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Progressive collapse resisting capacity of braced frames

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SUMMARY

In this study, the progressive collapse potential of braced frames was investigated using nonlinear static and dynamic analyses. Eight different bracing types were considered and their performances were compared with those of a special moment-resisting frame designed with the same design load. According to the pushdown analysis results, most braced frames designed per current design codes satisfied the design guidelines for progressive collapse initiated by loss of a first story interior column; however, most model structures showed brittle failure mode caused by buckling of braces and columns. Among the braced frames considered, the inverted-V type braced frames showed superior ductile behaviour during progressive collapse. The nonlinear dynamic analysis results showed that all the braced structures remained in stable condition after sudden removal of a column, and their deflections were less than that of the moment-resisting frame. Copyright © 2009 John Wiley & Sons, Ltd.

1. INTRODUCTION

In steel structures, braces are often applied as economic means of resisting lateral loads. The Seismic Provisions for Structural Steel Buildings of AISC specifies a special concentric braced frame (SCBF) based on experimental and analytical works (AISC, 2002). To ensure significant inelastic deformations when subjected to design seismic load, the SCBF is designed to meet various requirements such as slenderness, strength, width-thickness limitations, and special design and detailing of end connections, etc. Previously, the braced frame was widely investigated for seismic application both experimentally (Uang and Bertero, 1986; Whittaker *et al.*, 1990; Jones *et al.*, 2002; Ricles *et al.*, 2002) and analytically (Khatib *et al.*, 1988; Roeder, 1989; Remennikov and Walpole, 1997; Tremblay, 2002). The focus of those studies was on the strength of individual members, such as slenderness ratio or width/thickness ratio, or on the elastic/inelastic behaviour of braced frames for various geometries or locations of braces.

A series of recent accidents have led the structural engineering community towards the assessment and enhancement of structural robustness under abnormal loads. Previous research on progressive collapse of steel structures has been focused on moment-resisting frames (Kaewkulchai and Williamson, 2003; Kim and Kim, 2008; Kim and Park, 2008; Kim *et al.*, 2009). Khandelwal and El-Tawil (2007) and Kim and An (2009) investigated the effect of catenary action on collapse of steel moment-resisting frames. Recently, Khandelwal *et al.* (2008) investigated the progressive collapse resistance of seismically designed steel special concentrically braced frames and eccentrically braced frames. They found that the eccentrically braced frame is less vulnerable to progressive collapse than the concentrically braced frame. However, the progressive collapse performance of braced frames with various bracing configurations for accidental loss of a vertical element still needs further investigation.

In the USA, the General Service Administration (GSA) presents a practical guideline for design to reduce progressive collapse potential of federal buildings (GSA, 2003), and the Department of Defence (DoD) also presents a guideline for the new and existing DoD buildings (Unified Facilities Criteria (UFC)-DoD, 2005). In such guidelines and other design standards (British Standards Institute,

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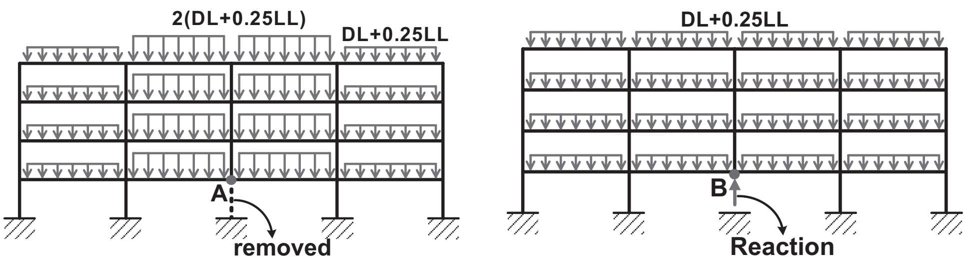
2000; Eurocode 1, 2002; New York City Department of Buildings, 2003), the braced frames are required to be distributed throughout the structure to enhance structural integrity against progressive collapse.

The present study focuses on the evaluation of progressive collapse potential of various braced frames. The performances of structures with eight different bracing types are investigated by nonlinear static and dynamic analyses. In addition, a steel special moment-resisting frame (SMRF) and an SMRF with knee braces are analysed and the results are compared with those of brace frames.

