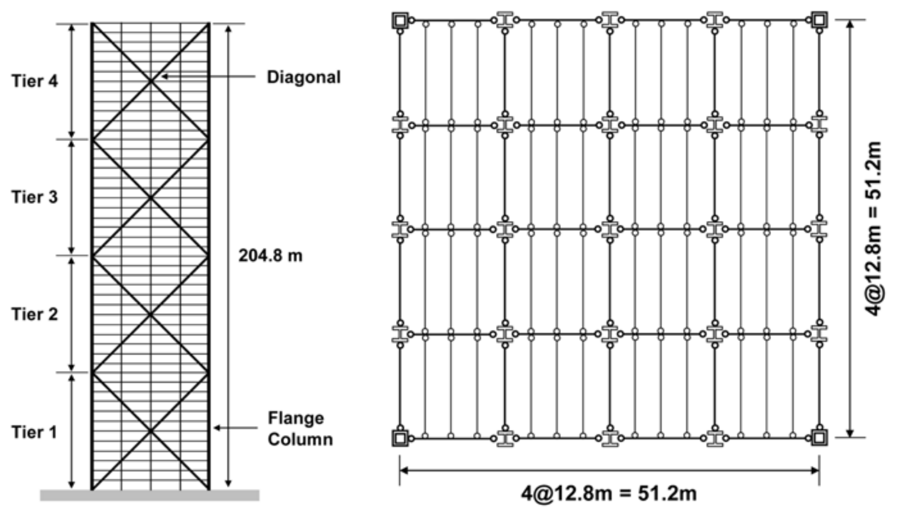
induced in meeting the wind serviceability criterion. The primary objectives of this paper were to study the feasibility of designing steel highrise buildings per elastic design criterion under moderate seismicity and to propose the elastic seismic design procedure that are directly usable for practicing design engineers.

2. Wind Design and System Overstrength

A hypothetical high-rise office building for a case study was assumed to be located in Seoul, Korea. Wind load was calculated based on the data summarized in Table 1 and by following the Korean Building Code (KBC 2005) (AIK, 2005). Multi-bay and multi-story mega CBF system with 12 stories as a tier was selected to meet the stiffness and strength requirements for wind design (see Fig. 1). The selected structural system is similar to that used by Englekirk (Englekirk, 1994; 1996). To study the effects of the tallness of high-rise buildings on seismic response, four case study models with the building slenderness (*H/D*) of 4 (48 story-high model with a building height of 205 m) to 7 (84 story-high model with a building height of 359 m) were designed. In the above definition of the building slenderness (*H/D*), *H* is the building height and *D* is the width of the building along the direction of the lateral force. In this system, lateral load is entirely resisted by the vertical mega truss consisting of the flange columns and the diagonals; that is, the flange columns and the diagonals resist the story overturning moment and the story shear, respectively. All the flange columns and the diagonals were designed against the story overturning moment and the story shear force, respectively, using welded built-up box shapes (SM490 steel material, F_y = 335 MPa) by following the AISC-LRFD (AISC, 2001) code. Wide flange shapes were used for all the gravity beams and columns. Every beam to column connection was assumed as a simple shear connection to keep the fabrication cost to a minimum.

After satisfying the wind strength demand, in order to meet wind serviceability requirements, both roof and story drift ratios were limited to 1/500 by controlling the shear and flexural mode deformations of the diagonals and the flange columns (see Tables 2 and 3). Together with these drift limits, nesting the inside dimensions of the tubular sections for easy fabrication and erection further increased member sizes [see the last columns of Table 2(a) and 2(b)]. This wind-design overstrength leads to significant increase in the elastic seismic capacity of the system, as will be shown in the following.

The speed of the wind storm with a return period of 10 years was computed to be 21.28 (m/sec) at the hypothetical building site. Top story acceleration induced by the wind storm with a return period of 10 years was calculated by following the detailed procedure of National Building Code of Canada (NBCC, 1995). Refer to Eqs. (1) and (2) for along- and across-wind acceleration, respectively. The symbols used in this paper are summarized in the appendix. Table 4 shows that all four models with a building density of 128.9 kg/m^3 satisfy the wind-induced acceleration limit for human comfort (30 gal for office usage) by a sufficient margin.



(a) Structural elevation (slenderness 4) (b) Structural plan

Figure 1. Structural plan and elevation.

