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Seismic Performance of Wind-Designed Diagrid Tall Steel Buildings in Regions of Moderate Seismicity and Strong Wind

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Abstract. This study analytically evaluated the seismic performance of wind-designed diagrid tall steel buildings in regions of moderate/low seismicity and strong winds. To this end, diagrid tall steel buildings with varying wind exposure and slenderness ratio (building height-to-width ratio) conditions were designed to satisfy the wind serviceability criteria specified in the Korean Building Code and the National Building Code of Canada. A series of seismic analyses were then performed for earthquakes having 43- and 2475-year return periods utilizing the design guidelines of tall buildings. The analyses demonstrated the good seismic performance of these wind-designed diagrid tall steel buildings, which arises because significant overstrength of the diagrid system occurs in the wind design procedure. Also, analysis showed that the elastic seismic design process of diagrid tall steel buildings might be accepted based on some wind exposures and slenderness ratios.

Keywords: moderate/low seismicity; tall steel buildings; diagrid; wind design; seismic performance

1. Introduction

As specifications for seismic design have become mandatory in building design codes, there has been controversy over the direction for seismic design of tall buildings in moderate/low seismicity regions. The study of seismic design of tall buildings in high seismicity regions was not even actively implemented until the advent of the 2000's. Representative seismic design guidelines for tall buildings have been suggested by the Los Angeles Tall Buildings Structural Design Council (LATBSDC 2005,2008), the Council on Tall Buildings and Urban Habitat (CTBUH 2008), and the Pacific Earthquake Engineering Research Center (PEER 2010) of the US. And many studies on the alternative design procedure for tall buildings and ground motions for design of tall buildings in high seismicity regions have been done (Moehle 2007, Lew *et al*. 2008). Seismic design procedure for tall buildings in moderate/low seismicity regions only changed some requirements in the alternative design procedure for tall buildings in high seismicity regions (Kelly and Zona 2006). However, the design provision for tall buildings in high seismicity regions can be too conservative for tall buildings in low to moderate seismicity regions because of the reduced seismic demand (Ho 2011). Until recently, no seismic design method for tall buildings in moderate/low seismicity

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regions has been agreed with any reasonable consensus.

The Korean peninsula is located in a region of strong wind that frequently experiences typhoons during the summer season. So far, the maximum instantaneous wind speed observed in the Korean peninsula is 63.7 m/sec, in October, 2010. Typhoons are usually classified into four grades; the lower bound of the maximum wind speed of the strongest typhoon is 44 m/sec. On the other hand, the earthquake hazard level in the Korean peninsula is of low seismicity, having an effective peak ground acceleration of 0.147g, which corresponds to two-thirds of ground motion with a 2475-year return period. The other regions of the world, such as the southeast regions of the United States, Australia, and Hong Kong, fit into the same conditions of strong wind and moderate/low seismicity. In these regions, the magnitude of seismic load applied to tall buildings is relatively smaller than the magnitude of wind load. Therefore, in practice, it is common to skip or simplify the evaluation of seismic performance of tall buildings, assuming the satisfactory inelastic behavior of the structural system under seismic load. However, the structural system applied to tall buildings is the so-called undefined system; here the problem is that it is not easy to classify the system by structural type defined in the current seismic design code, which is not appropriate for tall buildings.

Globally, there has been an increasing trend of demand for tall buildings as a symbol of land-use efficiency, and as landmarks of the particular country. So in South Korea, where many tall buildings are being constructed or planned. Recently, the shape of tall buildings has developed from a simple cubic form to a freeform. As a way to actively respond to the change in shape of buildings, many structural engineers have adopted the diagrid structural system, which can effectively resist both vertical and horizontal load through using only diagonal elements.

The diagrid structural system is a kind of concentrically-braced frame. In general, concentrically-braced steel frames have been considered as a relatively brittle system, because redistribution of forces during inelastic behavior is not expected due to low redundancy and soft story response which occurs when inelastic deformation accumulates on the buckled story after braces buckle (Tremblay 2002). Lee and Kim (2007) argued that it is desirable to limit the behavior of tall concentrically-braced steel systems in the elastic range, even under very rare ground motion. They proposed an elastic seismic design procedure for tall concentrically-braced steel frames in regions of strong wind and moderate seismicity, such as the Korean peninsula. Tall buildings are designed to be structures with significant system overstrength, in order to secure the serviceability required in the wind design process. In particular, in the case of steel frame buildings having a small self-weight, the effect of wind load on building increases and the effect of seismic load decreases as the slenderness ratio (height-to-width ratio) of the building increases. As a result, the base shear due to the wind may become close to the elastic base shear due to earthquakes for tall steel buildings in regions of strong wind and moderate/low seismicity. In other word, most primary structural members of wind-designed tall steel buildings in certain conditions may remain elastic under earthquake ground motion not considering response modification factor. Thus, the elastic seismic design of tall steel buildings in this region could be economically acceptable.

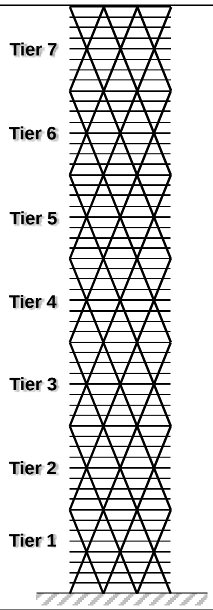
Taking into consideration these matters, this study assessed the seismic performance of wind-designed diagrid tall steel buildings in regions of moderate/low seismicity and strong wind. First, diagrid tall steel buildings with three different slenderness ratios were designed according to wind design criteria under differing wind exposure. Then, the seismic performance of the buildings was evaluated by conducting linear dynamic analysis using response spectrum method. Finally, the possibility of elastic seismic design of the buildings was assessed.

2. Wind Design of Steel-Framed Diagrid Structures for Tall Buildings

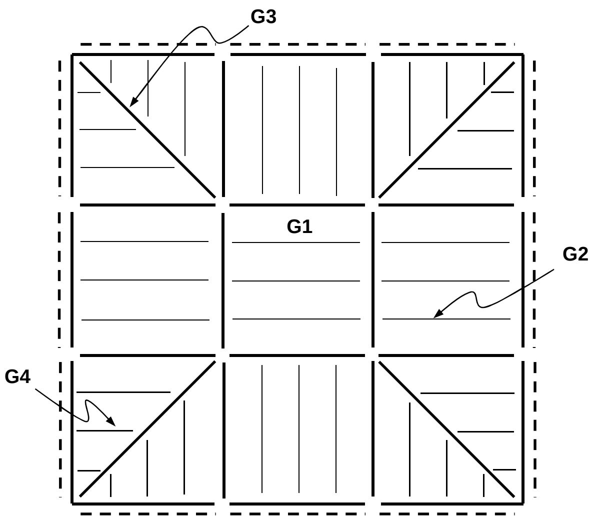
For seismic case studies, steel-framed diagrid structures for hypothetical tall buildings were designed by utilizing wind load design conditions, as indicated in Table 1. The buildings were assumed to be located in Seoul, South Korea, with various wind exposures. A basic wind speed of 30m/sec, topographic factor of 1.0, and importance factor of 1.1 were adopted from the Korean Building Code (2009). The approximate expression of the Architectural Institute of Japan (2004), introduced into the Korean Building Code (2009), was used for the first natural frequency (n_o) of building and the first damping coefficient (ζ_f) of building in the wind direction. A dead load of 4.6 kN/m^2 and live load of 2.5 kN/m^2 were applied to the buildings, respectively. The steel diagrid