2. ANALYSIS PROCEDURES FOR PROGRESSIVE COLLAPSE

In this study, the performance of various braced frames subjected to sudden removal of a column was investigated by nonlinear static and dynamic analysis using the program code SAP2000 (2004). For static analysis, both the GSA (2003) and the Unified Facilities Criteria (UFC)-DoD (2005) recommend the use of dynamic amplification factor of 2.0 in load combination as shown in Figure 1. The DoD guidelines recommend the use of larger gravity load than the load recommended by the GSA guidelines and include wind load in the load combination. The nonlinear static pushdown analysis method is applied to investigate the structural performance of buildings against progressive collapse by gradually increasing the vertical displacement in the location of the removed column. This procedure is useful in determining elastic and failure limits of the structure. To carry out dynamic analysis, the axial force acting on a column is computed before it is removed. Then the column is replaced by a point force equivalent of its member force as shown in Figure 1(b). To simulate the phenomenon that the column is abruptly removed from an equilibrium state, the force is removed after a certain time is elapsed as shown in Figure 2.



(a) Static analysis (b) Dynamic analysis

Figure 1. Imposed loads for progressive collapse analyses.

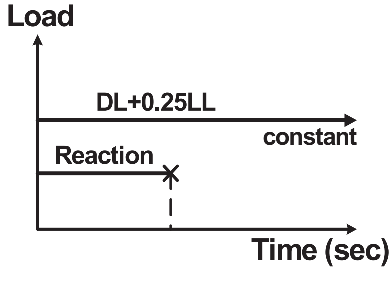


Figure 2. Time histories of imposed loads for dynamic analysis.

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3. ANALYSED STRUCTURAL MODELS

3.1. Design of structural models

Common configurations for concentric bracing systems include V-type, inverted-V (chevron)-type, X-type and diagonal braces, etc. The strength of the ordinary concentric braced frame is known to drop rapidly when the girders connected to the braces yield due to the unbalanced force caused by buckling of compression braces. The special concentric bracing systems, on the other hand, retain significant strength even after the first buckling of a brace because the girders are designed to resist the unbalanced force. The analysed structural models used in this study are the special concentric braced frames designed in accordance with the AISC Load and Resistance Factor Design (AISC, 2000) and the Seismic Provisions for Structural Steel Buildings (AISC, 2002). The design dead and live loads are 4.3 and 2.4 kN/m^2, respectively. The design seismic load is obtained from the Minimum Design Loads for Buildings and Other Structures (ASCE 7-05, 2005); the design spectral acceleration parameters S_s and S_1 are 1.5 and 0.6, respectively, in the IBC-2006 format (ICC, 2006a); and the site coefficients F_a and F_v are 1.0 and 1.5, respectively. The response modification factor of 6 is used for special concentric braced frames. The story height is 3.1 m in every story and the span length is 6.1 m. The beams and columns are made of ASTM A992 steel and the braces are hollow steel tubes made of A500-46 steel.

The analysed structural models are basically four-storey structures with four bays subjected to the loss of the first-storey centre column, in which the structure deforms symmetrically and the full capacity of bracing is activated. Figure 3 shows the various bracing configurations to be analysed, and for comparison, a special moment-resisting frame designed for the same loading condition is included in