Table 2. Selected member sizes

(a) Flange columns

H/D	Tier	Area required from strength design (cm^2)	Shapes	Wall thk. (mm)	Area provided for service-ability (cm^2)	Sectional area increase (%)
4	1	2,753.2	□-1450×1450	79.7	4,367.0	58.6
	2	2,219.8	□-1430×1430	69.8	3,795.4	71.0
	3	1,730.9	□-1400×1400	54.7	2,942.9	70.0
	4	909.5	□-1370×1370	39.7	2,113.0	132.3
5	1	4,345.6	□-2000×2000	114.3	8,620.4	98.4
	2	3,648.5	□-1970×1970	99.5	7,444.2	104.0
	3	2,998.1	□-1940×1940	84.7	6,286.9	109.7
	4	1,859.6	□-1910×1910	69.7	5,131.3	175.9
	5	964.5	□-1880×1880	54.7	3,990.3	313.7
6	1	6,383.8	□-2650×2650	155.0	15,466.3	142.3
	2	5,510.2	□-2630×2630	145.3	14,441.4	162.1
	3	4,684.9	□-2600×2600	130.7	12,905.1	175.5
	4	3,202.3	□-2570×2570	115.8	11,364.7	254.9
	5	1,969.9	□-2540×2540	100.8	9,834.3	399.2
	6	1,012.3	□-2500×2500	80.6	7,804.4	671.0
7	1	8,899.3	□-3200×3200	191.6	23,058.3	159.1
	2	7,837.4	□-3170×3170	176.1	21,090.3	169.1
	3	6,825.7	□-3140×3140	161.0	19,187.7	181.1
	4	4,975.0	□-3110×3110	146.0	17,310.8	248.0
	5	3,380.8	□-3080×3080	131.1	15,460.0	357.3
	6	2,067.4	□-3050×3050	116.0	13,610.3	558.3
	7	1,054.9	□-3020×3020	101.0	11,793.1	1,017.9

(b) Diagonals

H/D	Tier	Area required from strength design (cm^2)	Shapes	Wall thk. (mm)	Area provided for service-ability (cm^2)	Sectional area increase (%)
4	1	655.1	□-840×840	38.2	1,224.6	86.9
	2	606.3	□-820×820	37.3	1,167.0	92.5
	3	539.4	□-800×800	36.4	1,110.7	105.9
	4	384.2	□-780×780	35.5	1,055.9	174.8
5	1	887.7	□-1110×1110	50.5	2,138.4	140.9
	2	836.4	□-1080×1080	49.1	2,024.3	142.0
	3	766.6	□-1060×1060	48.2	1,950.0	154.4
	4	606.0	□-1040×1040	47.3	1,877.2	209.8
	5	425.1	□-1020×1020	46.4	1,805.7	324.8
6	1	1,138.6	□-1350×1350	61.4	3,163.0	177.8
	2	1,085.1	□-1330×1330	60.5	3,070.0	182.9
	3	1,012.7	□-1310×1310	59.5	2,978.4	194.1
	4	847.1	□-1290×1290	58.6	2,888.1	240.9
	5	661.5	□-1270×1270	57.7	2,799.2	323.2
	6	460.2	□-1250×1250	56.8	2,711.8	489.3
7	1	1,405.7	□-1700×1700	77.3	5,015.7	256.8
	2	1,350.1	□-1670×1670	75.9	4,840.2	258.5
	3	1,275.4	□-1640×1640	74.6	4,667.9	266.0
	4	1,105.3	□-1610×1610	73.2	4,498.7	307.0
	5	915.4	□-1580×1580	71.8	4,332.6	373.3
	6	709.9	□-1550×1550	70.5	4,169.6	487.4
	7	491.3	□-1520×1520	69.1	4,009.8	716.2

(c) Gravity members

Member	H/D	Shapes
Girders	4 - 7	W30×124
Beams		W16×45
Gravity columns	4	W14×176
	5	W14×233
	6	W14×283
	7	W14×342

Table 3. Roof drift check

Slenderness	Wind-induced lateral roof deflection
4	1/501 (= 40.89 cm) ≤1/500 (= 40.96 cm) (OK)
5	1/501 (= 51.09 cm) ≤1/500 (= 51.20 cm) (OK)
6	1/502 (= 61.25 cm) ≤1/500 (= 61.44 cm) (OK)
7	1/501 (= 71.53 cm) ≤1/500 (= 71.68 cm) (OK)

Along-wind acceleration:

$$a_D = 4\pi^2 n_D^2 g_p \sqrt{\frac{KsF}{C_e\beta_D}\frac{\Delta}{C_g}} \quad (1)$$