Table 1Factors for wind load calculation

Factors	Value	Remark
Basic wind speed (V_o)	30 m/sec	Seoul
Topographic factor	1.0	Flat regions no affected by mountains, hills and inclined ground
Importance factor (I_w)	1.1	above 35 stories, 100 m, or slenderness of 5
First natural frequency (n_o) of a building	0.2003	1/0.02H (steel frame: slenderness of 6.9)
First damping ratio of a building in wind direction	0.0026	0.013n_o (steel frame: slenderness ratio of 6.9)



(a) Elevation view



(b) Plan view

Fig. 1 Elevation and plan view of diagrid system (slenderness ratio = 6.1) system was designed by the limit state design method (AISC 2005), to ensure that the slenderness ratio (the ratio of building height (H) to building width (d)) was within the range of 5.2 (187.2 m, 48 stories) ～ 6.9 (249.6 m, 64 stories), considering the capacity limit for the thickness of steel plate, which aimed at examining the behavorial characteristics of diagrid tall steel buildings corresponding to the change in level of wind exposure (see Fig. 1). The slenderness ratio related to the angle of diagonals is also very critical for the optimal design of diagonals (Zhang *et al*. 2012). In the diagrid frame system, the diagonal members resist both the gravity force and the lateral force, without vertical columns. Lateral force is resisted by the web frame of the building, which is parallel to the lateral force, and the flange frame of the building, which is perpendicular to the lateral force, as shown in Fig. 1.

That is, the web frame resists shear force, and the flange frame resists overturning moment. For reference purposes, Moon *et al*. (2007) conducted a study of variables for steel quantity reduction in the wind design process of the diagrid steel frame system, and stated that as limit conditions for optimal design the slenderness ratio should be no less than 5, and the tilt angle of diagonal member from the vertical axis (θ) should be in the range of 60°～70°. In addition, they suggested that the wind resisting performance of the diagrid structure is optimal when the tilt angle of diagonal member is 69°. Based on such suggestions, a diagrid frame system of eight stories was designed as one tier in this study, ensuring that the tilt angle of the diagonal member was approximately 69°. The built-up rectangular steel tubes were used for exterior diagonal members and interior gravity columns. The cross-sectional areas of diagonal members in flange frame and web frame were calculated by using Eq. (1) and Eq. (2), respectively (Moon *et al*. 2007).

$$A_{d,f} = \frac{2ML_d}{\left(N_{d,f} + \delta_d\right) w^2 E_d \chi^* h \sin^2\theta} \qquad \text{for flange frame} \tag{1}$$

$$A_{d,w} = \frac{VL_d}{2N_{d,w} E_d \gamma^* h \cos^2\theta} \qquad \text{for web frame} \tag{2}$$

where E_d is the elasticity modulus of the diagonal member, h is the height of a tier, L_d is the length of the diagonal member, M is the overturning moment of a tier, $N_{d,f}$ is the number of diagonal members in the flange frame, $N_{d,w}$ is the number of diagonal members in the web frame, s is the ratio of roof story displacement due to shear force to roof story displacement due to overturning moment (= H/d-3), V is the shear force at a tier, α is the limit variable of roof story displacement for wind-resistant serviceability design (= 500 in this study), γ^* is $1/[(1+s)\alpha]$, δ_d is the contribution of web frame to flexural stiffness (= 2 in general), and χ^* is $(2\gamma^* s)/H$. For more details, kindly refer to the references (Moon *et al*. 2007).

Steel H-shapes (wide flange shapes) were used for girders and beams. All connections were assumed as simple connections to minimize connection cost. The dimensions of diagonals, columns and beams are listed in Table 2. The material properties of steel used in diagonal members and interior gravity columns followed the nominal values for SM 490 steel, with yield strengths (F_y) of 325 MPa (for plate thickness equal to or less than 40mm) or 295 MPa (for plate thicker than 40mm, but less than 100mm), and tensile strength (F_u) of 490 MPa. The material properties of steel used in girders and beams followed the nominal values for SS 400 steel, with yield strengths of 235 MPa (for plate thickness equal to or less than 40mm) or 215 MPa (for plate thicker than 40mm, but less than 100mm), and tensile strength of 400 MPa. An elastic modulus of 2.05×10^5MPa was used for all steel members.

