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INELASTIC BEHAVIOR OF STAGGERED TRUSS SYSTEMS

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SUMMARY

In this study 4-, 10-, and 30-story staggered truss systems (STS) were designed and their seismic performances were evaluated by pushover analysis. The results were compared with the seismic performance of conventional moment-resisting frames and braced frames. According to the analysis results, the STS showed relatively satisfactory lateral load-resisting capability compared with conventional braced frames. However, in the mid- to high-rise STS, plastic hinges formed at horizontal and vertical chords of a Vierendeel panel, which subsequently led to brittle collapse of the structure. Based on these observations, reinforcing schemes were applied and their effects on enhancing lateral load-resisting capacity were investigated. Copyright © 2007 John Wiley & Sons, Ltd.

1. INTRODUCTION

Staggered truss systems (STS) were developed to achieve a more efficient structural frame to resist wind loads and a versatility of floor layout. The STS is known to be appropriate for use in such buildings as apartments, condominiums, dormitories, and hotels (Taranath, 1998). The STS has the advantage that large clear-span open areas are possible at the first-floor level, because columns are located only on the exterior faces of the building. Other benefits include minimum deflection and greater stiffness in the structure while reducing seismic loads and foundation costs (Scalzi, 1971). It was also reported that the structural costs per unit building area, on a relative basis, turned out to be lowest in STS (Cohen, 1986).

Recently STS have been successfully applied to many large-scale building projects and their efficiency and economy have been reported (Brazil, 2000; Mcknamara, 1999; Pollak and Gustafson, 2004). The STS, however, has not been considered as one of the basic seismic-force-resisting systems in most design codes, which implies that further research is still necessary for the system to be accepted as a standard structure system. FEMA-450 (BSSC, 2002) requires that seismic-force-resisting systems that are not listed as the basic seismic-force-resisting systems shall be permitted if analytical and test data are submitted to demonstrate the lateral force resistance and energy dissipation capacity. To facilitate the application of the STS, the AISC (American Institute of Steel Construction) published Design Guide 14: Staggered Truss System Framing Systems (AISC, 2002), in which some recommendations and examples for structural design are provided. The design guide recommends using 3·0 for response modification factor for seismic design; however, other seismic behavior factors, such as overstrength and ductility factors, to define inelastic behavior of the structure are not specified.

In this study, 4-, 10-, and 30-story structures were designed with various structure systems, such as a moment frame, braced frame, and an STS, and their seismic behavior was compared through

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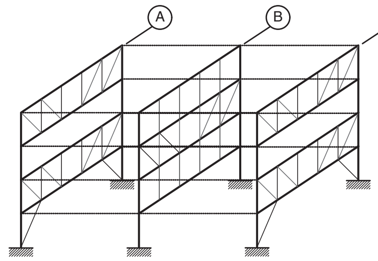

Figure 1. Schematic of STS

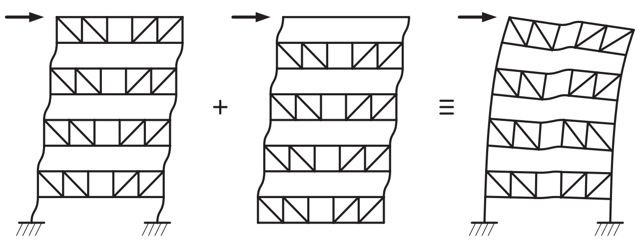

Figure 2. Behavior of an STS subjected to lateral load

nonlinear static analysis. Based on the results of the analysis, seismic reinforcing schemes were derived and their effects on enhancing lateral load-resisting capacity were evaluated.

2. STAGGERED TRUSS SYSTEMS

In an STS, shown in Figure 1, the story-high trusses that span the width of the building are located in a staggered pattern, and the trusses are generally concealed inside partition walls with Vierendeel openings for corridors; the trusses on the second floor extend across the building at column lines A and C, and are located at column line B on the third floor. Columns have minimum bending moments due to gravity and lateral loads, because of the cantilever action of the double-planar system of framing (Figure 2). With the columns only on the exterior walls of the building, the usual interior columns are omitted, thus providing a full width of column-free area on the first floor. Columns are oriented with their strong axis resisting lateral forces in the longitudinal direction of the building (AISC, 2002). As the total frame is acting as a stiff truss with direct axial loads acting in most structural members, drift is minimized. However, secondary bending occurs at the chords in the Vierendeel panels of the trusses, which may become a weak link of the system. The staggered arrangement of the floor-deep trusses placed at alternate levels on adjacent column lines allows an interior floor space of twice the column spacing to be available for freedom of floor arrangements. The floor system spans from the top chord of one truss to the bottom chord of the adjacent truss, serving as a diaphragm transferring the lateral shears from one column line to another. This enables the structure to perform as a single braced frame, even though the trusses lie in two parallel planes.

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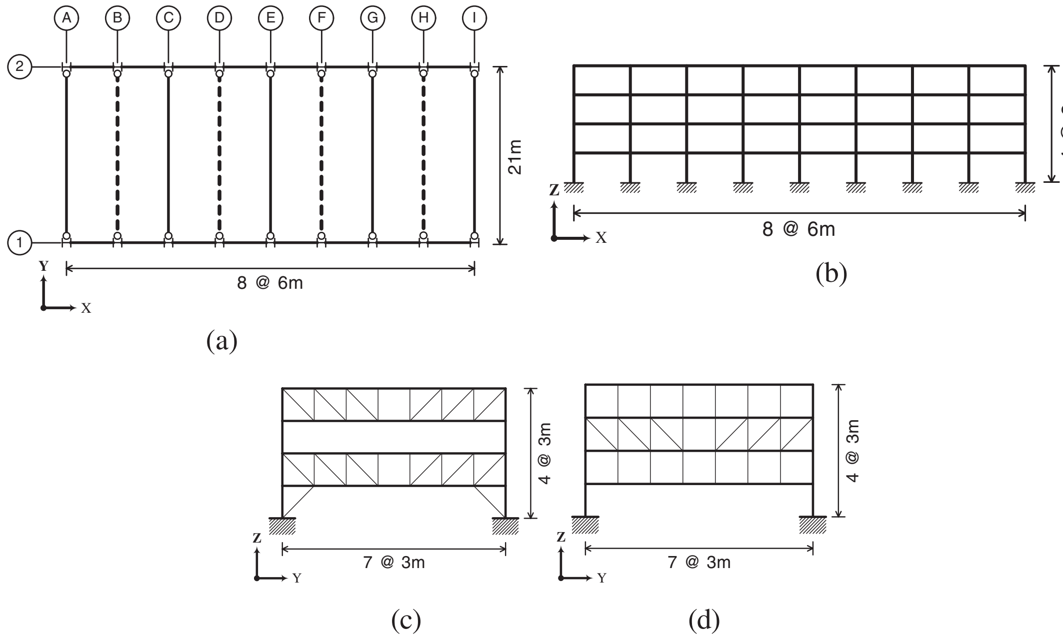

Figure 3. Structural shape of the 4-story STS: (a) plan; (b) side view (longitudinal direction); (c) side view (row A); (d) side view (row B)

3. DESIGN OF ANALYSIS MODEL STRUCTURES

The 4-, 10-, and 30-story structures with staggered trusses were designed to investigate the seismic-load-resisting capacity. The staggered trusses were located along the transverse direction, and the moment-resisting frames were placed along the longitudinal direction. Figure 3 shows the plan and side view of the 4-story STS. To compare the behavior of the STS with that of conventional systems, concentric braced frames (BF) and moment-resisting frames (MRF) were also designed for the same loading condition. In the conventional systems two internal columns were inserted along the column lines in the transverse direction (Figure 4). The 4- and 10-story BF were designed as the building frame system, in which braces resist most of the lateral load, whereas the 30-story BF was designed as a dual system, in which 25% of the lateral seismic load is resisted by moment frames. The design loads for the model structures were determined based on the FEMA-450 and the Korean Building Code (AIK, 2005). Structural member design was carried out based on the load and resistance factor design (LRFD) of AISC (1993). The dead load of 5·9 kN/m^2 and live load of 2·5 kN/m^2 were used as gravity load. The coefficients required for the design parameters for seismic and wind loads are presented in Tables 1 and 2, respectively. Along the transverse direction, where staggered trusses are located, a response modification factor of 3·0 was applied in the seismic design; along the longitudinal direction, where MRF were applied, a response modification factor of 6·0 was used. For BF, a response modification factor of 5·0 was applied in the design process. In all model structures, columns were designed with SM490 steel (F_y = 32·4 kN/cm^2) and other members were made of SS400 (F_y = 23·5 kN/cm^2).