Table 4. Wind-induced roof acceleration check

Slenderness	Across-wind direction (gal)	Along-wind direction (gal)	Limit (gal)
4	8.52	7.0	30 (OK)
5	11.01	7.65	30 (OK)
6	13.54	8.2	30 (OK)
7	20.53	10.66	30 (OK)

Across-wind acceleration:

$$a_W = n_W^2 g_p \sqrt{WD}\left(\frac{a_r}{\rho_B g \sqrt{\beta_W}}\right) \quad (2)$$

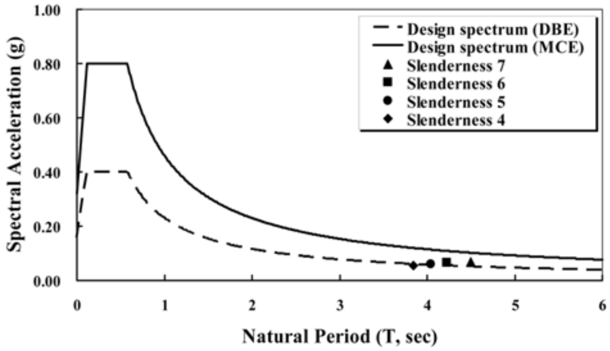


Figure 2. Comparison of design wind base shears and code-elastic spectrum.

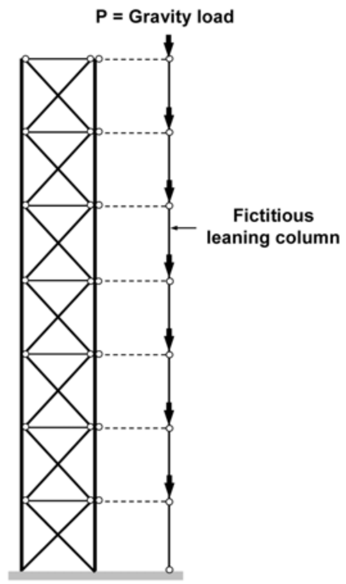


Figure 3. Modeling of P-Δ effect.

Figure 2 shows a comparison of the wind base shear (or factored wind load divided by total building mass) among the four models and the elastic spectral acceleration of the KBC 2005 (seismic zone I and stiff soil site S_D assumed) (AIK, 2005). In the figure, the DBE (Design Basis Earthquake) and the MCE (Maximum Considered Earthquake) designate earthquakes with a return period of 500 years and 2400 years, respectively. The KBC 2005 spectrum (AIK, 2005) is essentially a Newmark spectrum. In plotting the wind base shears, the fundamental periods obtained from SAP 2000 (CSI, 2000) eigenvalue analysis were used. Although the overstrength factor from the wind design was not included in the comparison, all the wind base shears exceed the elastic spectral demand of the DBE. The comparison in Fig. 2 also implies that seismic design strategy per elastic criterion becomes more feasible as the building slenderness increases.

Pushover analysis, with assuming the triangular (or 1st mode) lateral loading pattern, was conducted to evaluate the system overstrength factors of the four wind-designed models by using DRAIN-2DX (Prakash *et al.*, 1993). Element

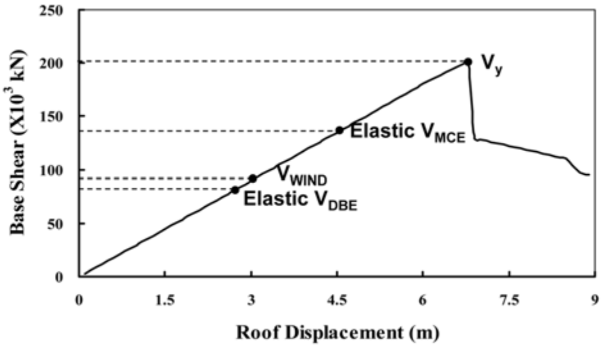


Figure 4. Pushover analysis curve (slenderness 7).

Table 5. System overstrength factors

Slenderness	4	5	6	7
V_y/V_{WIND}	1.22	1.66	1.86	2.34
V_y/Elastic V_{DBE}	1.17	1.73	2.08	2.89
V_y/Elastic V_{MCE}	0.58	0.86	1.15	1.45

*Notes: V_y = yield base shear; V_{wind} = factored wind base shear; Elastic V_{DBE} = elastic base shear demand of DBE; Elastic V_{MCE} = elastic base shear demand of MCE