Table 2 Size of main structural members

(a) Diagonal members

Exposure	Slender-ness	Tier	Required area (cm2)		Section	Width – to-Thk.	Designed area (cm2)	Strength increase (%)	
			Web	Flange				Web	Flange
A	5.2	6	192.17	46.85	□-300×300×17	15.65	192.44	0.14	310.75
		5	382.54	182.92	□-425×425×24	15.71	384.96	0.63	110.45
		4	556.66	401.10	□-510×510×29	15.59	557.96	0.23	39.11
		3	712.89	693.64	□-575×575×33	15.42	715.44	0.36	3.14
		2	848.61	1,051.81	□-700×700×40	15.50	1,056.00	24.44	0.40
		1	962.12	1,465.51	□-845×845×46	16.37	1,470.16	52.80	0.32
	6.1	7	284.13	57.90	□-360×360×21	15.14	284.76	0.22	391.85
		6	568.94	226.88	□-520×520×29	15.93	569.56	0.11	151.04
		5	833.69	499.56	□-615×615×36	15.08	833.76	0.01	66.90
		4	1,076.82	868.02	□-715×715×40	15.88	1,080.00	0.30	24.42
		3	1,296.15	1,323.61	□-800×800×44	16.18	1,330.56	2.65	0.52
		2	1,488.21	1,856.65	□-945×945×52	16.17	1,857.44	24.81	0.04
		1	1,650.72	2,455.87	□-1,100×1,100×59	16.64	2,456.76	48.83	0.04
	6.9	8	393.48	71.36	□-420×420×25	14.80	395.00	0.39	453.49
		7	791.27	280.42	□-605×605×35	15.29	798.00	0.85	184.57
		6	1,165.20	619.30	□-755×755×41	16.41	1,170.96	0.49	89.08
		5	1,513.76	1,079.68	□-855×855×47	16.19	1,519.04	0.35	40.69
		4	1,835.01	1,652.62	□-955×955×51	16.73	1,844.16	0.50	11.59
		3	2,126.16	2,328.41	□-1,065×1,065×58	16.36	2,336.24	9.88	0.34
		2	2,382.85	3,096.12	□-1,240×1,240×66	16.79	3,099.36	30.07	0.10
		1	2,602.18	3,943.08	□-1,390×1,390×75	16.53	3,945.00	51.60	0.05
B	5.2	6	241.14	58.31	□-345×345×20	15.25	260.00	7.82	343.63
		5	485.81	230.61	□-480×480×27	15.78	489.24	0.71	112.15
		4	715.81	509.73	□-580×580×33	15.58	722.04	0.87	41.65
		3	928.63	888.73	□-665×665×37	15.97	929.44	0.09	4.58
		2	1,119.92	1,358.33	□-820×820×44	16.64	1,365.76	21.95	0.51
		1	1,282.04	1,908.26	□-970×970×52	16.65	1,909.44	48.94	0.06
	6.1	7	344.67	70.04	□-400×400×23	15.39	346.84	0.63	395.19
		6	697.19	276.31	□-565×565×33	15.12	702.24	0.72	154.15
		5	1,032.46	612.54	□-690×690×40	15.25	1,040.00	0.73	69.78
		4	1,348.20	1,071.75	□-815×815×44	16.52	1,356.96	0.65	26.61
		3	1,641.06	1,645.85	□-890×890×49	16.16	1,648.36	0.44	0.15
		2	1,905.24	2,325.07	□-1,065×1,065×58	16.36	2,336.24	22.62	0.48
		1	2,130.58	3,096.29	□-1,240×1,240×66	16.79	3,099.36	45.47	0.10
	6.9	8	463.42	83.85	□-460×460×27	15.04	467.64	0.91	457.69
		7	940.17	331.40	□-660×660×38	15.37	945.44	0.56	185.28
		6	1,397.17	736.23	□-825×825×45	16.33	1,404.00	0.49	90.70
		5	1,832.34	1,291.30	□-950×950×51	16.63	1,833.96	0.09	42.02
		4	2,242.80	1,988.70	□-,045×1,045×57	16.33	2,252.64	0.44	13.27
		3	2,624.32	2,819.35	□-1,185×1,185×63	16.81	2,827.44	7.74	0.29
		2	2,969.55	3,772.20	□-1,365×1,365×73	16.70	3,772.64	27.04	0.01
		1	3,265.64	4,832.48	□-1,540×1,540×83	16.55	4,837.24	48.13	0.10

Exposure	Slender-ness	Tier	Required area (cm2)		Section	Width – to-Thk.	Designed area (cm2)	Strength increase (%)	
			Web	Flange				Web	Flange

C	5.2	6	278.35	67.51	□-355×355×21	14.90	280.56	0.79	315.56
		5	565.21	267.01	□-520×520×29	15.93	569.56	0.77	113.31
		4	839.95	593.39	□-620×620×36	15.22	840.96	0.12	41.72
		3	1,099.93	1,040.52	□-715×715×41	15.44	1,105.36	0.49	6.23
		2	1,340.30	1,600.63	□-885×885×48	16.44	1,607.04	19.90	0.40
		1	1,549.04	2,262.16	□-1,050×1,050×57	16.42	2,264.04	46.16	0.08
	6.1	7	389.19	78.95	□-430×430×24	15.92	389.76	0.15	393.66
		6	792.51	312.81	□-605×605×35	15.29	798.00	0.69	155.11
		5	1,182.01	696.62	□-765×765×41	16.66	1,187.36	0.45	70.45
		4	1,555.38	1,224.68	□-875×875×47	16.62	1,556.64	0.08	27.11
		3	1,909.07	1,890.17	□-970×970×52	16.65	1,909.44	0.02	1.02
		2	2,236.64	2,684.40	□-1,145×1,145×62	16.47	2,685.84	20.08	0.05
		1	2,522.06	3,594.52	□-1,325×1,325×72	16.40	3,608.64	43.08	0.39
	6.9	8	513.41	92.76	□-490×490×28	15.50	517.44	0.79	457.83
		7	1,047.61	367.98	□-695×695×40	15.38	1,048.00	0.04	184.80
		6	1,566.39	820.70	□-885×885×47	16.83	1,575.44	0.58	91.96
		5	2,067.67	1,445.34	□-1,015×1,015×54	16.80	2,075.76	0.39	43.62
		4	2,548.53	2,235.46	□-1,125×1,125×60	16.75	2,556.00	0.29	14.34
		3	3,004.50	3,183.37	□-1,255×1,255×67	16.73	3,183.84	5.97	0.01
		2	3,427.40	4,279.29	□-1,450×1,450×78	16.59	4,280.64	24.89	0.03
		1	3,796.96	5,508.69	□-1,655×1,655×88	16.81	5,515.84	45.27	0.13
D	5.2	6	303.07	73.40	□-370×370×22	14.82	306.24	1.05	317.22
		5	618.94	291.38	□-535×535×31	15.26	624.96	0.97	114.49
		4	925.73	650.12	□-665×665×37	15.97	929.44	0.40	42.96
		3	1,221.09	1,144.97	□-770×770×42	16.33	1,223.04	0.16	6.82
		2	1,500.66	1,769.77	□-935×935×50	16.70	1,770.00	17.95	0.01
		1	1,751.10	2,514.69	□-1,110×1,110×60	16.50	2,520.00	43.91	0.21
	6.1	7	417.23	84.54	□-445×445×25	15.80	420.00	0.66	396.83
		6	853.74	335.98	□-630×630×36	15.50	855.36	0.19	154.58
		5	1,280.09	750.73	□-790×790×43	16.37	1,284.84	0.37	71.15
		4	1,694.31	1,324.53	□-915×915×49	16.67	1,697.36	0.18	28.15
		3	2,093.31	2,052.18	□-1,010×1,010×55	16.36	2,101.00	0.37	2.38
		2	2,471.25	2,926.80	□-1,195×1,195×65	16.38	2,938.00	18.89	0.38
		1	2,810.37	3,937.45	□-1,390×1,390×75	16.53	3,945.00	40.37	0.19
	6.9	8	543.11	98.02	□-500×500×29	15.24	546.36	0.60	457.41
		7	1,112.86	389.91	□-720×720×41	15.56	1,113.56	0.06	185.60
		6	1,671.47	872.10	□-905×905×49	16.47	1,677.76	0.38	92.38
		5	2,217.20	1,540.53	□-1,050×1,050×56	16.75	2,226.56	0.42	44.53
		4	2,747.59	2,390.40	□-1,170×1,170×62	16.87	2,747.84	0.01	14.95
		3	3,258.71	3,415.84	□-1,310×1,310×70	16.71	3,425.16	6.55	1.64
		2	3,743.15	4,609.09	□-1,505×1,505×81	16.58	4,613.76	23.26	0.10
		1	4,178.44	5,957.80	□-1,715×1,715×92	16.64	5,972.64	42.94	0.25