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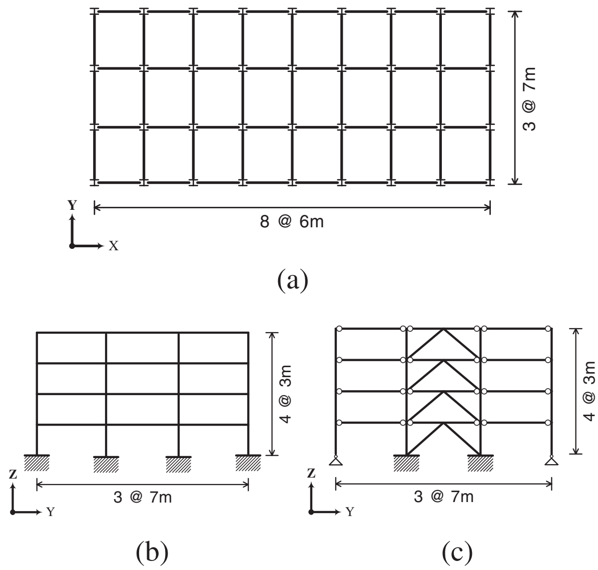

Figure 4. Structural shape of the 4-story MRF and the BF: (a) plan view of MRF and BF; (b) side view of MRF; (c) side view of BF

Table 1. Design parameters for seismic load

Peak ground acceleration	0·11	
Soil type	S_D	
Importance factor	1·5	30-story
	1·2	4-story, 10-story
Response modification factor	3	STS
	6	MRF
	5	BF

Table 2. Design parameters for wind load

Exposure	B
Basic wind speed	30
Importance factor	1
Gust factor	2·2

4. RESULTS OF PUSHOVER ANALYSIS

4.1 *Four-story structures*

Nonlinear static pushover analyses were carried out to investigate the seismic performance of the model structures. Incremental lateral load proportional to the code-specified equivalent static seismic lateral load was enforced to the model structures along the transverse direction, and the load–

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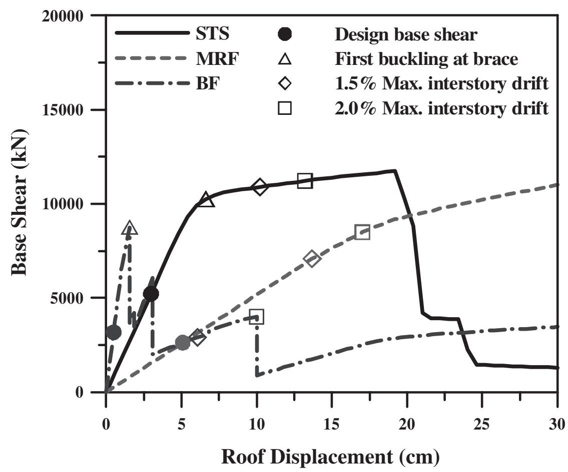

Figure 5. Pushover curves of the 4-story model structures

displacement relationship was observed. The general-purpose finite element analysis program MIDAS Genw (MIDAS, 2005) was used in the analysis.

Figure 5 shows the load–displacement relationship of the STS, MRF, and BF, where the points of the following quantities and events are marked: design base shear, first yield of a bracing member, and maximum interstory drifts reaching 1·5% and 2·0% of the story height. It can be observed that the strength of the BF drops rapidly right after the maximum strength is reached, due mainly to the formation of plastic hinges in the middle of the girders in the braced bays. The moment frame, as it was designed with the largest response modification factor, has the smallest stiffness and strength; however, it shows the best ductile behavior. The STS shows high strength and enough ductility to remain stable until the maximum interstory drift well exceeds 2·0% of the story height.

The interstory drifts of the model structures are plotted in Figure 6. Figure 6(a) shows the interstory drifts of the structures when the maximum drift reached 1·5% of the story height. It can be observed that in most of the structures the interstory drift is concentrated mostly at the lower stories. This can be confirmed in Figure 6(b), which shows that the interstory drifts of the STS increase only in the lower two stories, while the maximum interstory drift increases from 1·5% to 2·0% of the story height.

Figure 7 depicts the idealized nonlinear load–displacement relationship of a structural member presented in FEMA-450 with each seismic performance level (Immediate Occupancy, Life Safety, and Collapse Prevention) marked on it. FEMA 450 recommends 15 times the yield displacement and 9 times the buckling displacement as the limit states for tensile and compressive braces, respectively. Figure 8 presents the locations of inelastic deformation (buckling in compressive braces) in the STS when the maximum interstory drift reached 1·5% of the story height. It can be observed that most inelastic deformation occurred in members around Vierendeel panels; especially large plastic deformation occurred in the vertical member of the Vierendeel panel in row A. Figure 9 shows the plastic hinge formation immediately before collapse of the structure, in which it can be noticed that the horizontal members in the Vierendeel panel in row C reached the limit state. The results imply that the Vierendeel panel in STS plays an important role in the nonlinear behavior of the system.

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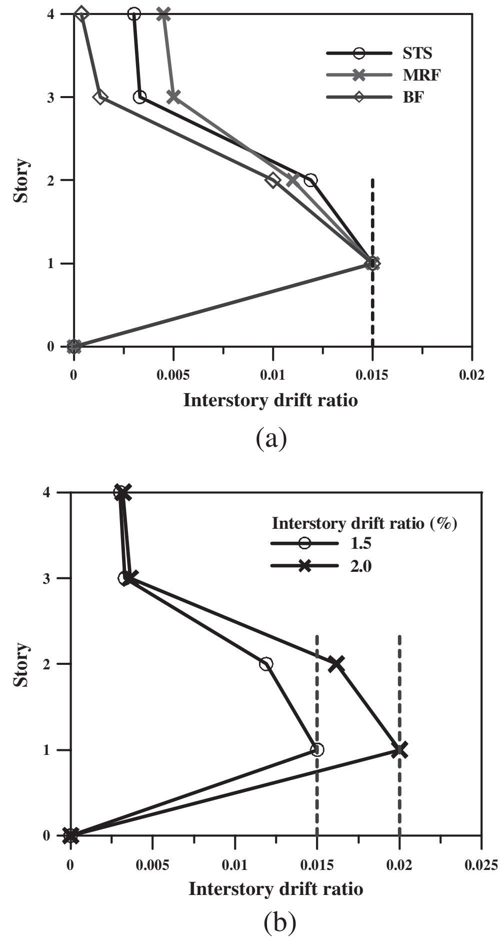

Figure 6. Interstory drift of the 4-story model structures: (a) interstory drifts at the maximum drift of 1·5%; (b) interstory drifts of the STS at the maximum interstory drift of 1·5% and 2·0%

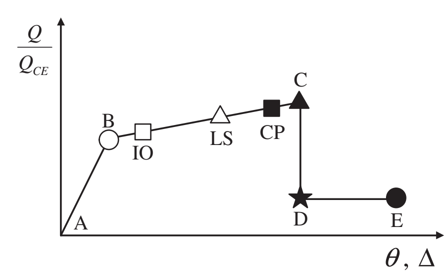

Figure 7. Damage state of plastic deformation

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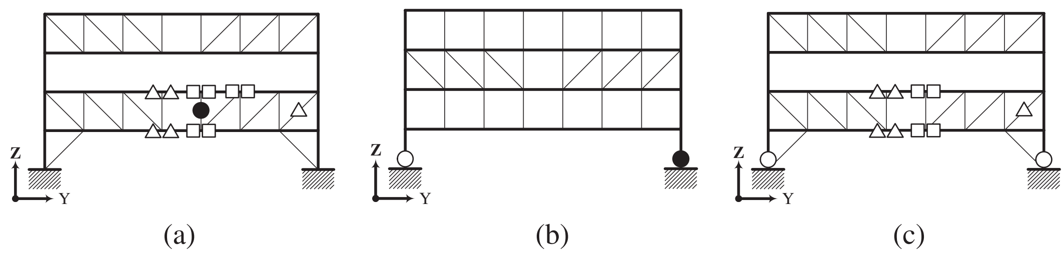