#9 in DRAIN-2DX and Jain's model for brace buckling (Jain and Goel, 1978) were used to analyze the vertical mega truss system up to the post-buckling range. The second order (P-Δ) effect was also considered in the analysis by imposing the gravity loading on a fictitious leaning column (see Fig. 3). The lateral degrees of freedom in the fictitious leaning column were slaved to the master degree of freedom of the mega bracing system. And P-Δ analysis option in DRAIN-2DX was activated. The pushover analysis curve obtained from the model with slenderness 7 is presented in Fig. 4. In Fig. 4, the yield base shear force level (V_y) corresponds to the yielding of the column in the 1st story. It is worthwhile to note that the overstrength resulting from wind design is incorporated in the pushover analysis. It is observed from Fig. 4 that the model with slenderness 7 possesses significantly increased elastic seismic capacity as high as 1.5 times the elastic spectral demand of the MCE. Table 5 clearly indicates that quality-wind designed high-rise buildings with a slenderness ratio of larger than five can withstand elastically even the MCE and corroborates the speculation that designing steel high-rise buildings per elastic design criterion (strength and stiffness solution) under moderate seismicity is highly feasible. This feasibility comes from the combined effects of the unique loading condition (strong wind, but low seismicity) and structural characteristics of high-rise buildings (very long fundamental period).

3. Seismic Performance Evaluation

In this section, the results of seismic response analysis and performance evaluation conducted for wind-designed concentrically braced steel high-rise buildings are presented to show the feasibility of the elastic seismic design in moderate seismic regions. Before presenting the results, the concept of strength demand -to-strength capacity ratio (DCR) and the seismic performance criteria of FEMA 273 (FEMA, 1997) are briefly described.

In this study, the DCR defined in Eq. (3) is proposed as a convenient index to check whether or not wind-designed structures can resist particular ground input motion elastically. The strength demand is taken as the SRSS (square root of sum of squares) value in the response spectrum analysis and the maximum value in the linear dynamic time history analysis, respectively. The strength capacity is calculated by following AISC-LRFD (AISC, 2001) strength equations for the compression member (for the flange columns) and the flexural-compression member (for the diagonals) for the strength reduction factor of 1.0. Of course, members that satisfy Eq. (3) will remain elastic. Theoretically, if any one member in a structure does not satisfy Eq. (3), the DCR analysis based on linear analysis loses its physical meaning because this analysis does not consider the redistribution of forces in the inelastic range. However, the DCR distribution among the structure is still useful because it can convey an overall picture of the degree of expected inelastic behavior to the analyst.

$$\frac{\text{Strength Demand}}{\text{Strength Capacity}} \leq 1 \tag{3}$$

The basic safety objective (BSO) of ordinary buildings as recommended by FEMA 273 [10] is to achieve the LS (Life Safety) and the CP (Collapse Prevention) seismic performance level for DBE and MCE, respectively. The FEMA 273 acceptance criteria for the seismic performance of braced steel frames for IO (Immediate Occupancy), LS, and CP are based on the structural response levels of 0.5, 1.5 and 2% story drift, respectively.

3.1. Results based on response spectrum analysis

Figure 5 presents the critical DCR values obtained from the response spectrum analysis for the DBE (PGA = 0.16 g) and the MCE (PGA = 0.32 g). The results were obtained from the SRSS modal combination; the CQC (complete quadratic combination) of modal responses showed little difference. All four models can resist the DBE elastically by a sufficient margin [see Fig. 5(a)]. For the MCE, the

