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Potentials of elastic seismic design of twisted high-rise steel diagrid frames

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Abstract. This paper is to investigate the potentials of the elastic seismic design of twisted high-rise steel diagrid frame buildings in the strong wind and moderate/low seismicity regions. First, the prototypes of high-rise steel diagrid frames with architectural plans that have a twist angle of 0 (regular-shaped), 1, and 2 degrees were designed to resist wind. Then, the effects of the twist angle on the estimated quantities and structural redundancies of the diagrid frames were examined. Second, the seismic performance of the wind-designed prototype buildings under a low seismicity was evaluated. The response spectrum analysis was conducted for the service level earthquake (SLE) having 43-year return period and the maximum considered earthquake (MCE) having 2475-year return period. The evaluation resulted that the twisted high-rise steel diagrid frames resisted the service level earthquake elastically and most of their diagrid members remained elastic even under the maximum considered earthquake.

Keywords: high-rise steel diagrid frames; twist angle; quantities; redundancy; seismic performance; elastic seismic design

1. Introduction

High-rise buildings have been constructed as an alternative to solve social, economic, and cultural problems, according to the growing centralization of cites worldwide. Since the late 1980's, researches related to the seismic design of high-rise buildings have been actively conducted worldwide. In the 2000's, seismic design guidelines of high-rise buildings were introduced in the United States (CTBUH 2008, LATBSDC 2008, PEER 2010). They mentioned that the current seismic design provisionsfor buildings, such as ASCE7-10 (ASCE 2010), KBC (AIK 2009) and so on, have many problems to be applied to high-rise buildings, since they apply to moderate- and low-rise buildings with a building height of less than 100 m.

Key problems are the following. First, the response modification factors applied to determine seismic design loads are uncertain for high-rise buildings. They are based on the behavior and damage of the moderate- and low-rise building structural systems. Also, the structural systems of

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high-rise buildings are not easy to be classified as a structural system defined in the current seismic design code. Second, the high-rise buildings should be checked to resist not only the seismic design load but also maximum considered earthquake. Considering the social symbolism and pervasiveness of high-rise buildings, the desirable design goal of the high-rise buildings should be operable or repairable even after they experience very rare ground motion such as maximum considered earthquake. Third, a recent trend in design of high-rise buildings is to design an irregular building with tapered shape, twisted shape, or a tilted shape. This implies that the requirements and criteria in the structural design of high-rise buildings become more complex and sophisticated.

Therefore, the reasonable seismic design load for high-rise buildings and their seismic performance including sophisticated and inelastic behavior should be carefully considered in the seismic design procedure. However, these problems cannot be solved easily. If anything, the elastic seismic design of regular-shaped or irregular-shaped high-rise buildings may be the solution in special cases.

Many studies on the design ground motions and seismic design procedure for high-rise buildings in the regions of high seismicity have been done (Lew *et al*. 2008, Moehle 2007). However, Ho (2011) insisted that the seismic design provision for high-rise buildings in the regions of high seismicity may be too conservative for high-rise buildings in the regions of low seismicity because of the reduced seismic demand. Lu *et al*. (2012) investigated the seismic behavior of a 53-story high-rise building with a lateral resisting system of outrigger systems using the shaking table test and the numerical analysis. Balendra *et al*. (2013) investigated the range of the over-strength, that is, a component of seismic capacity, of high-rise building in Singapore using dynamic collapse analysis. Çelebi *et al*. (2014) addressed how structures sensitive to low-frequency motions can be affected by sources through responses of a high-rise building at 770 km from the epicenter of great ground motions (Great Eat Japan earthquake) occurred in 11 March 2011. And Wei and Qing-Ning (2012) proposed a design procedure of the high-rise buildings beyond the code specification and verified the feasibility of performance-based seismic design for it.

The earthquake hazard level in the Korean Peninsula is classified as low seismicity. On the other hand, the Korea Peninsula is a region of strong winds including typhoons occurring frequently in the summer season. The southeast regions of the United States, Hong Kong, and Australia, have the similar conditions of strong winds and moderate/low seismicity. In the regions of a strong wind and moderate/low seismicity, the wind design load is very large and the elastic wind design of buildings is performed. Therefore, it is expected that high-rise buildings are designed to be structures with significant system redundancy or over-strength in the wind design process. Lee and Kim (2007) and Kim and Lee (2013) argued that the elastic seismic design of regular-shaped high-rise steel frame buildings with slenderness ratio limit could be acceptable in regions of strong wind and moderate/low seismicity.