Figure 8. Location of inelastic deformation at the maximum interstory drift ratio of 1·5%: (a) row A; (b) row B; (c) row C

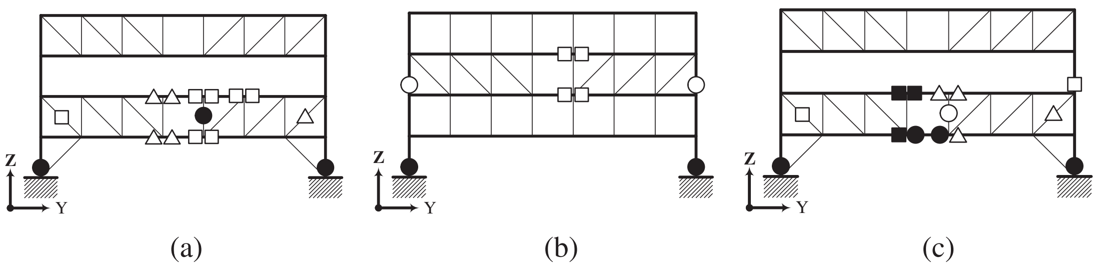

Figure 9. Location of inelastic deformation at near collapse: (a) row A; (b) row B; (c) row C

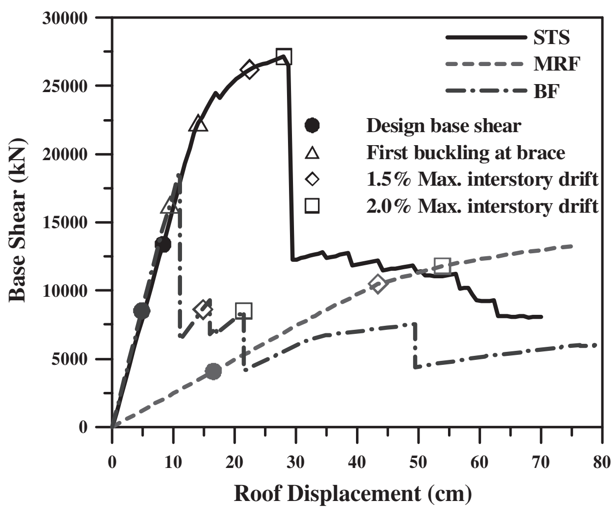

Figure 10. Pushover curves of the 10-story model structures

4.2 Ten-story model structures

Figure 10 shows the pushover curves of the 10-story model structures. Compared with the results of the 4-story structures, the MRF and the CBF show similar results, while the STS shows somewhat brittle behavior right after the maximum strength is reached. Figure 11(a) plots the interstory drifts of

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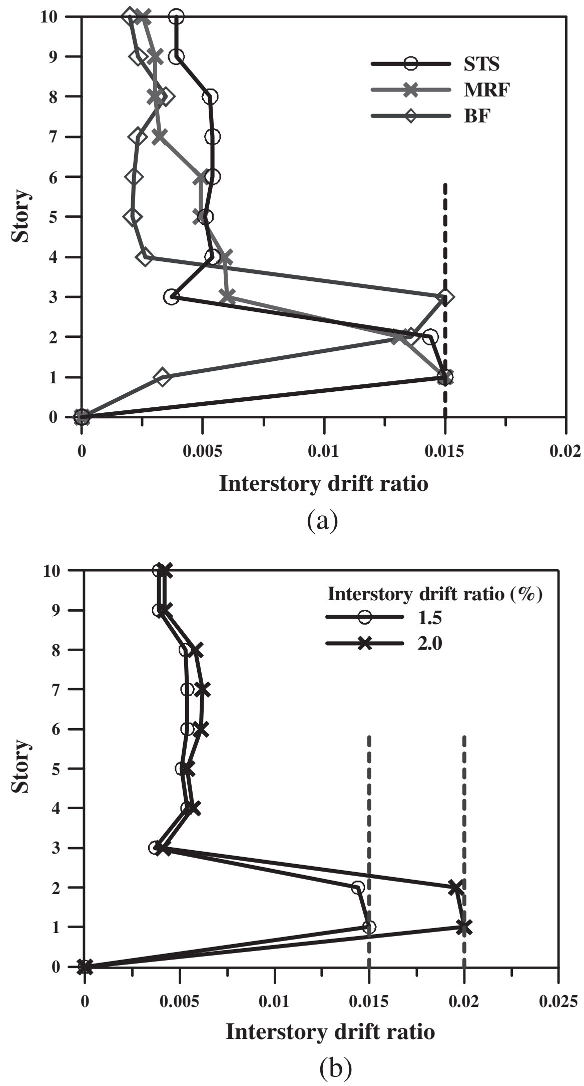

Figure 11. Interstory drift of the 10-story model structures: (a) inter-story drifts at the maximum drift of 1·5%; (b) interstory drifts of the STS at the maximum drift of 1·5% and 2·0%

the structures, where it can be observed that in all model structures the interstory drifts are concentrated in the lower few stories. Figure 11(b) shows that the interstory drifts of the higher stories in STS do not change while the maximum interstory drift in the second story increased from 1·5% to 2·0% of the story height.

Figures 12 and 13 show the plastic hinge formation or buckling of compression braces of the 10-story structure when the maximum interstory drift reached 1·5% of the story height and immediately before collapse, respectively. As in the case of the 4-story structure, many plastic hinges first formed in members located near the second story Vierendeel panels (Figure 12). At the state of near collapse, the plastic hinges are distributed to the Vierendeel panels in the higher stories (Figure 13). However,

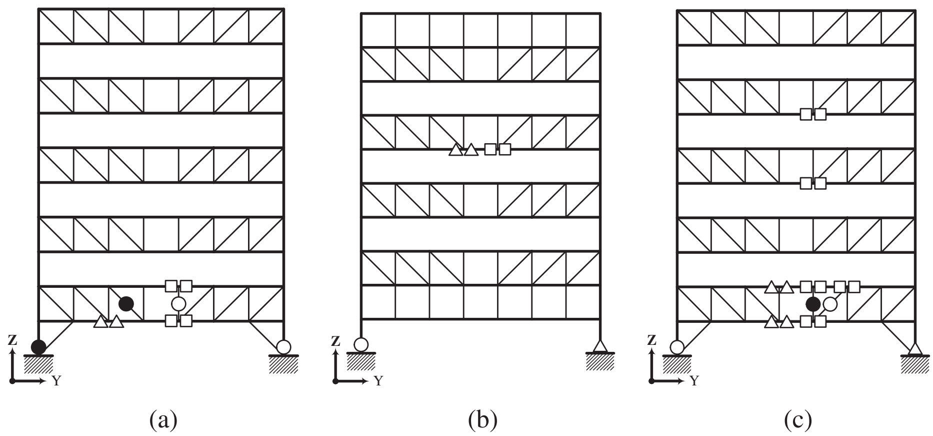

Figure 12. Location of inelastic deformation at the maximum interstory drift ratio of 1·5%: (a) row A; (b) row B; (c) row C

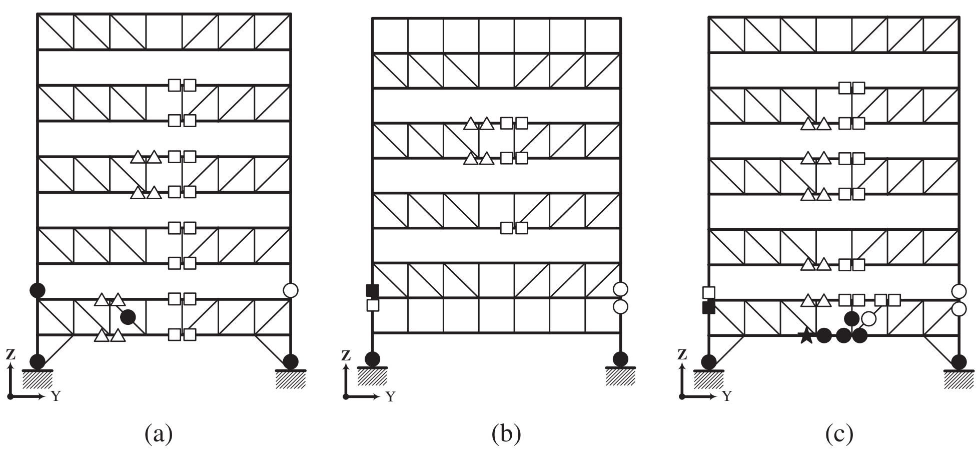

Figure 13. Location of inelastic deformation at near collapse: (a) row A; (b) row B; (c) row C

collapse is caused by the plastic hinges formed at the second story Vierendeel panel in row C and at the exterior columns in the lower stories.