(b) Girders, beams, and gravity columns

Slenderness	Member	Section (beam depth×beam width×web thickness×flange thickness)
5.2 ~ 6.9	G1	H-900×300×16×28
	G2	H-506×201×11×19
	G3	H-890×299×15×23
	G4	H-340×250×9×14
	Gravity column	□-455×455×27 ~ □-1340×1340×71

The roof story displacement, which is the serviceability requirement against wind load, was limited to be less than 1/500 of the building height in the process of calculating the cross-sectional area of diagonal members in Eq. (1) and Eq. (2) (see Table 3). The width-to-thickness ratio of the sections of diagonal members was also limited to satisfy seismic design criteria (AISC, 2005). Then, as shown in the 4th and 5th columns (area of web and flange frame) of Table 2 (a), the required total cross-sectional areas of diagonal members in web and flange frame at each tier were respectively determined. Consequently, considering an arbitrary wind direction, the sectional size of the diagonal members in both web frame and flange frame should be designed to be identical. In other words, the resulting_cross-sectional areas of designed diagonal members at each tier were calculated as listed in the 8th column (designed section) of Table 2(a). It should be noted that the size of diagonal members tends to increase significantly, especially in the upper part of the flange frame of buildings. These overstrength factors are expected to make it possible that the diagrid tall steel buildings may behave elastically under moderate or weak earthquake. Table 4 summarized the model base shears induced by wind and seismic loads. The values indicate the possibility of elastic behavior of the wind-designed buildings subjected to such moderate or weak earthquake.

As wind-induced vibration of a building causes unpleasant feelings for building residents, it is generally a requirement in the process of wind design to investigate wind-induced vibration acceleration of the building (AIK, 2009; NBCC, 2005). According to the National Building Code of Canada (2005), the building should be checked for design wind load and its effect by performing static analysis, dynamic analysis, or wind-tunnel test. Static procedure targets most mid-rise and low-rise buildings, and dynamic procedure targets tall buildings of a height of 120 m or higher, as well as slender buildings. Since the buildings considered in this study stand more than 120 m high, the dynamic procedure was applied to calculate the vibration accelerations of the buildings in both the along-wind and the across-wind directions. As suggested by NBCC (2005), a one-hour average wind speed with a return period of 10 years was used as follows:

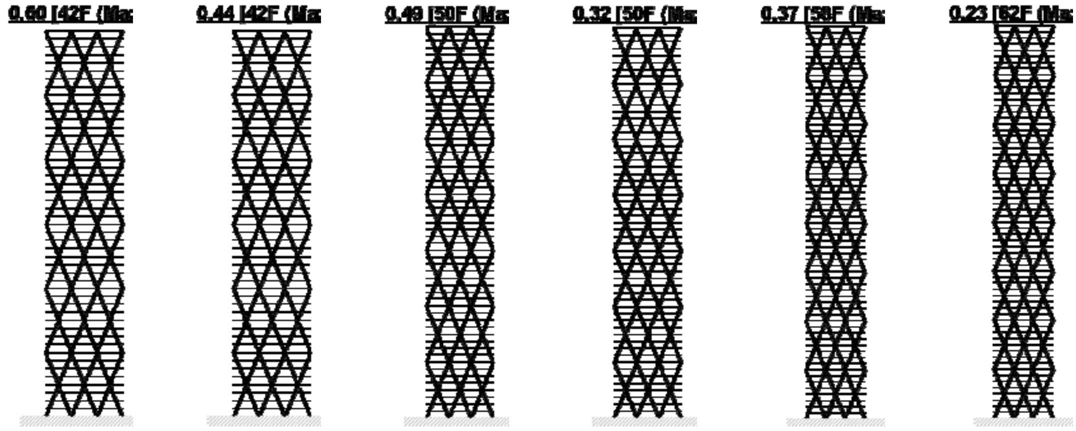
Wind-induced vibration acceleration in the along-wind direction

$$a_D = 4\pi^2 f_{nD}^2 g_p \sqrt{\frac{KsF}{C_{eH}\beta_D}}\left(\frac{\Delta}{C_g}\right) \tag{3}$$

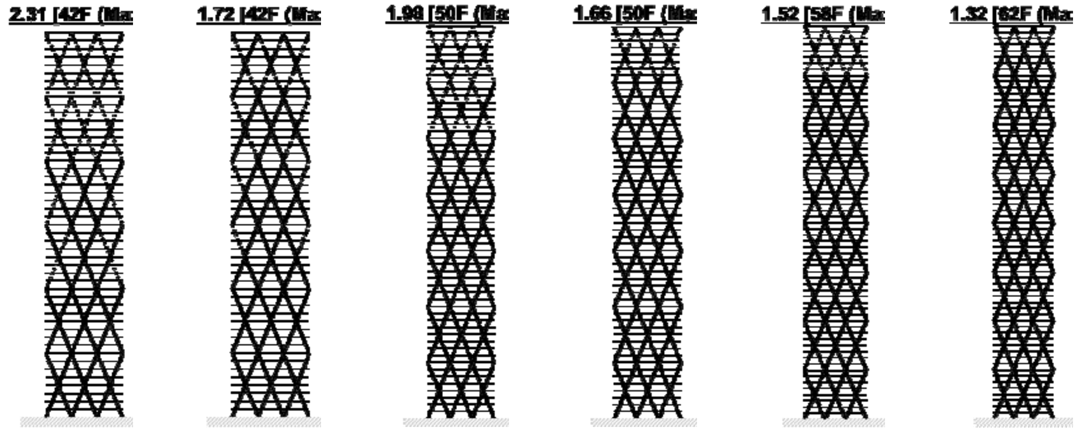
Wind-induced vibration acceleration in the across-wind direction

$$a_w = f_{nW}^2 g_p \sqrt{wd}\left(\frac{a_r}{\rho_B g\sqrt{\beta_W}}\right) \tag{4}$$

where a_r is $78.5\times10^3\left(V_H / f_{nW}\sqrt{wd}\right)^{3.3}$ (N/m^3), C_{eH} is the height distribution coefficient of wind speed according to exposure classification, C_g is the dynamic gust factor, F is the gust energy ratio, g is the acceleration of gravity (= 9.81 m/s^2), g_p is the peak factor, K is the surface roughness coefficient of the terrain, f_{nD} is the fundamental natural frequency in the along-wind direction, f_{nW} is the fundamental natural frequency in the across-wind direction, s is the size reduction factor



Web Flange Web Flange Web Flange
Slenderness 5.2 Slenderness 6.1 Slenderness 6.9
(a) SLE



Web Flange Web Flange Web Flange
Slenderness 5.2 Slenderness 6.1 Slenderness 6.9
(b) MCE

Fig. 2 DCR distribution from response spectrum analysis (Exposure A) (—: elastic member, …: inelastic member)

according to the aspect ratio of the building, w is the width of building in the across-wind direction (m), β_D is the critical damping fraction in the along-wind direction, β_W is the critical damping fraction in the across-wind direction, ρ_B is the average density of the building (= 120.3 kg/m^3), and Δ is the maximum wind-induced lateral displacement at the top of the building in the along-wind direction (m).