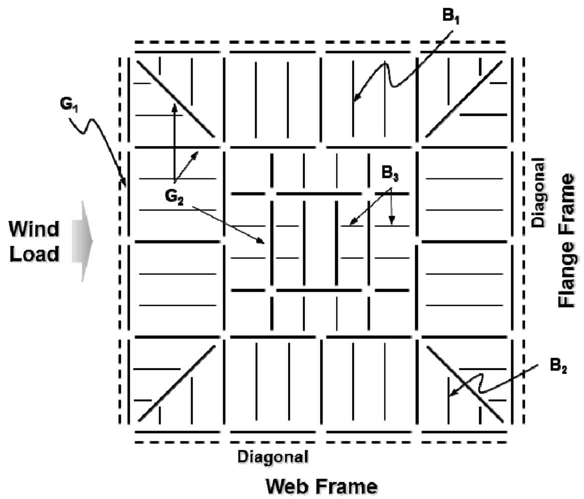
This paper aims to investigate the potentials of the elastic seismic design of twisted high-rise steel diagrid frame buildings in the strong wind and moderate/low seismicity regions. To this end, the high-rise steel diagrid frame buildings with different twist angles were first elastically designed to resist a design strong wind. Then, quantity analysis was conducted to estimate the total designed quantities and structural redundancies (or over-strengths) of the diagrid members in each building. Second, the seismic performance of the wind-designed buildings was evaluated by response spectrum analysis. Based on analysis results, the possibility of the elastic seismic design of twisted high-rise steel diagrid frame buildings was estimated.

Table 1 Conditions to calculate the wind design load

Design condition	Value	Remark
Basic wind speed	30 m/sec	Near Seoul (Exposure B)
Importance factor	1.1	Building with greater than 35 floors, 100 m or slenderness of 5
Topographic factor	1.0	Flat terrain
First natural frequency of a building	0.128 Hz	46/*H* for the slenderness ratio of 7
First damping ratio in the along-wind direction of a building	0.01	Steel frames

A. Regular-shape B. Twist angle of 1 degree C. Twist angle of 2 degrees

(a) Three-dimensional perspective view of prototype models



(b) Typical structural plan

Fig. 1 Structural models of prototype high-rise steel diagrid frames

2. Wind design of twisted irregular high-rise steel diagrid frames

Irregular high-rise steel diagrid frame buildings with different inter-story twist angles of architectural plan were first wind-designed before analyzing the economics and evaluating their seismic performance.

In practice, the height of irregular high-rise buildings is limited to around 60 stories because of the insufficient knowledge and experience of structural design. So, 234 m-high 60-story prototype buildings near Seoul on flat terrain were assumed in this study. Then, the design wind load was calculated by using the basic wind speed of 30 m/sec, importance factor of 1.1, and topographic factor of 1.0 as summarized in Table 1. Note that the basic wind speed is defined as the 10-minute average wind speed of 10 m above the ground level having a 100-year return period. A natural frequency of building of 0.128Hzwas calculated based on Elli's equation (AIK 2009). The first damping coefficient of buildings in the wind direction was assumed to be 0.01.A dead load of 4.6 kN/m^2 and a live load of 2.5 kN/m^2 were applied to the buildings, respectively.

Moon *et al*. (2007) suggested that diagrid frames are economical and efficient to resist wind load when the building slenderness ratio is greater than 5 and the inclined angle of diagrid members is about 60 degrees. Based on such suggestion, building slenderness ratio of 6.5 and the inclined angle of diagrid members of 60 degrees were selected. Building slenderness ratio is defined as the ratio of building height (*H*, 234 m) to building width (36 m). Also, structural plans suggested by Moon (2011) were adopted for the prototype buildings.

Then, three prototype buildings with twist angles of 0 (regular-shaped), 1, and 2 degrees were designed based on AISC- LRFD (AISC 2005) (see Fig. 1(a)). Typically, as a lateral load resisting system, the web frames of the regular-shape building parallel to the lateral load were designed to resist shear force and the flange frames perpendicular to the lateral load were designed to resist overturning moment (see Fig. 1(b)).