4.3 Thirty-story model structures

Figure 14 presents the pushover curves of the 30-story BF and the STS. The BF, designed as a dual system with a larger response modification factor, shows lower strength than STS. The STS, however, has little ductility, even smaller than the BF. Although the STS possesses enough ductility to remain stable up to the collapse prevention limit state of 2% maximum interstory drift, the failure mode is quite brittle compared to that of the BF. Figure 15(a) plots the interstory drift of the two structures at the maximum drift of 1·5% of the story height. It can be observed that in both structures large interstory drifts occurred only in a few stories. Compared with the BF, the interstory drifts of the STS are

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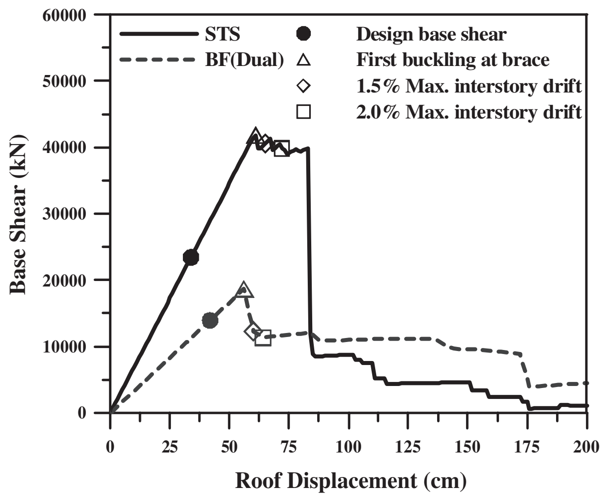

Figure 14. Pushover curves of the 30-story model structures

relatively more uniformly distributed. Figure 14(b) shows that while the maximum interstory drift increases from 1·5% to 2·0%, only the interstory drifts of the 8th to 11th stories increase, implying that inelastic damage is concentrated in those stories.

Figure 16 depicts the location of inelastic deformation, where it can be observed that most inelastic deformation occurred in the horizontal and the vertical members around the Vierendeel panels located in the 6th to 10th stories.

5. BEHAVIOR FACTORS OF THE STAGGERED TRUSS SYSTEM

Response modification factors are used in seismic design to induce nonlinear behavior of a structure for a design earthquake. The theoretical value for response modification factors can be computed from the results of nonlinear analysis. The ATC-19 (ATC, 1995) proposed a simplified procedure to estimate the response modification factors, in which the response modification factor, R, is calculated as the product of the three parameters that profoundly influence the seismic response of structures:

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

where R_o is the overstrength factor to account for the observation that the maximum lateral strength of a structure generally exceeds its design strength. R_μ is a ductility factor which is a measure of the global nonlinear response of a structure, and R_γ is a redundancy factor to quantify the improved reliability of seismic framing systems constructed with multiple lines of strength. In this study it is assumed that the redundancy factor is equal to 1·0. In this case the response modification factor is determined as the product of the overstrength factor and the ductility factor. Figure 17 represents the bilinear representation of the base shear versus roof displacement relation of a structure, which can be developed by a nonlinear static analysis. The ductility factor R_μ and the overstrength factor R_o are defined as follows:

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

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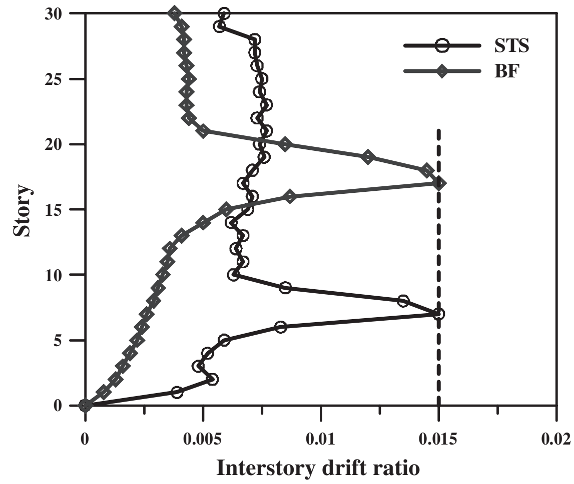
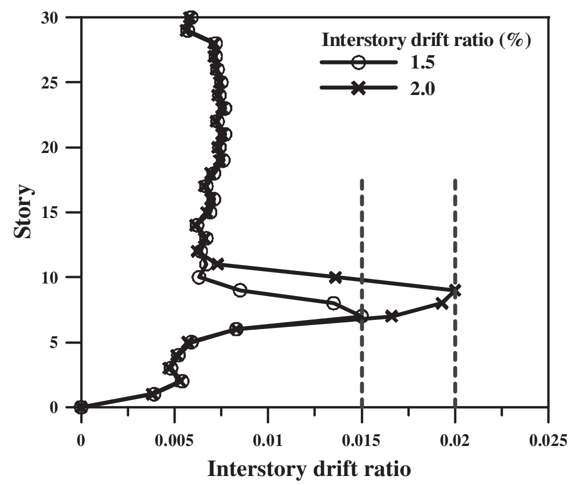


(a)



(b)

Figure 15. Interstory drift of the 30-story model structures: (a) interstory drifts at the maximum drift of 1·5%; (b) interstory drifts of the STS at the maximum drift of 1·5% and 2·0%

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

In this study the capacity envelopes obtained from pushover analysis were utilized to evaluate overstrength factors, R_o. To find the yield point, a straight line was drawn in such a way that the area under the original curve is equal to that of the idealized one as recommended in FEMA-356 (BSSC, 2000). The overstrength factor of STS is defined as 3·0 in IBC-2003 (ICC, 2003) and FEMA 450. Figure 18(a) depicts the overstrength factors of the model structures, which shows that the computed overstrength factors range from 1·7 to 1·9. These correspond only to 57–63% of the code-specified value. It can also be observed that as the height of the structure increases the overstrength factor decreases.

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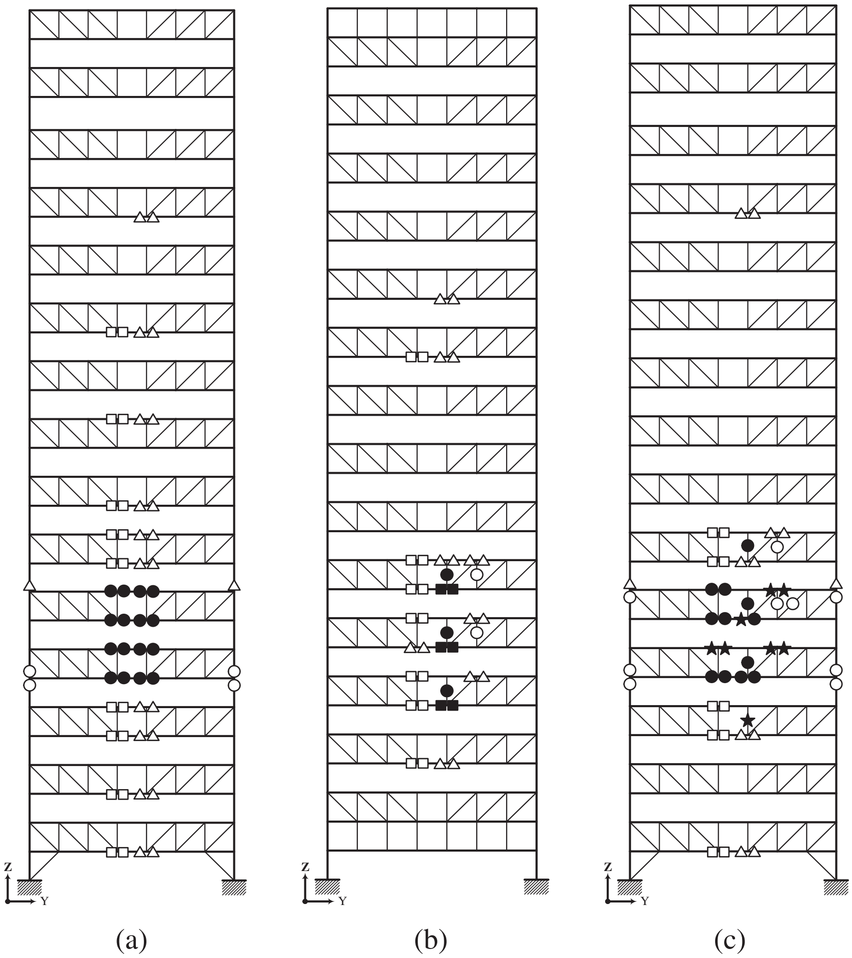