The wind speed with a return period of 10 years, V_H, which is required to calculate the wind-induced acceleration, was obtained by utilizing the Gumbel statistics distribution equation (KBC, 2009) as follows, based on data provided by the Korea Meteorological Administration.

$$V_{(T)} = -\frac{1}{a}\ln\left[\ln\left(\frac{T}{T-1}\right)\right] + b \tag{5}$$

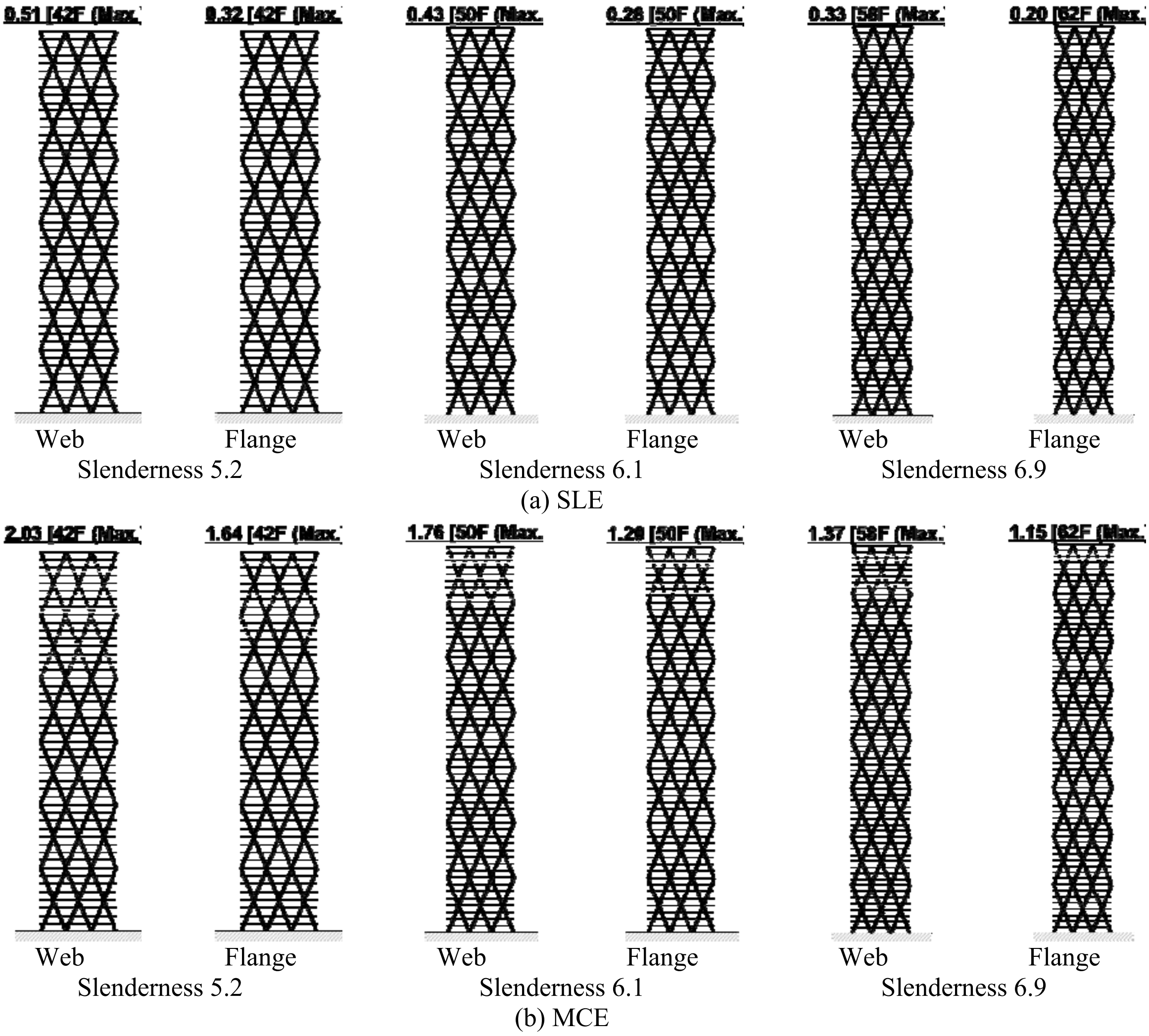


Fig. 3 DCR distribution from response spectrum analysis (Exposure B) (—: elastic member, …: inelastic member)

Table 3 Roof displacement check

Slenderness	MIDAS Gen				Limit (H/500)
	Exposure A	Exposure B	Exposure C	Exposure D	
5.2	29.10 cm	28.43 cm	28.64 cm	28.54 cm	37.44 cm
6.1	34.40 cm	33.74 cm	33.93 cm	33.74 cm	43.68 cm
6.9	39.59 cm	38.96 cm	39.13 cm	38.99 cm	49.92 cm

where, a (= 0.42) and b (= 14.32) are characteristic values of the Gumbel extreme value distribution in Seoul, $V_{(T)}$ is the wind speed with a return period of T years, and T is the time (year).

Generally, in the case of tall buildings [usually in the case of $(wd)^{1/2}/H \leq 1/3$], it is known that wind-induced vibration in the across-wind direction causes a greater problem to serviceability than wind-induced vibration in the along-wind direction. When a preliminary assessment of tall buildings is conducted, the wind-induced vibration acceleration due to wind speed with a return period of 10 years generally lies in the range of 1～3% of the acceleration of gravity. For example, most tall buildings constructed in North America from 1975 to 2000 were designed to have wind-induced vibration acceleration within the range of 1.5～2.5% of the acceleration of gravity, through the result of wind-tunnel tests (NBCC, 2005). In general, the lower limit value in this range is applied to residential buildings, while the upper limit value is applied to office buildings.

In addition, KBC (2009) classifies the wind exposure to four levels (A, B, C, and D), whereas NBCC (2005) classifies the wind exposure to three levels (A, B and C). The wind exposures A and B in KBC correspond to the wind exposure C in NBCC, the wind exposure C in KBC corresponds to the wind exposure B in NBCC, and the wind exposure D in KBC corresponds to the wind exposure A in NBCC, respectively. As shown in Table 5, it is confirmed that the diagrid frame system designed in this study satisfied all serviceability criteria (i.e. 30 gal or less for an office building) against wind-induced vibration accelerations in both the along-wind direction and the across-wind direction.

3. Seismic Performance Evaluation Based on Linear Dynamic Procedure

In this section, the seismic performance of diagrid tall steel buildings designed in the previous section was evaluated and the possibility of elastic response of diagonal members was checked by conducting linear dynamic analysis using response spectrum method.