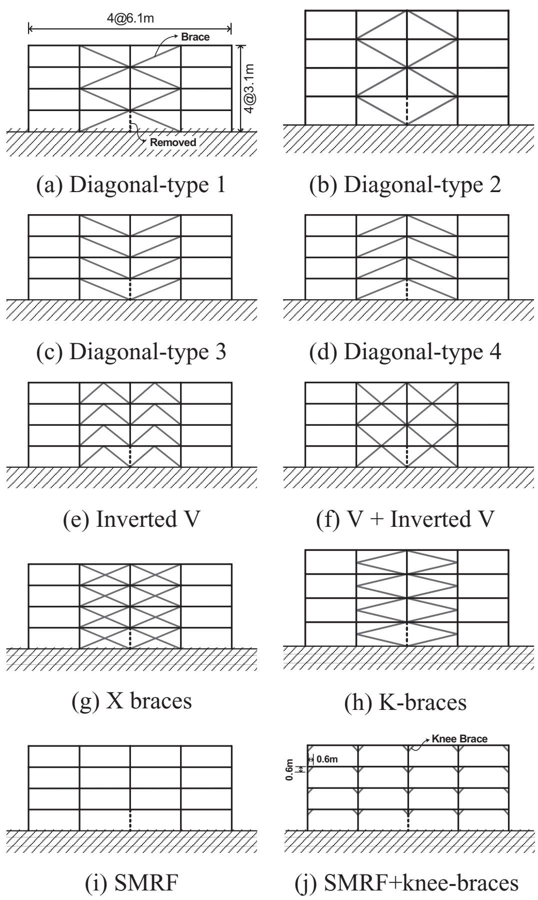

Figure 3. Configurations of various braced frames.

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the analysis models (Figure 3(i)). The bracing configurations used are the four types of diagonal bracing (Figure 3(a–d)), a chevron (inverted V) bracing (Figure 3(e)), an X-bracing (Figure 3(f)), a bracing with V and inverted V bracing located alternatively (Figure 3(g)) and a knee bracing (Figure 3(j)). For seismic design of the special moment-resisting frame and the knee-braced frame, the response modification factors of 8 and 3 are applied, respectively. Table 1 shows the member sizes of the structure with chevron bracing.

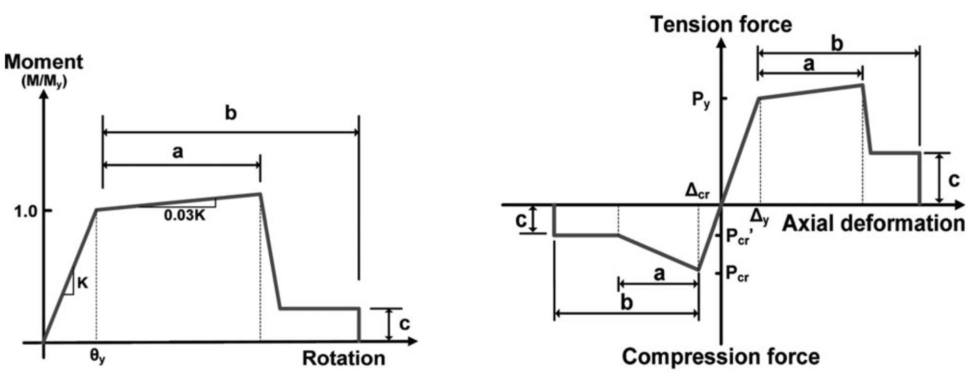
3.2. Modeling for analysis

The GSA guidelines for progressive collapse provide acceptance criteria for steel members, where the maximum ductility and rotation angle of steel beams, columns and braces are presented. In this study, however, the load–displacement relationships given in the FEMA-356 (FEMA, 2000) are used instead because failure criteria are provided more in detail considering width-thickness ratios and connection types. Figure 4 shows the force–deformation relationships for flexural members and braces given in the FEMA-356. In Figure 4(a), the column rotation angle at yield is computed considering axial load using Equation (5.2) of the FEMA-356, and the coefficients a, b and c in Figure 4(a) are determined based on the computed yield rotation. In Figure 4(b), the post-buckling strength of braces, P_{cr}, is determined to be 40% or 20% of the buckling strength, P_{cr}, depending on the width-thickness ratio. The post-yield stiffness of members is assumed to be 3% of the initial stiffness and the beam–column joints are pin-connected. The damping ratio of the structures is assumed to be 5% of the critical damping. To simulate dynamic performance of the structures caused by sudden removal of a column, the reaction at the beam–column joint is suddenly removed 5 s after the loading.

Bracing members are typically connected to the beams and columns in the braced bays by gusset plates with either welding or bolting. Bracing connections in SCBF are not permitted to be the weak link in the building design, and are required to have sufficient strength to ensure that they will not fracture under forces corresponding to a more desirable yield mechanism of the structure (ICC, 2006b). The AISC Seismic Provisions of 2002 require that the strength of bracing connections should be the lesser of the nominal axial tensile strength of the bracing member and the maximum force that

Table 1. Member size of chevron braced frame.