Figure 16. Location of inelastic deformation at near collapse: (a) row A; (b) row B; (c) row C

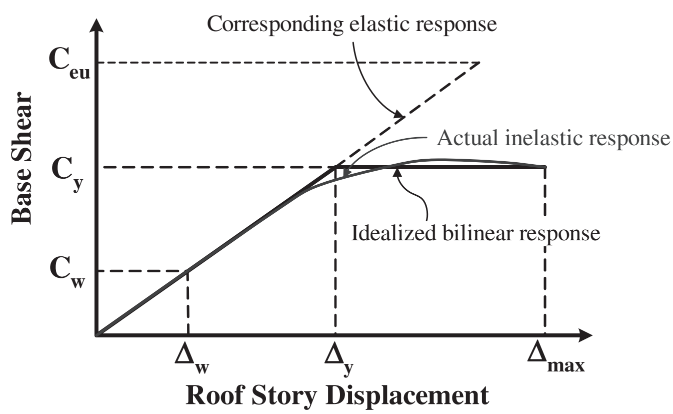

Figure 17. Idealized base shear–roof story displacement relationship of a structure

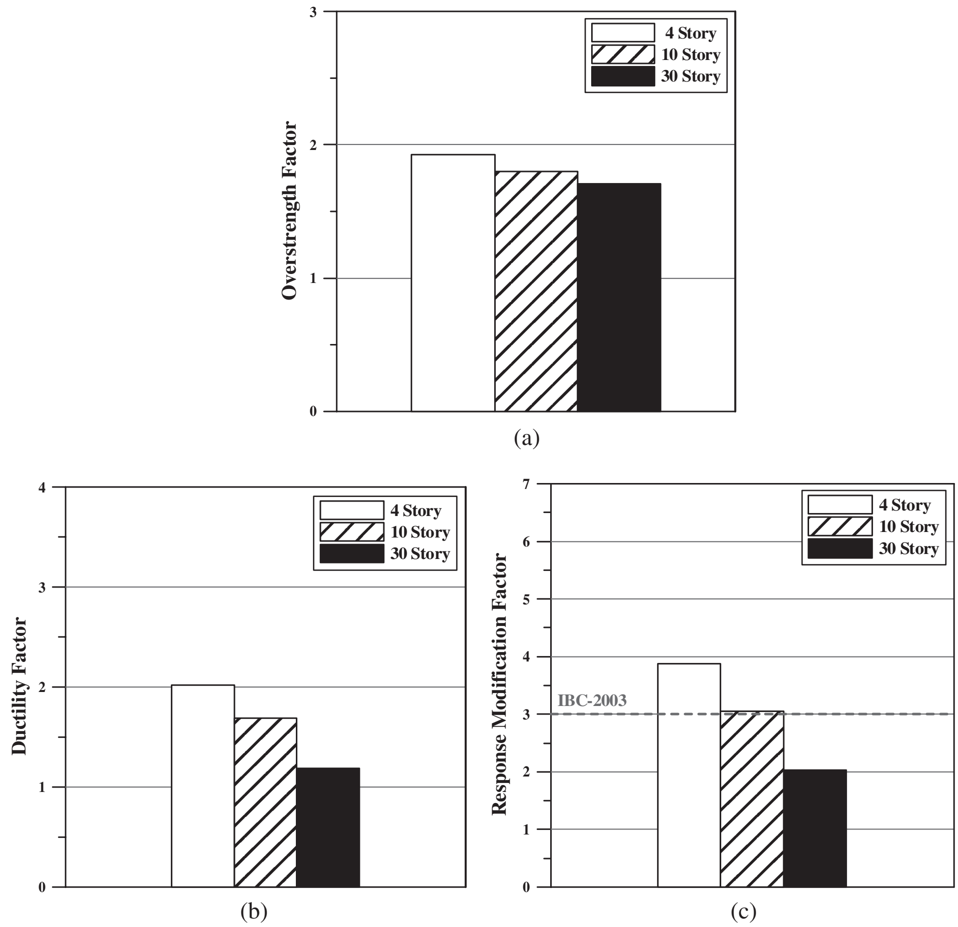

Figure 18. Behavior factors of the STS model structures: (a) overstrength factors; (b) ductility factors; (c) response modification factors

The ductility factor R_μ was computed using the system ductility factor μ and the procedure proposed by Newmark and Hall (1982):

$$
\begin{aligned}
&R_\mu = 1{\cdot}0 \quad (T < 0{\cdot}03\,\mathrm{s})\\
&R_\mu = \sqrt{2\mu - 1} \quad (0{\cdot}12 < T < 0{\cdot}03\,\mathrm{s})\\
&R_\mu = \mu \quad (T > 0{\cdot}12\,\mathrm{s})
\end{aligned}
\tag{3}
$$

where the ductility ratio μ is obtained by dividing the target displacement by the yield displacement. The target displacement used to obtain the ductility factor was the roof displacement when the maximum interstory drift reached 2% of the story height. Figure 18(b) presents the ductility factors of model structures; the ductility factor of the 4-story structure turned out to be slightly over 2·0; however, those of the higher structures were significantly less, implying brittle behavior of the structure.

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Table 3. Response modification factor and weight of structural steel used for model structures

(a) 4-story structures		
	R	Weight (kN)
STS	3	1134
MRF	6	1543
BF	5	1359

(b) 10-story structures		
	R	Weight (kN)
STS	3	4395
MRF	6	5109
BF	5	4799

(c) 30-story structures		
	R	Weight (kN)
STS	3	21,810
BF (dual system)	5	20,530

Figure 18(c) presents the response modification factors computed by multiplying overstrength factor and the ductility factor. It can be observed that the factor of the 4-story structure is larger than the code-specified value of 3·0, whereas it is smaller than 3·0 in the 30-story structure. As in the overstrength and the ductility factors, the computed response modification factor decreases as the number of stories increases.

6. WEIGHT OF STRUCTURAL STEEL

The amount of structural steel used in the design of model structures is compared in Table 3. In the 4- and the 10-story STS, even though they were designed with a smaller response modification factor and therefore with larger seismic load, the required amount of structural steel turned out to be less than that of the MRF and the BF. In the 30-story STS, however, a slightly larger amount of structural steel was required than the steel used to design the BF. Nevertheless, considering the inherent advantage of STS, such as higher strength and elimination of internal columns, the STS can be a potential alternative of conventional structure systems.