Table 4 Comparisons of model base shears from wind and seismic loads

Slenderness	Wind load (kN)				Seismic load (kN)
	Exposure A	Exposure B	Exposure C	Exposure D	
5.2	17,731.4	23,627.5	28,548.2	32,272.1	18,838.4
6.1	23,938.7	30,897.6	36,574.9	40,755.9	19,578.2
6.9	31,107.3	39,038.6	45,390.2	49,950.5	20,244.5

Table 5 Wind-induced vibration acceleration check per NBCC 2005

Exposure		Vibration acceleration (gal)				
KBC 2009	NBCC 2005	Wind direction	Slenderness			Limit
			5.2	6.1	6.9	
A, B	C	Along-wind	1.50	2.07	2.71	30 for office building
		Across-wind	1.70	2.73	4.11	
C	B	Along-wind	1.71	2.25	2.84	
		Across-wind	2.64	4.00	5.70	
D	A	Along-wind	1.86	2.35	2.86	
		Across-wind	3.55	5.05	6.84	

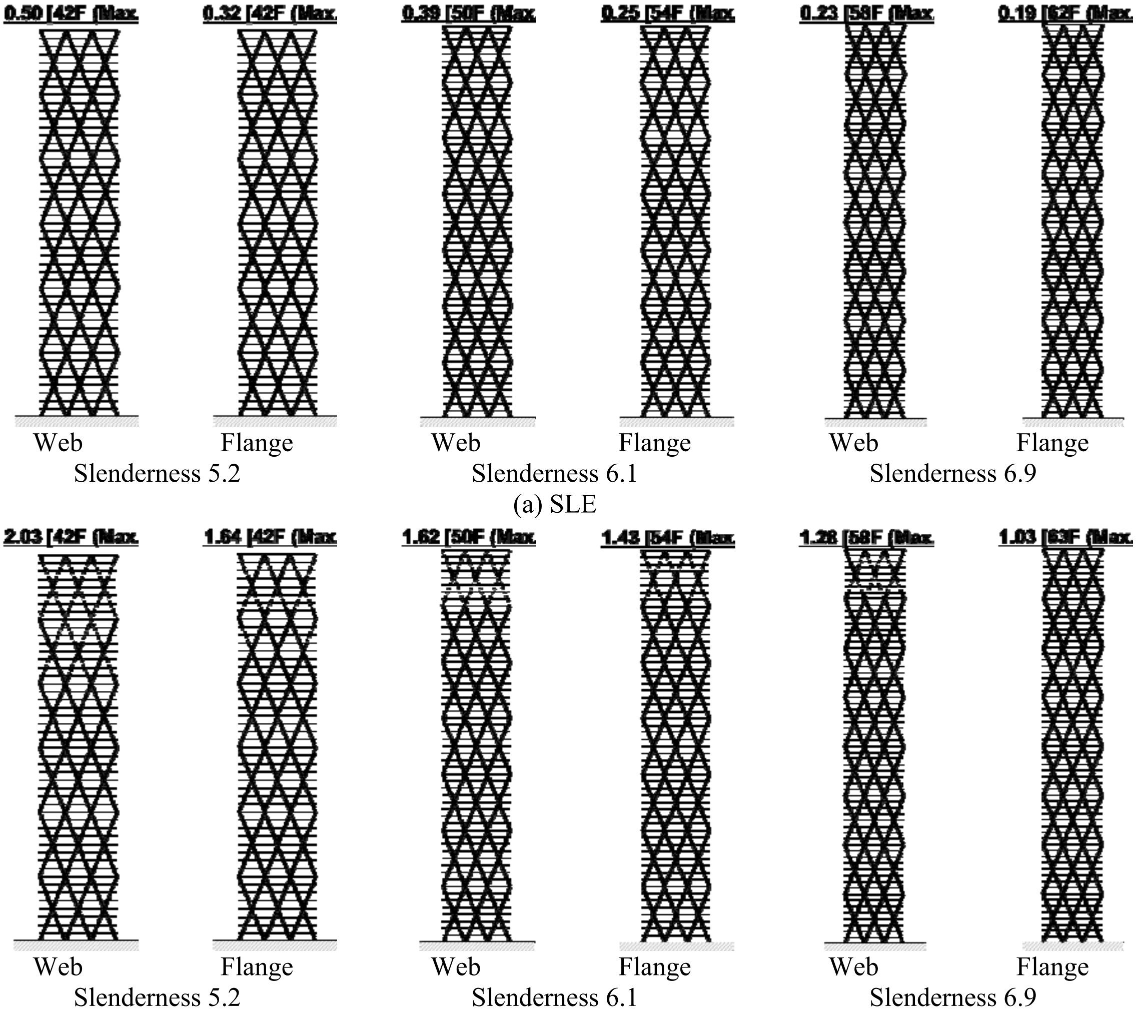


Fig. 4 DCR distribution from response spectrum analysis (Exposure C) (—: elastic member, …: inelastic member)

First, with regard to seismic performance evaluation, it shall be noted that the current tall building design guidelines, such as CTBUH (2008), LATBSDC (2008), and TBI (PEER 2010), do not suggest the standard procedure to evaluate the seismic performance of mega structural members in tall buildings, and recommend following the procedure in ASCE 41-06 (2007). Therefore, this paper adopted hazard levels and target building performance levels by tall building design guidelines and followed evaluation procedure by ASCE 41-06 (2007).

CTBUH (2008), LATBSDC (2008), and TBI (PEER 2010), which are the latest seismic design guidelines for tall buildings, define two basic earthquake hazard levels: service level earthquake (SLE) and maximum considered earthquake (MCE). SLE hazard level corresponds to ground motion that has a 50% probability of exceedance in 30 years (or has a return period of 43 years; 50%/30years). MCE hazard level corresponds to the earthquake with a return period of 2475 years

(2%/50years). Therefore, response spectrums were developed for ground motions with a return period of 43 years (SLE) and with a return period of 2475 years (MCE), which correspond to levels of design peak ground acceleration (namely, effective peak ground acceleration) of about 0.044g and 0.22g, respectively, in South Korea. Site class D (stiff soil) was applied and spectral acceleration parameter at short period and at one-second period was calculated for response spectrums (ASCE 2010). No response modification factor was applied.

ASCE 41-06 (2007) defines three target building performance levels: Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP). The design guidelines of tall buildings (CTBUH 2008. LATBSDC 2008, PEER 2010) recommend that the target seismic performance levels for tall buildings should be IO at an SLE and CP at an MCE. Such target performance levels are in accordance with other various standards, such as ASCE 7-10 (2010), LATBSDC alternative procedure (2008), etc.