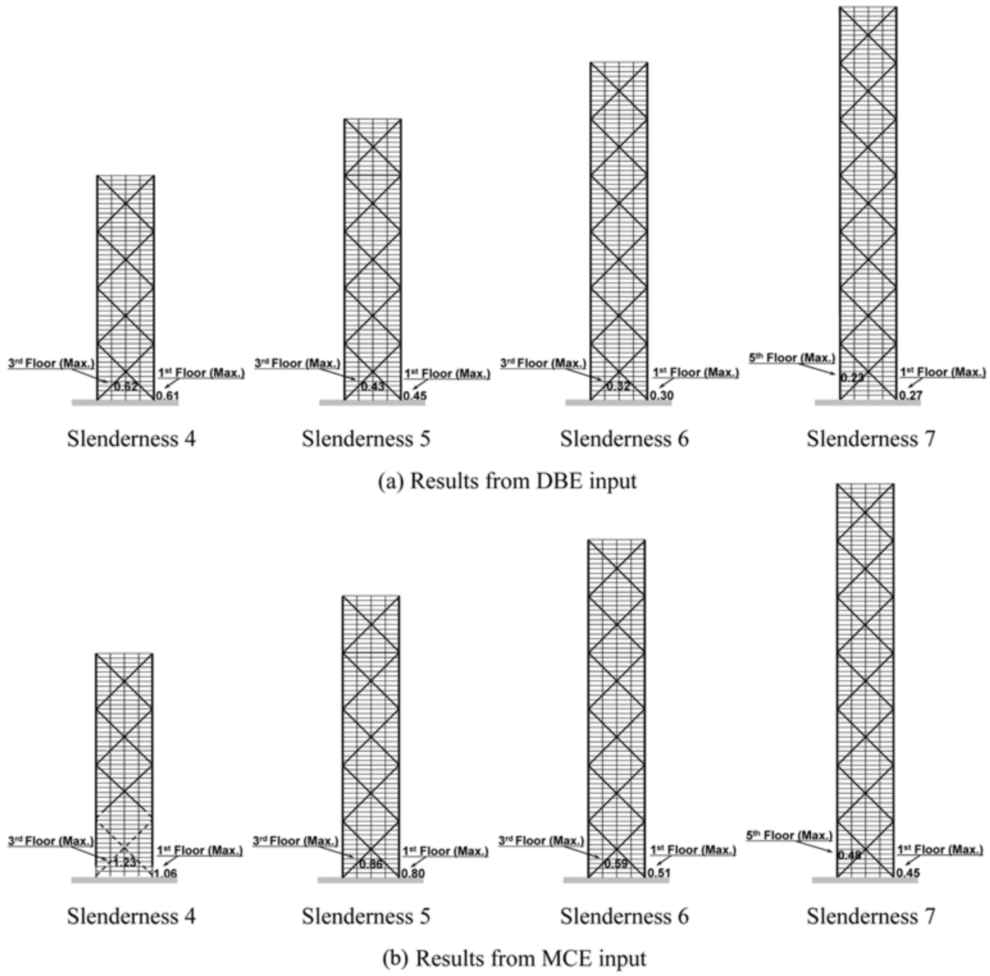


Figure 5. Maximum values of DCR along the building height from response spectrum analysis.

Table 6. Seismic performance evaluation results based on response spectrum analysis

Building slenderness	DBE		MCE	
	Maximum story drift	Seismic performance level	Maximum story drift	Seismic performance level
4	0.20%	IO	N.A.	-
5	0.17%	IO	0.34%	IO
6	0.15%	IO	0.28%	IO
7	0.14%	IO	0.27%	IO

Table 7. Input ground motions used for linear dynamic time history analysis

Earthquakes	Station	Component	PGA (g)	PGV (cm/sec)	PGD (cm)	PGV/PGA (sec)	PGD/PGV (sec)
Imperial Valley (1979)	5060 Brawley Airport	225	0.16	35.9	22.44	0.229	0.625
Kern County (1952)	1095 Taft Lincoln School	021	0.156	15.3	9.25	0.100	0.605
Kern County (1952)	1095 Taft Lincoln School	111	0.178	17.5	8.99	0.100	0.514
Lander s(1992)	12149 Desert Hot Springs	090	0.154	20.9	7.78	0.138	0.372
Landers (1992)	22074 Yermo Fire Station	360	0.152	29.7	24.69	0.199	0.831
Loma Prieta (1989)	57066 Agnews State Hospital	000	0.172	26	12.64	0.154	0.486
Loma Prieta (1989)	57504 Coyote Lake Dam (Downst)	285	0.179	22.6	13.2	0.129	0.584
Taiwan SMART1(45) (1986)	68 SMART1 O12	NS	0.159	23.3	10.61	0.150	0.455
Victoria, Mexico (1980)	6621 Chihuahua	102	0.15	24.8	9.2	0.169	0.371
Westmorland (1981)	5051 Parachute Test Site	315	0.155	26.6	12.97	0.175	0.488
Duzce, Turkey (1999)	Duzce	180	0.348	60	42.09	0.176	0.702
Imperial Valley (1979)	6605 Delta	352	0.351	33	19.02	0.096	0.576
Imperial Valley(1979)	955 El Centro Array #4	230	0.36	76.6	59.02	0.217	0.770
Imperial Valley (1979)	5028 El Centro Array #7	140	0.338	47.6	24.68	0.144	0.518
Imperial Valley (1979)	5165 El Centro Differential Array	270	0.352	71.2	45.8	0.206	0.643
Imperial Valley (1979)	5155 EC Meloland Overpass FF	000	0.314	71.7	25.53	0.233	0.356
Kocaeli, Turkey (1999)	Duzce	180	0.312	58.8	44.11	0.192	0.750
Kocaeli, Turkey (1999)	Yarimca	330	0.349	62.1	50.97	0.182	0.821
Loma Prieta (1989)	58065 Saratoga-Aloha Ave	090	0.324	42.6	27.53	0.134	0.646
Loma Prieta (1989)	58235 Saratoga-W Valley Coll.	270	0.332	61.5	36.4	0.189	0.592

model with slenderness 4 shows critical DCR values slightly larger than one in the lower part of the structure, but other models with a slenderness ratio of larger than five can withstand even the MCE without yielding, as was expected in the pushover analysis of the preceding section.