7. SEISMIC REINFORCEMENT OF STAGGERED TRUSS SYSTEMS

The analysis results presented above show that inelastic deformation occurs first in the horizontal and vertical members of Vierendeel panels and this leads to formation of a weak story and subsequently to total collapse. In this section some reinforcing schemes for a staggered truss were applied and their effects on the overall lateral load-resisting capacity were evaluated. As it turned out that the Vierendeel panels formed the weakest link of the system, the reinforcing schemes were focused on increasing stiffness and strength of the Vierendeel panels. Figure 19 shows the pushover curves of the STS with the moment of inertia of the horizontal members (upper and lower cords) increased by 30%, 50%, and 100%. It can be observed in the 4-story structure that the strength and stiffness of the reinforced

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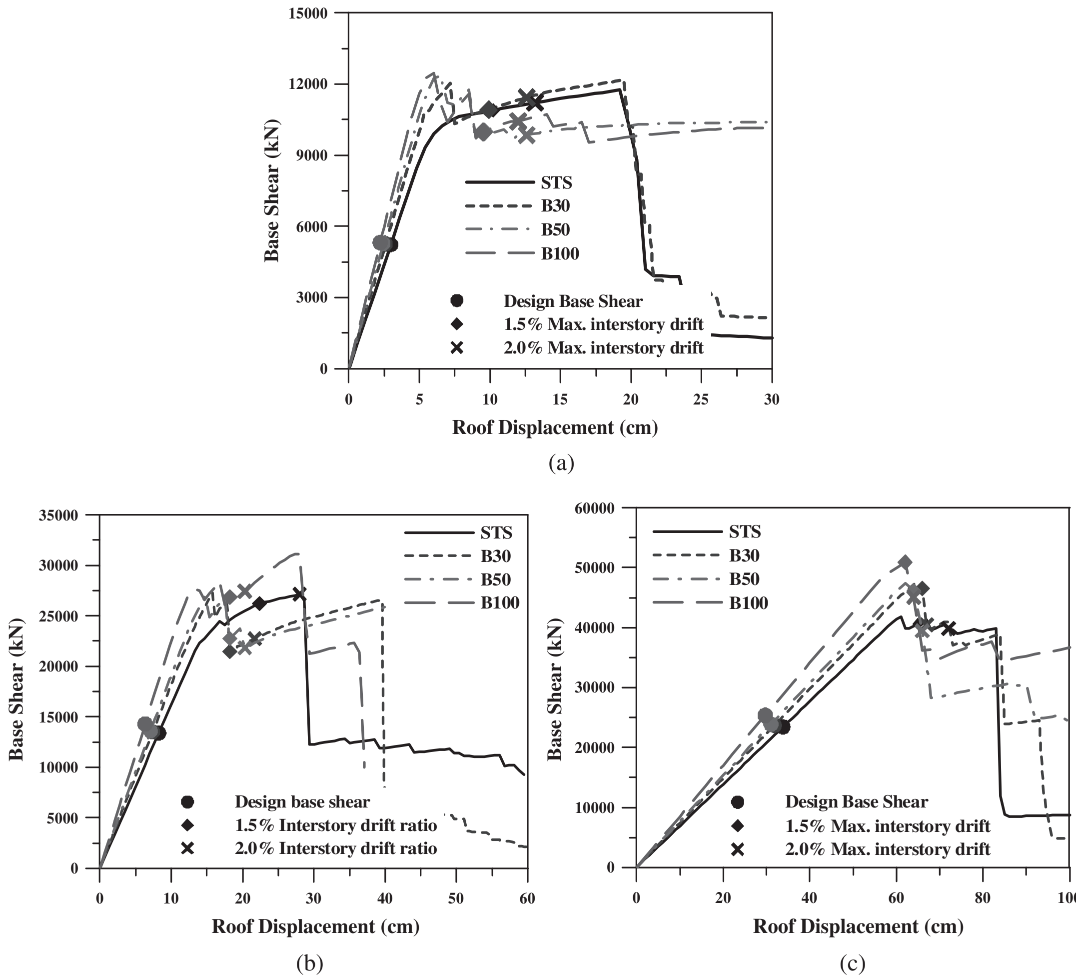

Figure 19. Pushover curves of the STS model structures with increased moment of inertia of horizontal members: (a) 4-story STS; (b) 10-story STS; (c) 30-story STS

structure slightly increased, but the strength soon dropped to three-quarters point. After that the structure remained stable until a significant amount of lateral drift occurred, especially when the moment of inertia of the horizontal members was increased by more than 50%. In those cases it was observed that the plastic hinges formed first in the second story spreads to Vierendeel panels in other stories before collapse. In the 10-story structure, even though the strength slightly dropped prematurely, as in the case of the 4-story structure, the overall system ductility at failure generally increased with the increase of the moment of inertia of the horizontal members. In the 30-story structure, the strength increased slightly with the reinforcement of the horizontal members, but the ductility rather decreased. Figure 20 depicts the pushover curves of the model structures with both ultimate strength and stiffness of the horizontal members increased, where it can be observed that the increase of the member strength did not contribute much to the seismic performance of the structures.

In the next approach to increase the overall ductility of the system, the moment of inertia of the vertical members in the Vierendeel panels was increased by 50% (V50) and 100% (V100). It can be

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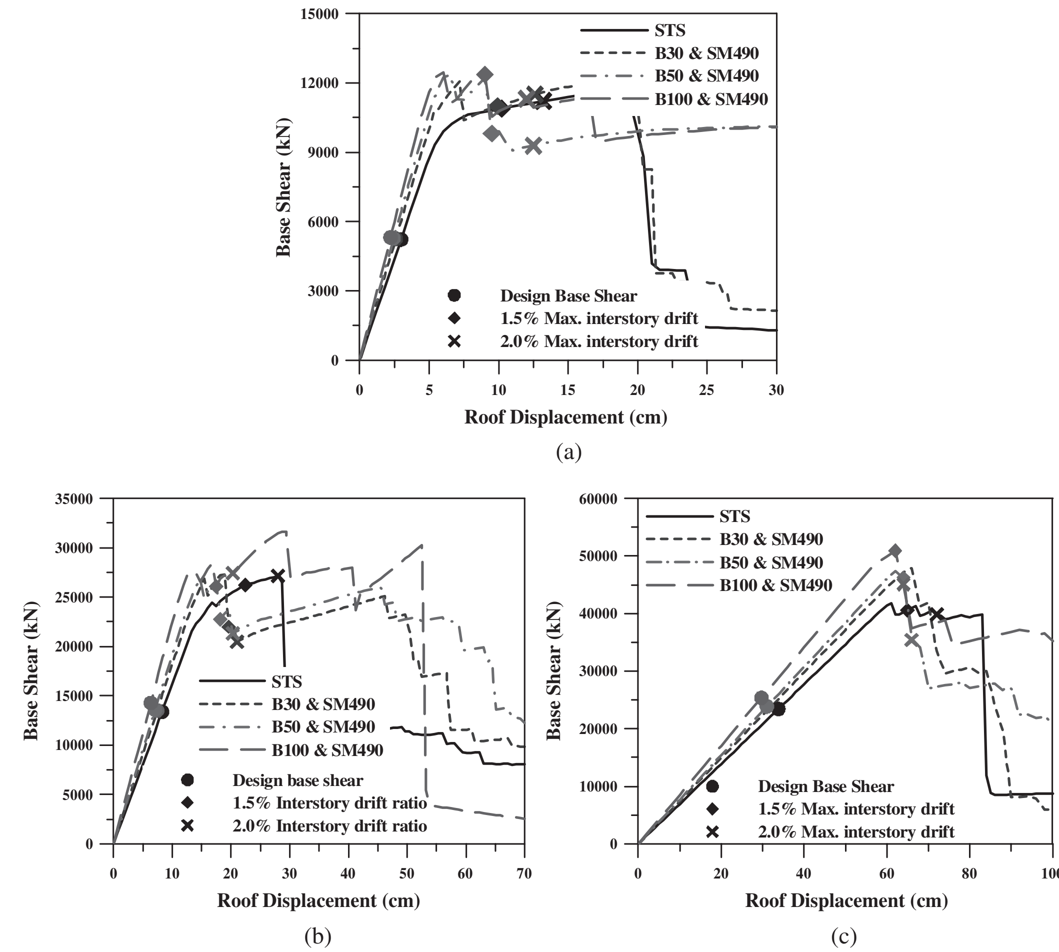

Figure 20. Pushover curves of the STS model structures with increased strength of horizontal members: (a) 4-story STS; (b) 10-story STS; (c) 30-story STS

observed in Figure 21 that strengthening of the vertical members increased the ductility of the 10- and 30-story structures significantly without premature drop of strength. It was observed that the plastic hinges first formed at horizontal members of a Vierendeel panel; then they occurred in the Vierendeel panels of the nearby stories, not in the vertical members of the same story, which delayed the formation of a weak story. Compared with the reinforcement of the horizontal members, which normally span the whole trusses, the strengthening in this case is limited only to the two vertical members in each Vierendeel panel.