The diagrid members in every four floors are unified as one tier having the same member size. Steel built-up circular tube sections were used for the exterior diagrid members because the circular tube section possesses the great resistance capacity against torsion in irregular

Table 2 Member sizes of the diagrid members in the prototype buildings

(a) Regular model

Tier	Required cross-sectional area (cm^2)	Section (○-external diameter*thickness)	Designed cross-sectional area (cm^2)	Width-to-thickness ratio		Strength increase (%)
				Design	Limitation	
15	67.55	○-730*25	553.71	27.20	27.75	719.7
14	82.01	○-840*29	738.87	26.97		800.9
13	102.98	○-970*34	999.78	26.53		870.9
12	125.91	○-1,040*34	1,165.88	26.11		825.9
11	299.73	○-1,080*39	1,275.46	25.69		325.5
10	342.33	○-1,100*40	1,332.04	25.50		289.1
9	435.65	○-1,120*40	1,357.17	26.00		211.5
8	481.04	○-1,140*40	1,382.30	26.50		187.4
7	584.08	○-1,160*40	1,407.43	27.00		141.0

(a) Continued

Tier	Required cross-sectional area (cm^2)	Section (○-external diameter*thickness)	Designed cross-sectional area (cm^2)	Width-to-thickness ratio		Strength increase (%)
				Design	Limitation	
6	664.71	○-1,180*40	1,432.57	27.50	30.58	115.5
5	816.50	○-1,190*40	1,445.13	27.75		77.0
4	953.13	○-1,210*41	1,505.73	27.51		58.0
3	1,054.01	○-1,210*41	1,505.73	27.51		42.9
2	1,068.71	○-1,240*41	1,544.38	28.24		44.5
1	1,395.85	○-1,260*41	1,570.14	28.73		12.5

(b) Model of the twist angle of 1 degree

Tier	Required cross-sectional area (cm^2)	Section (○-external diameter*thickness)	Designed cross-sectional area (cm^2)	Width-to-thickness ratio		Strength increase (%)
				Design	Limitation	
15	125.69	○-730*25	553.71	27.20	27.75	340.5
14	140.99	○-880*30	801.11	27.33		468.2
13	263.12	○-1,000*34	1031.82	27.41		292.2
12	337.32	○-1,100*37	1235.62	27.73		266.3
11	466.29	○-1,150*44	1528.82	24.14	30.58	227.9
10	564.58	○-1,180*42	1501.56	26.10		166.0
9	690.08	○-1,210*45	1646.98	24.89		138.7
8	792.33	○-1,240*43	1617.01	26.84		104.1
7	952.98	○-1,270*41	1583.02	28.98		66.1
6	1,047.59	○-1,300*41	1621.66	29.71		54.8
5	1,219.18	○-1,320*42	1686.28	29.43		38.3
4	1,200.05	○-1,335*41	1666.74	30.56		38.9
3	1,342.55	○-1,360*42	1739.06	30.38		29.5
2	1,441.29	○-1,380*43	1806.13	30.09		25.3
1	1,613.86	○-1,400*44	1874.40	29.82		16.1

(c) Model of the twist angle of 2 degrees

Tier	Required cross-sectional area (cm^2)	Section (○-external diameter*thickness)	Designed cross-sectional area (cm^2)	Width-to-thickness ratio		Strength increase (%)
				Design	Limitation	
15	111.29	○-730*25	553.71	27.20	27.75	397.5
14	182.98	○-1060*36	1,158.12	27.44		532.9
13	388.11	○-1090*48	1,571.30	20.71	30.58	304.9
12	512.24	○-1,090*48	1,571.30	20.71		206.7

(c) Continued

Tier	Required cross-sectional area (cm^2)	Section (○-external diameter*thickness)	Designed cross-sectional area (cm^2)	Width-to-thickness ratio		Strength increase (%)
				Design	Limitation	
11	564.26	○-1,090*37	1,224.00	27.46	27.75	116.9
10	715.60	○-1,120*38	1,291.70	27.47		80.5
9	846.68	○-1,150*39	1,361.22	27.49	30.58	60.8
8	991.34	○-1,180*40	1,432.57	27.50		44.5
7	1,256.58	○-1,220*44	1,625.59	25.73		29.4
6	1,414.65	○-1,240*41	1,544.38	28.24		9.2
5	1,629.80	○-1,330*42	1,699.48	29.67		4.3
4	1,763.49	○-1,400*43	1,833.15	30.56		4.0
3	1,964.40	○-1,460*45	2,000.41	30.44		1.8
2	2,163.29	○-1,530*48	2,234.80	29.88		3.3
1	2,314.40	○-1,580*50	2,403.32	29.60		3.8