When following FEMA 273 seismic performance acceptance criteria mentioned above, wind-designed steel CBFs of this study with a slenderness ratio of larger than four or five can withstand elastically even the MCE with the seismic performance level of immediate occupancy (see Table 6).

3.2. Results based on time history analysis

After selecting twenty ground motions recorded at stiff soil sites (or the ground motions compatible with site soil condition of the seismic response spectrum) (http://peer.berkeley.edu/sm), linear dynamic time history analysis was conducted, and the results were compared with those of the response spectrum analysis presented above. See Table 7 for the input details. Input ground motions in Table 7 were scaled based on the EPA (Effective Peak Acceleration) by following the ATC procedure (ATC, 1978); EPA = 0.16 g for the DBE and 0.32 g for the MCE, respectively. All the time history analysis results presented in the following are the average values of twenty responses.

Figure 6 presents the critical DCR values obtained from linear dynamic time history analysis for the DBE and the MCE. In contrast to the results of the response spectrum analysis, all the four models including the case of slenderness 4 can resist even the MCE elastically. Time history analysis results in Table 8 indicate that all the models including the case of slenderness 4 can withstand elastically even the MCE with the seismic performance level of immediate occupancy.

Overall, the strength demand, story drift, and seismic performance predictions based on the response spectrum analysis are comparable to those based on linear time history analysis. Considering the very high uncertainties in predicting the details of future earthquakes at a particular site, it is recommended that much simpler but reasonably accurate response spectrum analysis be used in evaluating the seismic performance of high-rise steel CBFs.