As a next approach to increase seismic load-resisting capacity of STS, the structures were redesigned with some truss members replaced by buckling-restrained braces (BRB). BRB are steel members with a load-resisting internal core confined by an external steel tube. In this study all or part of the vertical and diagonal members of STS were replaced by BRB and the seismic performance was investigated. Table 4 shows four different application schemes of BRB investigated in this study. Figure 22 presents the nonlinear static analysis results of the STS with four different applications of BRB. In

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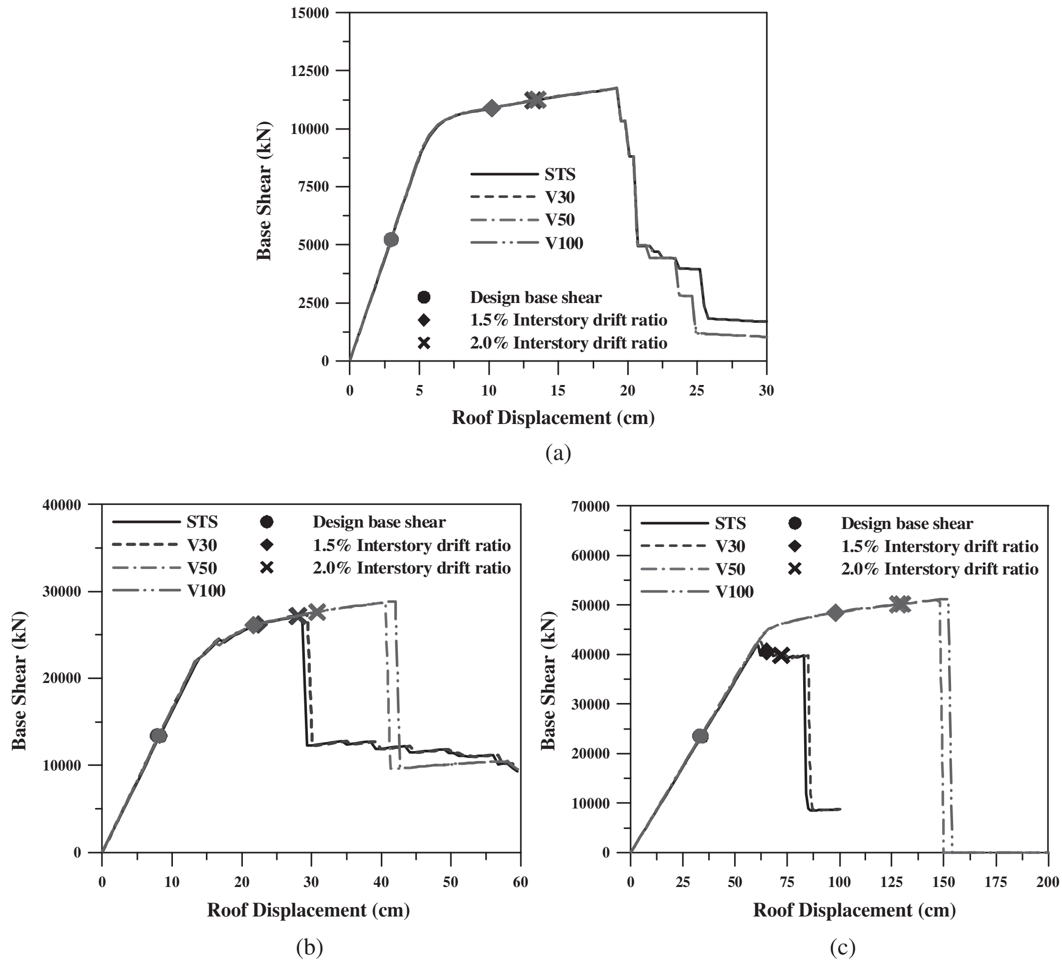

Figure 21. Pushover curves of the STS model structures after reinforcing vertical members: (a) 4-story STS; (b) 10-story STS; (c) 30-story STS

Table 4. Seismic retrofit schemes

Scheme	BRB location
Type 01	Vertical members of Vierendeel panels
Type 02	All vertical members in truss
Type 03	All diagonal members in truss
Type 04	All vertical and diagonal members

the 4-story structure, which has large inherent ductility compared with higher-story structures, the use of BRB is not so effective. In the 10- and 30-story STS, however, the application schemes Types 01, 02 and 04, in which various members including the two vertical members were replaced by BRB, results in significant increase in ductility. When BRB were utilized only at diagonal members (Type

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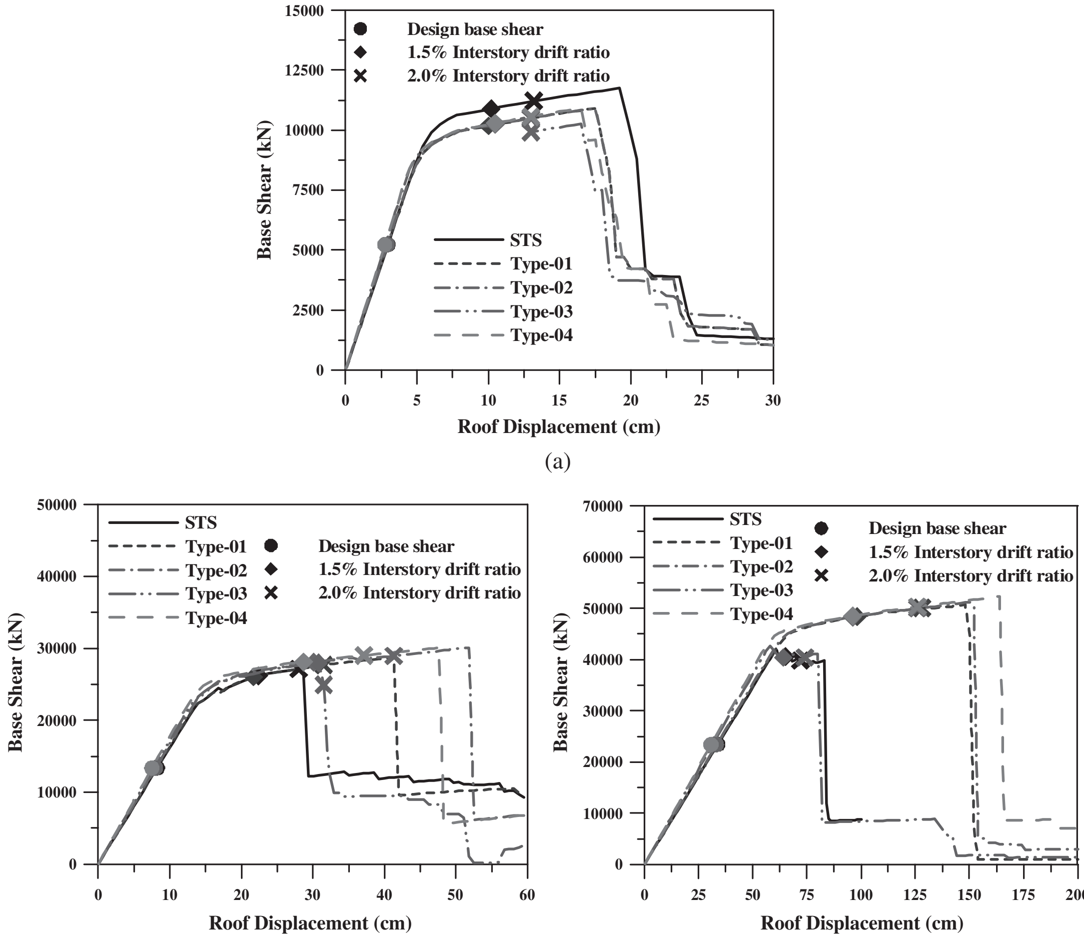

Figure 22. Pushover curves of the STS model structures with BRB: (a) 4-story STS; (b) 10-story STS; (c) 30-story STS

03), there was no significant change in the overall behavior. The Type 01 case, in which BRB were placed at two vertical members of Vierendeel panels, turned out to be very effective considering that only two BRBs were used per a staggered truss.