Table 3 Member sizes of the gravity columns in the prototype buildings

Tier	Section (□-width*height*thickness)		
	Regular model	Model with the twist angle of 1 degree	Model with the twist angle of 2 degrees
15	□-215*215*12	□-205*205*12	□-425*425*25
14	□-285*285*16	□-275*275*16	□-425*425*25
13	□-340*340*20	□-325*325*20	□-450*450*28
12	□-395*395*22	□-375*375*22	□-485*485*28
11	□-435*435*25	□-425*425*24	□-520*520*30
10	□-475*475*27	□-455*455*26	□-570*570*32
9	□-520*520*29	□-485*485*27	□-630*630*36
8	□-550*550*31	□-510*510*29	□-645*645*37
7	□-580*580*33	□-535*535*31	□-660*660*38
6	□-605*605*35	□-560*560*33	□-700*700*40
5	□-645*645*36	□-580*580*34	□-730*730*40
4	□-670*670*38	□-605*605*35	□-805*805*43
3	□-735*735*41	□-650*650*35	□-850*850*47
2	□-770*770*42	□-665*665*37	□-875*875*48
1	□-805*805*43	□-715*715*40	□-1,030*1,030*56

structures and a mitigated seismic width-to-thickness ratio limit compared to a square tube section. Also, Steel built-up square tube sections were used for the interior gravity columns. Steel wide flange sections were used for all girders and beams (see Table 2). In addition, all connections were assumed as simple connections.

Table 4 Member sizes of the girders and the beams in the prototype buildings

Member	Section (H-height*width*thickness of web*thickness of flange)		
	Regular model	Model with the twist angle of 1 degree	Model with the twist angle of 2 degrees
G_1	H-488*300*11*18	H-594*302*14*23	W40*133
G_2	H-440*300*11*18	H-414*405*18*28	W36*720
B_1	H-300*300*10*15		W24*192
B_2	H-350*150*9*15	H-386*299*9*14	H-434*299*10*15
B_3	H-300*150*6.5*9	H-300*150*6.5*9	H-300*150*6.5*9

Table 5 Check of the roof displacement of the prototype buildings

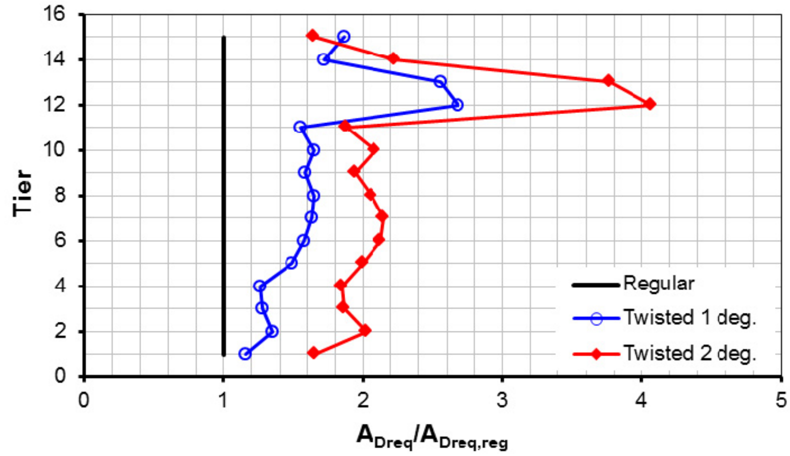
Model	Roof displacement	Limitation (cm)
Regular model	46.86 (H/499.4)	
Model with the twist angle of 1 degree	46.81 (H/499.9)	46.80 (H/500)
Model with the twist angle of 2 degrees	46.74 (H/500.7)	

The nominal yield strength (F_y) of steel used in all members is 325 MPa (for plate thickness equal to or less than 40 mm), or 295 MPa (for plate thickness more than 40 mm but less than 100 mm). An elastic modulus of steel is 2.05×10^5 MPa.

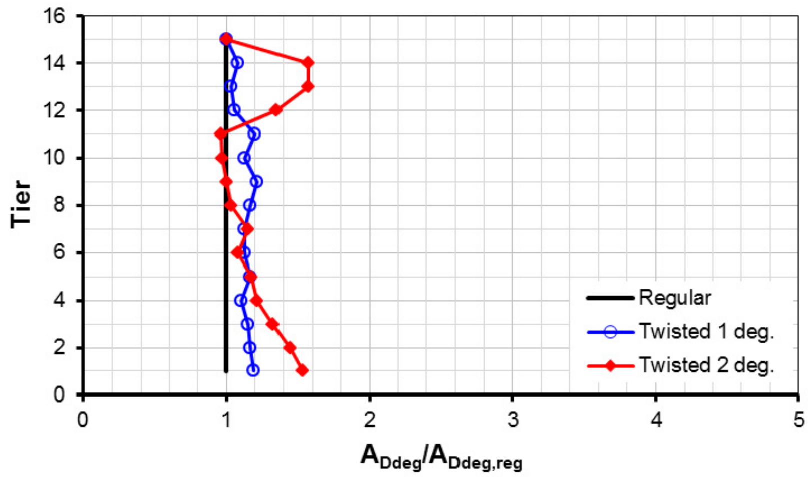
The maximum roof displacement, which is the serviceability requirement for wind design, is generally targeted within the range of H/400 to H/600. The prototype buildings were designed to satisfy roof displacement acceptance criteria (= H/500), based on the recommendations of NBCC (2005) (see Table 5). The designed cross-sectional areas of the main structural members were greatly increased compared to the required cross-sectional areas to satisfy the roof displacement requirement. The increases of the cross-sectional areas of the main structural members, especially in the upper part of buildings, occurred in both regular and twisted buildings (see the strength increase in Table 2). These large redundancies indicate that the twisted high-rise steel diagrid buildings may behave elastically under moderate or weak earthquake.