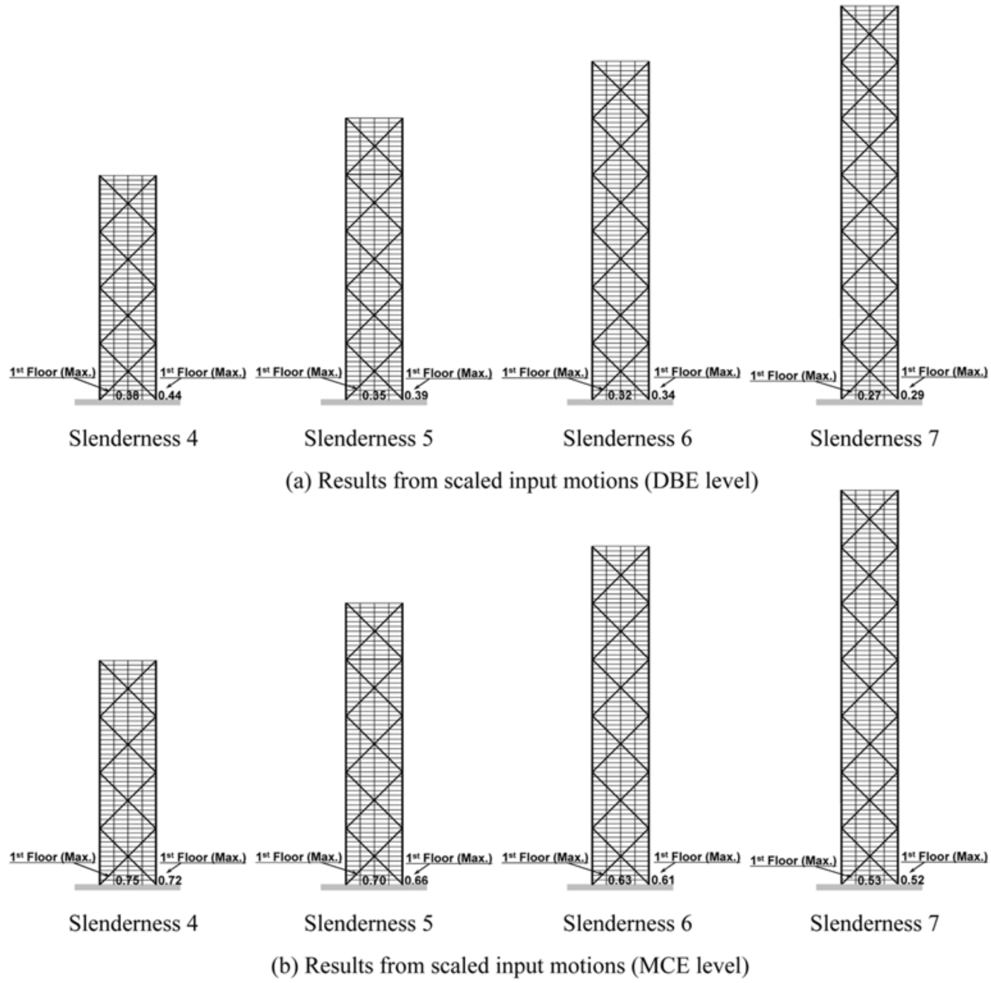


Figure 6. Maximum values of DCR along the building height from linear time history analysis.

Table 8. Seismic performance evaluation results based on time history analysis

Slenderness	DBE		MCE	
	Maximum story drift	Seismic performance level	Maximum story drift	Seismic performance level
4	0.19%	IO	0.38%	IO
5	0.20%	IO	0.40%	IO
6	0.21%	IO	0.42%	IO
7	0.21%	IO	0.43%	IO

Of course, the above results were obtained by analyzing a limited set of steel building and considering only one soil type (stiff soil), and should be considered as optimistic. The results obtained in this paper will be generalized by considering different soil conditions and different values of the width of the facade through the continuing work.

4. Elastic Seismic Design Procedure Proposed

Based on the discussions above, a step-by-step seismic design procedure per elastic criterion that is directly usable for practicing design engineers in regions of strong wind and moderate seismicity is proposed as follows (refer to Fig. 7):

Step 1. Selection of structural system: axial load path systems are recommended for the most cost-effective stiffness and strength solution.

Step 2. Set performance objectives for wind and earthquake loading: considering the results of this study, high seismic performance objectives are recommended.

Step 3. Perform quality-wind design that meets all serviceability criteria.

Step 4. Check seismic code requirements, if any.

Step 5. Perform response spectrum analysis for the DBE or the MCE, and check the DCR. Two branches are possible:

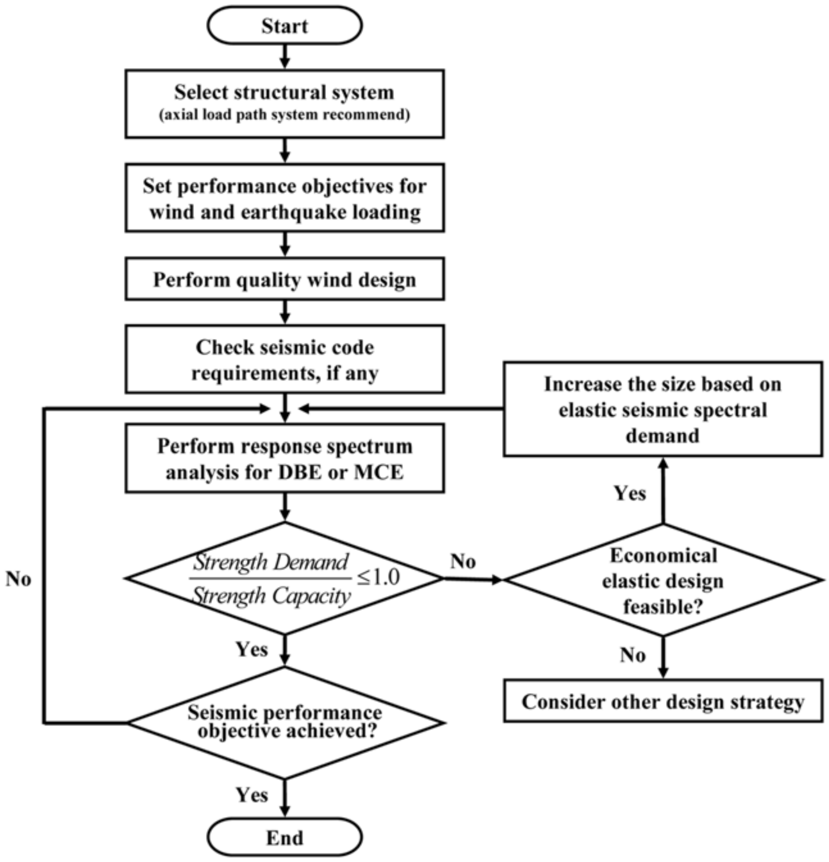


Figure 7. Recommended elastic seismic design procedure.