Lee and Kim (2007) demonstrated that linear dynamic analysis using response spectrum method for seismic performance evaluation of tall buildings showed more conservative results than linear dynamic analysis using time history method. Therefore, this study conducted seismic performance evaluation of diagrid tall steel buildings based on linear dynamic analysis using response spectrum method. Seismic response analysis of low- and mid-rise buildings usually considers the effect of bi-directional earthquake at a ratio of 100:30 (KBC 2009. ASCE 2010). However, the recent tall building design guidelines (LATBSDC 2008. PEER 2010) suggest the effect of bi-directional earthquake at a ratio of 100:100. This study adopted the orthogonal effect of ground motions suggested by the tall building design guidelines.

ASCE 41-06 (2007) considers the action of diagonal members in steel braced frame as deformation-controlled action. In linear analysis procedure, the seismic performance level of deformation-controlled member is evaluated by using the m-factor of Eq. (6) that is the value that indicates the expected ductility of the member.

$$m = \mathrm{DCR}/\kappa \tag{6}$$

Table 6 Seismic performance evaluation criteria of steel diagonal member from linear analysis procedure per ASCE 41-06

Rectangular cold-formed steel tube	m-factor for primary member		
	Immediate ccupancy	Life safety	Collapse prevention
$d/t \leq 236.4/\sqrt{F_y}$	1.25	5	7
$d/t \geq 499.0/\sqrt{F_y}$	1.25	2	3
$236.4/\sqrt{F_y} \leq d/t \leq 499.0/\sqrt{F_y}$	Linear interpolation shall be used.		

where DCR is the demand-to-capacity ratio and κ is the knowledge factor which is the index to reflect the uncertainty of material properties and seismic rehabilitation objective.

In this study, since the value of the knowledge factor was selected as 1.0, the m-factor value is identical to the DCR value. The strength demand for diagonal members was obtained from the SRSS (square root of sum of squares) values resulting from the linear dynamic analysis using MIDAS Genw (2010). The strength capacity of diagonal members was obtained from the strength equations for the flexural-compression members in the AISC-LRFD manual (AISC, 2005) with the strength reduction factor of 1.0. The expected yield strength was applied for calculation of strength capacity of diagonal members. Table 6 shows the seismic performance evaluation criteria

Table 7 Maximum *m*-factor and seismic performance level of diagonal members from response spectrum analysis

			Slenderness ratio					
			5.2		6.1		6.9	
Exposure	Earthquake		Web	Flange	Web	Flange	Web	Flange
A	SLE	m-factor	0.60	0.40	0.49	0.32	0.37	0.23
		Performance level	IO	IO	IO	IO	IO	IO
		Remark	42F	42F	50F	50F	58F	62F
	MCE	m-factor	2.31	1.72	1.98	1.66	1.52	1.32
		Performance level	LS	LS	LS	LS	LS	LS
		Remark	42F	42F	50F	50F	58F	62F
B	SLE	m-factor	0.51	0.32	0.43	0.28	0.33	0.20
		Performance level	IO	IO	IO	IO	IO	IO
		Remark	42F	42F	50F	50F	58F	62F
	MCE	m-factor	2.03	1.64	1.76	1.20	1.37	1.15
		Performance level	LS	LS	LS	LS	LS	IO
		Remark	42F	42F	50F	54F	58F	62F
C	SLE	m-factor	0.50	0.32	0.39	0.25	0.23	0.19
		Performance level	IO	IO	IO	IO	IO	IO
		Remark	42F	42F	50F	54F	58F	62F
	MCE	m-factor	2.03	1.64	1.62	1.43	1.28	1.03
		Performance level	LS	LS	LS	LS	LS	IO
		Remark	42F	42F	50F	54F	58F	63F
D	SLE	m-factor	0.48	0.31	0.37	0.24	0.22	0.18
		Performance level	IO	IO	IO	IO	IO	IO
		Remark	42F	42F	50F	54F	58F	62F
	MCE	m-factor	1.95	1.60	1.52	1.37	1.24	1.00
		Performance level	LS	LS	LS	LS	IO	IO
		Remark	42F	42F	50F	54F	58F	63F

for rectangular cold-formed steel tube used in diagonal members. The limits of the width-to-thickness ratio (d/t) in the table were converted into SI units.

Table 7 shows the maximum *m*-factor values and the corresponding seismic performance level of critical diagonal members in each model. In the SLE, all models with different slenderness ratios satisfied the target performance level of IO regardless of exposure. In the MCE, all models demonstrated the seismic performance levels of IO or LS beyond the target performance level of CP, regardless of the exposure. In particular, in the MCE, the model with slenderness ratio of 6.9 demonstrated that the critical diagonal members in the web frame satisfied the seismic performance level of IO in the exposure of D, and the members in the flange frame satisfied the seismic performance level of IO in the exposure of B, C, and D. It was also recognized that the seismic performance of wind-designed diagrid tall steel buildings improved as the slenderness ratio became larger.

Next, the possibility of elastic seismic design of wind-designed diagrid tall steel buildings was tried to be assessed. The elastic and inelastic behaviors of the diagonal members in the diagrid frame are easily determined based on the DCR (or *m*-factor in this study). It can be interpreted that a diagonal member responds elastically to the given earthquake ground shaking if the controlling DCR for the member is less than or equal to 1.0 and, otherwise, a member responds inelastically to the earthquake ground shaking (ASCE 41-06, 2007). Therefore, it is possible to estimate the degree of elastic and inelastic behaviors of the structural system based on the DCR distribution of major structural members.

The distribution of the DCR resulting from the linear dynamic analysis of wind-designed diagrid tall steel buildings according to exposure is illustrated in Figs. 2 to 5, respectively. In general, the degrees of plasticity of models were conspicuously greater in the web frame that takes up shear force than in the flange frame that takes up overturning moment, regardless of slenderness ratios and exposure. Furthermore, it can be generally said that the greater the slenderness ratio becomes, the greater the possibility of elastic resistance provided by the diagrid frame. That is, the feasibility of elastic seismic design of wind-designed diagrid tall steel buildings increases if wind-designed diagrid tall steel buildings have substantial slenderness ratios. The reason is that, as mentioned by Lee *et al*. (2007) and in the previous chapter, the seismic spectral acceleration is significantly reduced due to extension of the fundamental vibration period of tall buildings, and a considerable system overstrength is provided to satisfy the serviceability conditions of wind design (see Table 2).

In regard to the SLE, all models showed that all diagonal members of the web and flange frames had the possibility of elastic resistance, that is, all models could adequately resist elastically regardless of slenderness ratios and exposure. Therefore, it can be said that the elastic seismic design of diagrid tall steel buildings in any wind exposures is possible when the buildings are subjected to SLE ground shaking.