3. Quantity analysis of twisted high-rise steel diagrid frames

In this study, the cross-sectional area of the diagrid members represents steel quantity. And the quantity ratio was defined by the ratio of the cross-sectional areas of the diagrid members of each prototype building to the cross-sectional areas of the diagrid members of regular prototype building. Then, the required quantity ratio represents the required cross-sectional areas of the diagrid members of each prototype building (A_{Dreq}) normalized by them of the regular prototype building ($A_{Dreq,\,reg}$). The designed quantity ratio represents the designed cross-sectional areas of the of the diagrid members of each prototype building (A_{Ddeg}) normalized by them of the regular prototype building ($A_{Ddeg,\,reg}$). Here, the required cross-sectional areas of the diagrid members are the cross-sectional areas needed in order to resist the gravity load and the wind load. The designed cross-sectional areas of the diagrid members are the cross-sectional areas finally determined in



(a) The quantities of the required diagrid members



(b) The quantities of the designed diagrid members

Fig. 2 Comparison of the quantity of diagrid members in the prototype buildings

order to additionally satisfy the roof displacement limit and the seismic compact section criteria.

Fig. 2 compares the quantity variance of the diagrid members in tiers, in accordance with alteration of the twist angle. Fig. 2(a) shows the comparisons of the required quantity ratios and Fig. 2(b) shows the comparisons of the designed quantity ratios.

As shown in Fig. 2(a), when the twist angle of the building is 1 degree, the required steel quantity ratio is about 1.5, on average, in the lower- and middle-tiers, and increases up to2.7 in the upper-tiers. If the twist angle of the building is 2 degrees, the required steel quantity is about 2 times greater than that of the building with the twist angle of 1degree.As a result, the total required quantity ratios for all diagrid members in each prototype building are 1:1.44:1.98 in order of the buildings with the twist angle of 0, 1, and 2 degrees.

On the other hand, the designed steel quantity does not increase proportionally with the

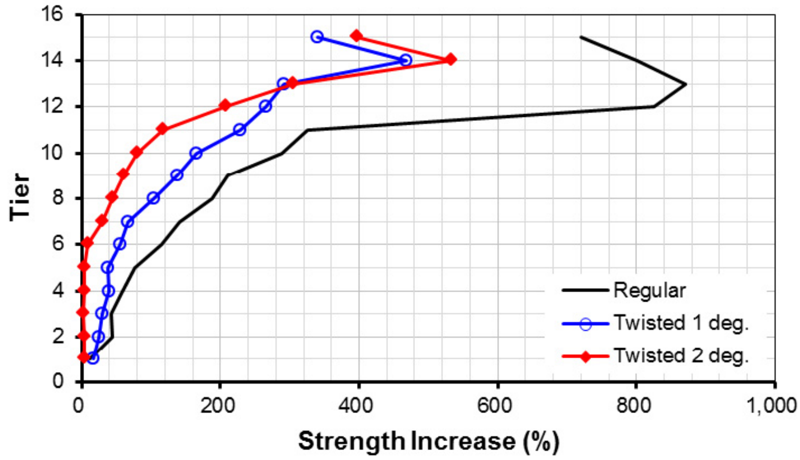


Fig. 3 Comparison of strength increase ratios of diagrid members

increase of the twist angle. As shown in Fig. 2(b), when the building is twisted 1 degree, the designed steel quantity ratio is about 1.2 in the lower- and middle-tiers, and less than 1.1 in the upper-tiers. However, when the twist angle of the building is 2 degrees, the designed steel quantity ratio reaches about 1.5 in the lower-floor tiers, decreases to 1.0 in the middle-floor tiers, and increases again up to about 1.6 in the upper-floor tiers. The total designed quantity ratios for all diagrid members in each prototype building are 1:1.14:1.22 in order of the buildings with the twist angle of 0, 1, and 2 degrees.