Figure 23 depicts the interstory drifts of the model structures with BRB applied in the vertical members of the Vierendeel panels (Type 01) when the maximum interstory drift reached 1·5% and 2·0% of the story height. It can be observed that with the use of BRB the interstory drifts became less concentrated in a few stories, especially in the 30-story structure.

The seismic behavior factors of the original and the reinforced structures (Type 01 and V50) were compared in Figure 24, where it can be seen that the overstrength factors of all structures were not changed significantly. However, the ductility and response modification factors of the 30-story structures increased well above the code-specified values as a result of the application of the reinforcing schemes. Therefore the reinforcement of a few critical members of high-rise STS may be quite effec-

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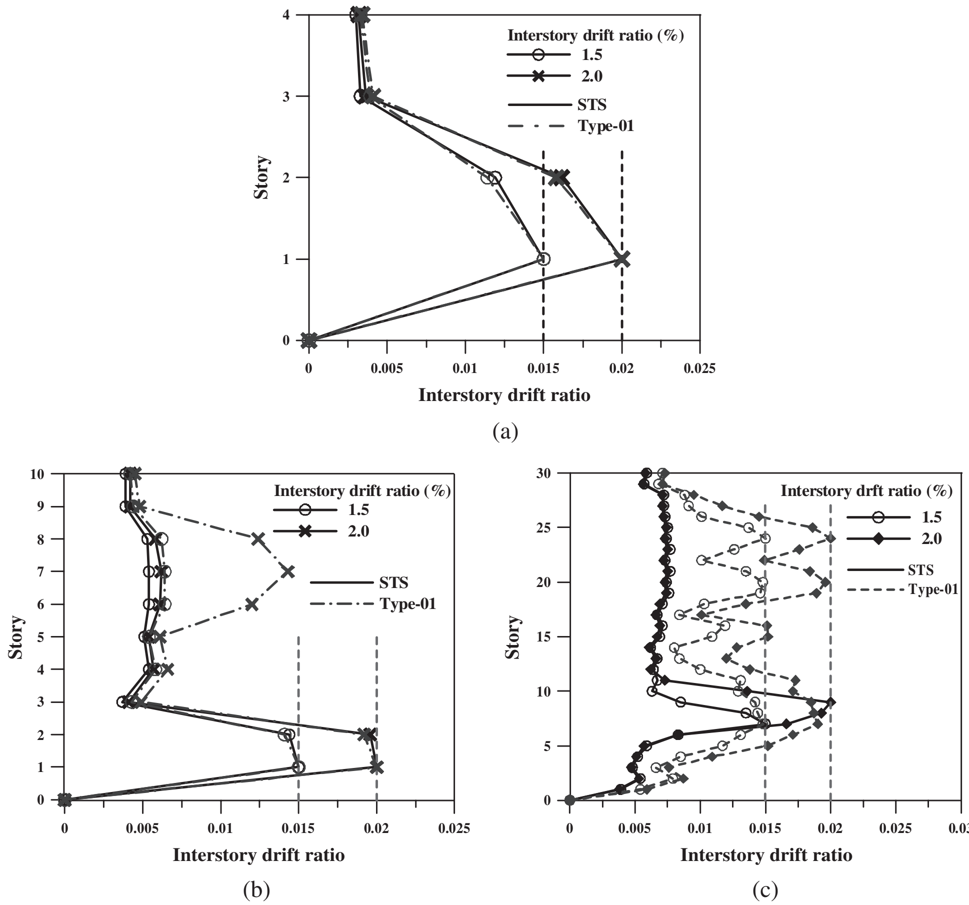

Figure 23. Interstory drifts of STS model structures with and without BRB: (a) 4-story STS; (b) 10-story STS; (c) 30-story STS

tive in enhancing the seismic load-resisting capacity of the entire system. Compared to the V50 scheme, which requires 50% increase of moment of inertia of the vertical members, the use of BRB with smaller cross-sectional area than the original members may be useful, especially when smaller section is required for architectural reasons.

8. CONCLUSIONS

In this study 4-, 10-, and 30-story staggered truss systems were designed and their seismic performances were evaluated by pushover analysis. The results were compared with the seismic performance of conventional moment-resisting frames and braced frames. Some reinforcing schemes were applied and their effects on enhancing lateral load-resisting capacity were investigated.

According to the analysis results, the staggered truss system showed superior or at least equivalent seismic load-resisting capacity to conventional ordinary concentric braced frames. The 4-story low-rise structure with staggered truss system turned out to have sufficient seismic-load-resisting capacity. However, in mid- to high-rise structures, localization of plastic damage in a Vierendeel panel

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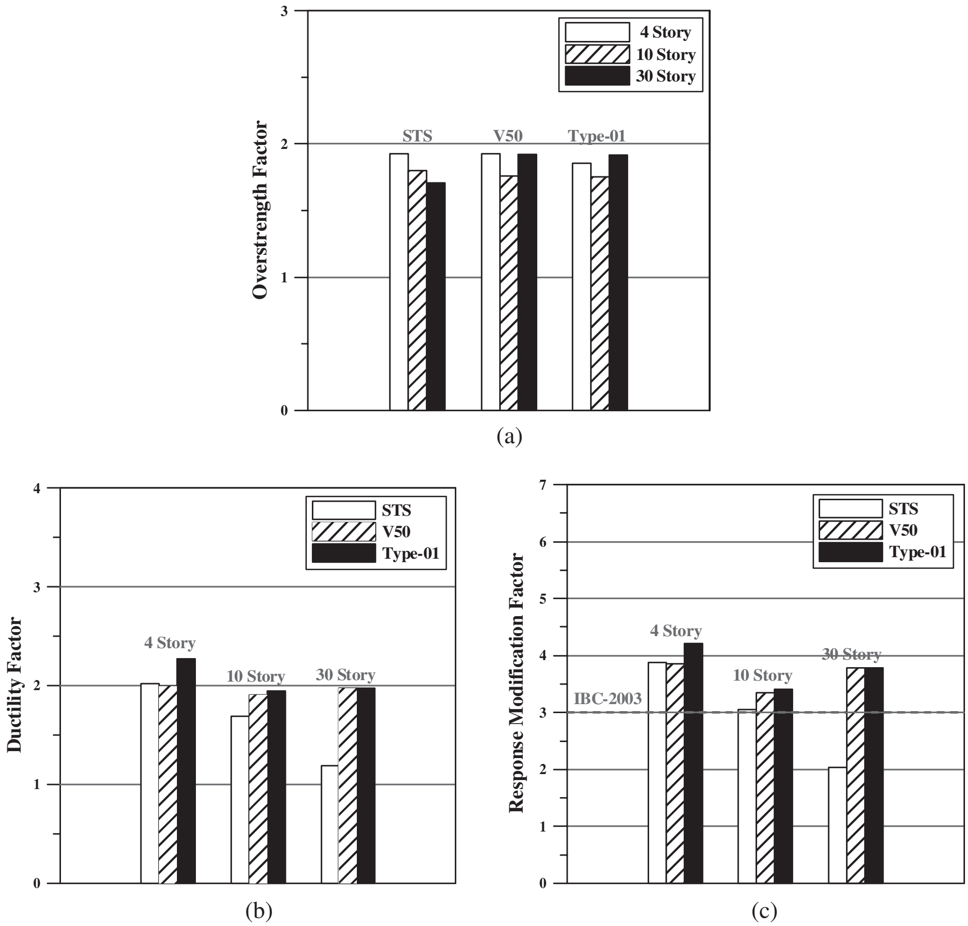

Figure 24. Behavior factors of the STS model structures with and without reinforcement: (a) overstrength factors; (b) ductility factors; (c) response modification factors

caused a weak story and resulted in brittle failure of the structure. It was also shown that strengthening of the horizontal and vertical members of the Vierendeel panels generally increased the overall ductility of the system. In particular, the use of buckling restrained braces in the vertical members of the Vierendeel panels enhanced the system ductility without increasing the cross-sectional area of the elements.

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