On the other hand, with respect to the MCE, the model with slenderness ratio of 5.2 showed that all diagonal members of both the web frame and the flange frame experienced significant plasticity across the whole structure regardless of the exposure. In the model with slenderness ratio of 6.1, most diagonal members of the uppermost tier of the web and flange frames experienced plasticity in any exposure. In the model with slenderness ratio of 6.9, some diagonal members of the uppermost tier of the web and flange frame showed a slight inelastic behavior (or DCR values are a little greater than 1.0) in any exposure. Getting insight into the DCR distributions and the strength ratios of diagonal members in the model with slenderness ratio of 6.9, it was found out that the proportion of inelastic members was about 13.6 percent of total members in the wind exposure A, 10.3 percent of total members in the wind exposure B, 6.7 percent of total members in the wind exposure C, and 1.6 percent of total members in the wind exposure D, respectively. Therefore, it is expected that a little effort makes the diagrid structure behave elastically under MCE when the proportion of inelastic members is not greater than 10 percent of total members and the members show a slight inelastic behavior. In this study, it is suggested that the elastic seismic design of diagrid tall steel buildings in the wind exposures of B, C, or D, if their slenderness ratios is 6.9 or more, is possible when the buildings are subjected to MCE ground shaking.

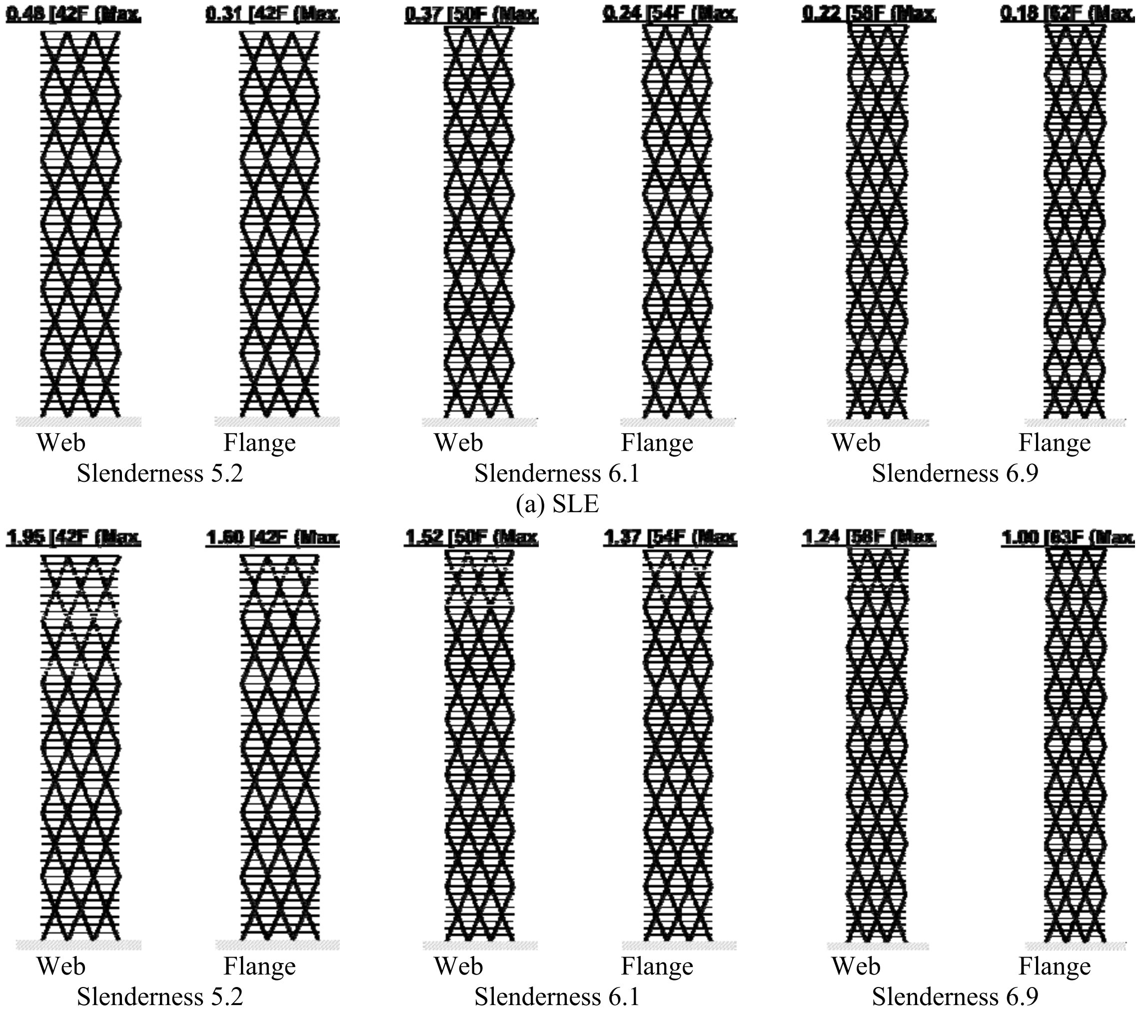


Fig. 5 DCR distribution from response spectrum analysis (Exposure D) (—: elastic member, …: inelastic member)

ASCE 41-06 (2007) suggests that the seismic performance level of structural system is also evaluated based on the maximum story drift. In this study, the seismic performance evaluation based on the maximum story drift was excluded, because it was difficult to evaluate the precise seismic performance level of structure when the structure showed distinct inelastic behavior (or the DCR values were much greater than 1.0) and to take additional consideration for damages such as rupture in connections.

4. Conclusions

In region of strong wind and low/moderate seismicity, such as Korean peninsula, the seismic performance of diagrid tall steel buildings wind-designed according to exposure was evaluated and the elastic seismic design possibility of the buildings was studied. To this end, linear dynamic analyses using response spectrum method were carried out for tall buildings with three different slenderness ratios of 5.2, 6.1, and 6.9. The results of this study can be summarized as follows.

(1) Wind-designed diagrid tall steel frames satisfied the seismic performance objective because they tend to have enough system overstrength, due to design to resist wind load. All models showed the elastic seismic performance under ground motion with peak ground acceleration of 0.044g (SLE) and the seismic performance level of Life Safety under ground motion with peak ground acceleration of 0.22g (MCE), while the target performance level is Immediate Occupancy for SLE and Collapse Prevention for MCE, respectively.

(2) Analysis showed that the seismic performance levels of diagrid tall steel buildings were more distant from the Immediate Occupancy level as the slenderness ratio of the building decreased and the exposure shifted from D to A. In general, diagrid tall steel buildings showed greater plasticity in the flange frame that takes up overturning moment, than in the web frame that takes up shear force.

(3) Finally, it was confirmed that, in the region of strong wind with the basic wind speed of 30 m/s or more, the diagrid frame of wind-designed tall steel building with slenderness ratios of 5.2 or more could elastically resist SLE ground motion with peak ground acceleration of 0.044g, regardless of wind exposure levels. Also, in such strong wind region, the elastic seismic design strategy of diagrid tall steel building subjected to MCE ground motion with peak ground acceleration of 0.22g may be accepted if the building has the slenderness ratios of 6.9 or more and is located in the wind exposure of B, C, or D.

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