Fig. 3 shows the comparisons of the strength increase (the ratio of the designed cross-sectional area to the required cross-sectional area) of the diagrid members of each prototype building. As demonstrated by the previous studies (Lee and Kim 2007, Kim and Lee 2013), the strength increase of the regular building becomes dramatically larger in the upper-floor tiers due to the roof displacement limit. This tendency also appears in twisted irregular high-rise buildings. However, it should be noted that the amount of the strength increase of the diagrid members decreases as the twist angle of the building increases. The reason is that the required quantity becomes very larger, but the designed quantity becomes a little larger as the twist angle of the building increases. This finding represent that a final wind-designed twisted steel high-rise diagrid frame building will have the smaller redundancy as the twist angle becomes larger.

4. Seismic performance evaluation of twisted high-rise steel diagrid frames

In this section, the seismic performance of the wind-designed prototype buildings was evaluated to check the possibility of the elastic response of the diagrid members under seismic loading. The recent seismic design guidelines of high-rise buildings (CTBUH 2008, LATBSDC 2008, PEER 2010) have adopted two basic earthquake hazard levels: the service level earthquake (SLE) corresponding to ground motion with 43-years return period and the maximum considered earthquake (MCE) corresponding to ground motion with 2475-years return period. In Fig. 4, the 2-percent damped elastic seismic design spectra was developed for two ground motions. Effective peak ground accelerationsare0.072 g and 0.359 g, respectively, for SLE and MCE hazard levels in

Seoul, South Korea. Stiff soil (site class D) and importance factor of 1.2 was assumed (AIK 2009).

4.1 Screening using wind load base shears

The fundamental periods of the prototype buildings were obtained from the three-dimensional eigen value analysis results using MIDAS-Gen structural analysis program (2010). Also, the wind base shear coefficient, which is the base shear by factored wind load divided by the total weight of the building, was calculated for each prototype building.

Then, the elastic seismic design spectrums and the wind base shear coefficients were plotted together in Fig. 4. This comparison easily shows the possibility of elastic seismic design of the wind-designed twisted high-rise steel diagrid frame buildings. It is simply predictable that all prototype buildings can elastically resist within the SLE hazard level, but cannot elastically resist within the MCE hazard level. Therefore, it can be said that the detailed seismic performance evaluation of the buildings is required to estimate the potentials of elastic seismic design of the buildings in the MCE hazard level.

4.2 Seismic performance evaluation by response spectrum analysis

Response spectrum analyses corresponding to both SLE and MCE hazard level were conducted. First, a 2-percent damping ratio was used based on the seismic design recommendations of high-rise buildings (CTBUH 2008). Second, in order to consider the concurrent multidirectional seismic effects, a ground motion ratio of 100:100 in two orthogonal horizontal directions was applied according to the seismic design guidelines of high-rise buildings (LATBSDC 2008, PEER 2010). These two conditions are different from those used for the moderate- or low-rise buildings (ASCE 2010).

ASCE 41-06 (ASCE 2007) recommends procedures to assess the seismic performance of a building in the member level. It classifies structural members as the force-controlled action members and the deformation-controlled action members. The force-controlled action member implies a brittle member that does not have inelastic deformation capacity and the deformation-