(i) If all members have the DCR values less than one, check whether or not the seismic performance objectives are satisfied; if not, iteration is needed.

(ii) If some members have the DCR values larger than one, increase the member size based on the elastic seismic spectral demand and iterate until the objectives are satisfied.

Of course, iteration should not accompany too costly material increase. If the elastic design is not economically feasible, other design strategies such as the limited ductility design approach may be considered.

5. Summary and Conclusions

The results of this study can be summarized as follows:

1. This study showed that designing steel high-rise buildings per elastic design criterion (strength and stiffness solution) under strong wind and moderate seismicity is economically feasible. A step-by-step elastic seismic design procedure that is directly usable for practicing design engineers was also proposed.

2. Seismic design strategy per elastic criterion proposed in this study becomes more feasible as the building slenderness increases. This implies that the cost for ductile connection is saved and desirable self-centering property can be realized in tall buildings.

3. Considering the brittle nature of steel CBFs, very high uncertainties in predicting the details of future earthquakes, and the critical importance and monumental nature of high-rise buildings, sufficient system overstrength is recommended.

4. Finally, it is recommended that much simple but reasonably accurate and practically-friendly response spectrum analysis be used to predict the strength and drift demand and also to evaluate the seismic performance of high-rise steel CBFs.

Acknowledgments

Financial support to this study provided by the Ministry of Construction and Transportation of Korea (03 R&D C04-01) is gratefully acknowledged.

Notation

The following symbols are used in this paper:

a_D: wind-induced peak roof acceleration in along-wind direction;

a_r: $78.5\times10^{-3}[V_H/(n_W\sqrt{WD})]^{3.3}$, Pa;

a_W: wind-induced peak roof acceleration in across-wind direction;

C_e: exposure factor;

C_g: dynamic gust factor;

D: along-wind building dimension, m;

F: gust energy ratio;

g: acceleration due to gravity: 9.81 m/s^2;
g_p: peak factor;
K: factor related to the surface roughness coefficient of the terrain
n_D: fundamental natural frequency in along-wind direction;
n_W: fundamental natural frequency in across-wind direction;
s: size reduction factor as a function of W/H;
V_H: the mean speed at the top of the structure, m/s;
W: across-wind building dimension, m;
β_D: fraction of critical damping in along-wind direction;
β_W: fraction of critical damping in across-wind direction;
Δ: maximum wind-induced lateral deflection at the top of the building in along-wind direction, m; and
ρ_B: average density of the building, kg/m^3.

References

American Institute of Society Construction (AISC) (2001). *Manual of Steel Construction: Load and Resistance Factor Design; 3rd edn*, Chicago.

Applied Technology Council (ATC) (1978). *Tentative Provisions for the Development of Seismic Regulations for Buildings*, prepared by Applied Technology Council.

Architectural Institute of Korea (AIK) (2005). *Korean Building Code-Structural*, Architectural Institute of Korea.

CSI. (2000). *SAP2000 Analysis Reference*, Computers and Structures Inc.

Englekirk, R. (1996). *Highrise Design Considerations in Regions of Moderate Seismicity*, Special Lecture, Korea University.

Englekirk, R. (1994). *Steel Structures: Controlling Behavior through Design*, John Wiley & Sons.

Federal Emergency Management Agency (FEMA) (1997). *NEHRP commentary on the guidelines for the seismic rehabilitation of buildings, FEMA 273*, Applied Technology Council for the Building Seismic Safety Council, published by the Federal Emergency Management Agency, Washington, D.C.

http://peer.berkeley.edu/sm

Jain, A. K., Goel, S. C. (1978). Inelastic Cyclic Behavior of Bracing Members and Seismic Response of Braced Frames of Different Proportions, *Rep. No. UMEE 78R3*, Dept. of Civil Engineering, Univ. of Michigan, Ann Arbor.

National Building Code of Canada (NBCC) (1995). *NBC 1995 Structural Commentaries (Part 4)*.

Prakash, V., Powell, G. H., Campbell, S. (1993). DRAIN-2DX Base Program Description and User Guide, *Rep. No. UCB/SEMM-1933/17*, Dept. of Civil Engineering, Univ. of California, Berkeley, Calif.

Tremblay, R. (2002). “Achieving a Stable Inelastic Seismic Response for Multi-Story Concentrically Braced Steel Frames”, *Eng. J.*, 40 (2), 111-130.