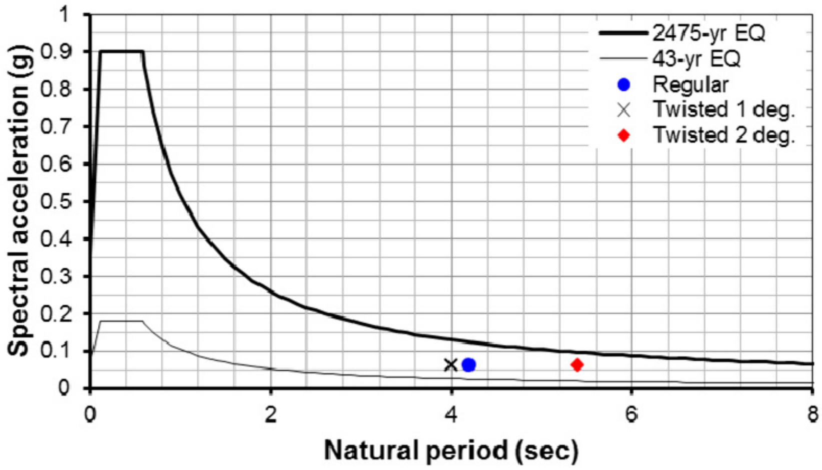
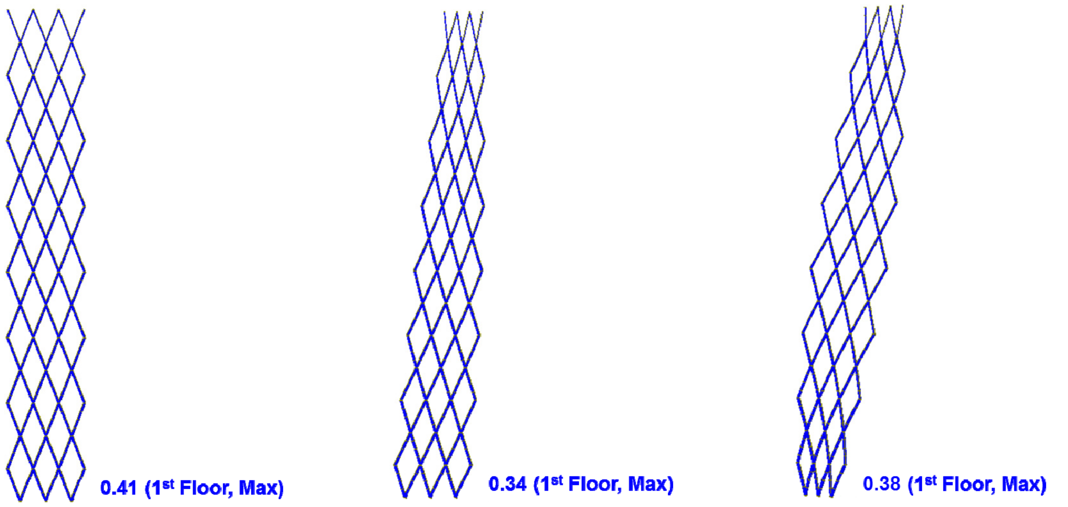


Fig. 4 Comparison of wind design base shear with elastic seismic design spectrum

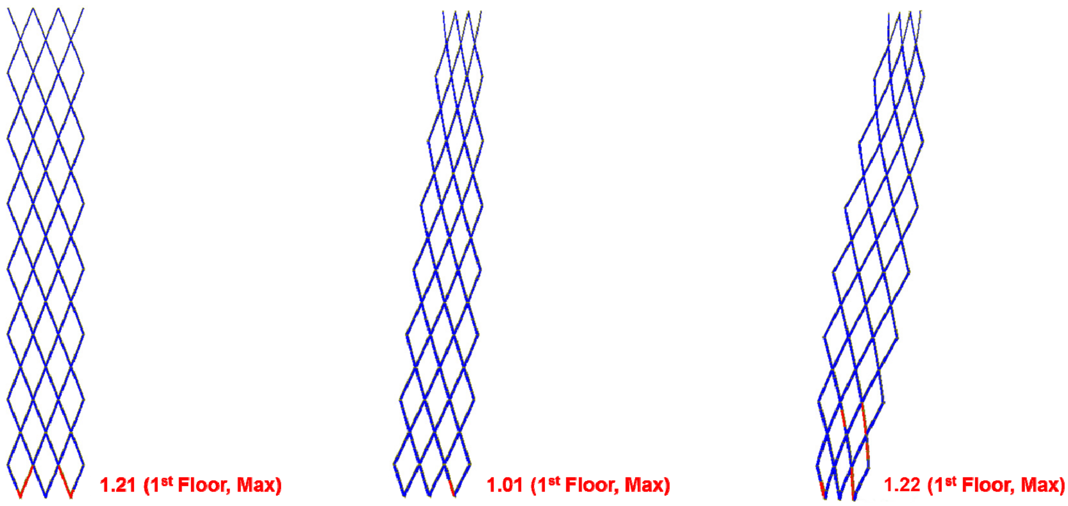
Table 6 Seismic performance evaluation criteria of steel diagonal member (ASCE 41-06)

Circular steel tube	m-factor acceptance criteria		
	IO level	LS level	CP level
$Kl/r \leq 2.1\sqrt{(E/F_y)}$	1.25	4	6
$Kl/r \geq 4.2\sqrt{(E/F_y)}$	1.25	5	7
$2.1\sqrt{((E/F_y)} < Kl/r < 4.2\sqrt{(E/F_y)}$	Linear interpolation between the values should be used.		



A. Regular-shape B. Twist angle of 1 degree C. Twist angle of 2 degrees

(a) Service level earthquake



A. Regular-shape B. Twist angle of 1 degree C. Twist angle of 2 degrees

(b) Maximum considered earthquake

Fig. 5 Distribution of the DCR value obtained from response spectrum analysis controlled action member implies a ductile member that has inelastic deformation capacity. The diagrid members can be classified as deformation-controlled action members.

Also, based on linear analysis results, an index of *m*-factor is used to evaluate the seismic performance of a building in the member level. It is defined as the demand-to-capacity ratio (DCR) amplified by a knowledge factor which reflects the objective level of seismic rehabilitation, and the uncertainty of material strength. In this study, the *m*-factor value of diagrid members is identical to the DCR value because no amplification by a knowledge factor was assumed.

The seismic design guidelines of high-rise buildings (CTBUH 2008, LATBSDC 2008, PEER 2010) regulate to satisfy the elastic response at an SLE hazard level. Then, the elastic or inelastic behavior of the structural members may be easily determined by the DCR. If the DCR value of a structural member is less than or equal to 1.0, the structural member remains elastic. If it exceeds 1.0, a brittle member fails and a ductile member undergoes inelastic deformation.

The seismic design guidelines of high-rise buildings (CTBUH 2008, LATBSDC 2008, PEER 2010) also regulate to satisfy Collapse Prevention building performance level at an MCE hazard level. Table 6 shows the seismic performance evaluation criteria for diagrid members to satisfy a target performance level; Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) (ASCE 2007). That is, the calculated *m*-factor (or the DCR in this study) should be less than the acceptance criterion in the target performance level. The calculated DCR of diagrid members should be less than the appropriate values in Table 6 to satisfy CP performance level.

For calculation of the DCR, the strength demand for diagrid members was obtained from the SRSS (square root of sum of squares) values resulting from the response spectrum analysis. The strength capacity of diagrid members in Table 2 was determined based on the strength equations for the compressive member per AISC-LRFD (AISC 2005) except for the strength reduction factor of 1.0 (ASCE 2007) and the expected yield strength of material (= $1.2*F_y$) (AIK 2009).

Fig. 5(a) shows the distribution of the DCR of the twisted high-rise steel diagrid frames resulting from the response spectrum analysis at the SLE hazard level. The DCR values and distribution patterns of three prototype buildings are not significantly different. The maximum DCR value of the diagrid members appeared in the lowest tier of each prototype building. In the SLE hazard level, all diagrid members in three prototype buildings are expected to remain elastic because their maximum DCR values are much less than 1.0.

On the other hand, Fig. 5(b) shows the distribution of the DCR of the diagrid frames at the MCE hazard level. Only several diagrid members in the lower tiers of each prototype building experienced a slight inelastic deformation because their maximum DCR values were slightly greater than 1.0. Moreover, they satisfied even Immediate Occupancy (IO) performance level because their maximum DCR values are less than the acceptance criterion of 1.25 as listed in Table 6. Also, based on the DCR distribution patterns, it is expected that the amount of the diagrid members showing inelastic behavior will increase rapidly if the twist angle of the building becomes more than 2 degrees.

5. Conclusions

This study analytically investigated the potential of elastic seismic design of high-rise steel diagrid frames with twisted irregularity in the regions of a strong wind low seismicity. The prototype buildings with twist angles of 0 (regular-shaped), 1, and 2 degrees were designed based on wind load design criteria. The quantity, redundancy, and seismic performance of their diagrid members were evaluated. The summarized results are as follows:

- It was found that the significant system redundancy (over-strength) was brought into the all the regular-shape and twisted buildings during the wind design procedure to satisfy roof displacement requirement. This system redundancy is a potential factor to increase the possibility of elastic seismic design. However, as the twist angle of the building grew, the quantity of the diagrid members required only to resist gravity and wind loads increased more rapidly than the designed quantity of the diagrid members designed to satisfy roof displacement requirement. Therefore, increasing the twist angle of building decreased the system redundancy.
- Regardless of the various twisted irregularity, the prototype buildings all showed the similar values and distribution pattern of demand-to-capacity ratio in their diagrid members. Also, they all behaved elastically at the service level earthquake (SLE) hazard level, and a few members in the lower floors of the buildings showed a little plastification, which still made the building within Immediate Occupancy performance level, at the maximum considered earthquake (MCE) hazard level. It is expected that a little increase of quantity of the diagrid member makes it possible to show the elastic response under MCE.
- To summarize, the elastic seismic design of the irregular high-rise steel diagrid frame buildings with a twist angle of 2 degrees or smaller can be suggested against MCE seismic hazard level as well as the SLE seismic hazard level. The elastic seismic response of the diagrid members will make the full strength design of their connections available and the connection rupture may not be considered.

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