		1st–2nd storeys	3rd–4th storeys
Columns	Exterior	W5 × 19	W4 × 13
	Interior	W8 × 40	W5 × 16
Beams	Exterior	W12 × 35	W12 × 35
	Interior	W21 × 132	W21 × 122
Braces		HSS4-1/2 × 4-1/2 × 0.3125	HSS4 × 4 × 0.3125



(a) Flexural members (b) Braces

Figure 4. Nonlinear force–deformation relationships for structural members.

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can be transferred to the brace by the system (AISC, 2002). Therefore, in this paper, it is assumed that the connections of the analysed structural models are designed to remain essentially elastic at all times and the plastic deformation occurs only in the bracing members.

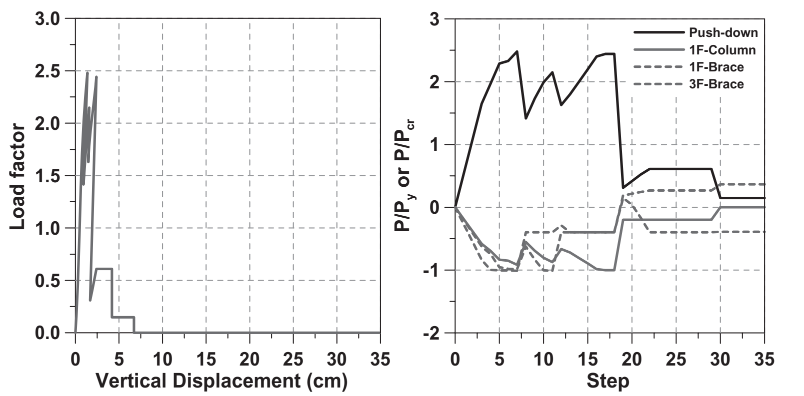
4. ANALYSIS RESULTS

4.1. Static pushdown analysis

Nonlinear static pushdown analyses are carried out by removing the first-storey centre column and gradually increasing the vertical deflection at the beam–column joint from which the column is removed. In the pushdown analysis, the maximum strength less than 1.0 implies that the structure cannot resist the load 2(DL+0.25LL) specified in the GSA guidelines.

The braces installed in each storey of the Diagonal types 1 and 2 braced frames shown in Figure 3(a,b), respectively, are subjected to tension and compression alternatively when the first-storey centre column is suddenly removed, whereas in the Diagonal types 3 and 4, all the braces are subjected to only tension or compression. The pushdown curve depicted in Figure 5(a) shows that the load factor of the Diagonal brace type 1 (Figure 3(a)) reaches up to 2.5 and the structure has enough strength to resist progressive collapse caused by loss of a first-storey column. However, after the maximum strength is reached, the strength drops abruptly with further increase of vertical displacement. Figure 5(b) shows the variation of member axial forces divided by the buckling load (P_{cr}) for compression members or by the yield strength (P_y) for tension members along with the pushdown curve. It can be observed that the braces located in the first and the third storeys buckle first followed by the buckling of the first-storey columns. After buckling of the columns, the structure loses most of its resistance to progressive collapse. Similar results are obtained in the Diagonal brace type 2 as shown in Figure 6 except that the maximum load factor is smaller than that of the Diagonal brace type 1 since the first-storey braces do not participate in resisting progressive collapse in this case. In the structure with Diagonal brace type 3 shown in Figure 3(c), all braces are subjected to tension, and before the tensile yield strength of the braces is reached, the structure becomes unstable due to buckling of the third-storey columns (Figure 7). When diagonal braces are installed in such a way that all braces are subjected to compression (Figure 3(d)), the first-storey braces buckle first, followed by buckling of the first-storey column and the braces in the other stories (Figure 8(b)).

Figure 9 depicts the performance of the structure with inverted V braces (Figure 3(e)) subjected to vertical pushdown force. Figure 9(c) illustrates the buckled members in the structure at the analysis steps corresponding to the six points marked on the pushdown curve in Figure 9(b), where it can be observed that the third-storey braces under compression buckle first followed by the buckling of the third-storey columns and second-storey braces. Once the compression braces buckle, the girders con-



(a) Pushdown curve (b) Performance of members

Figure 5. Load–displacement relationship of the Diagonal-type 1 braced frame.

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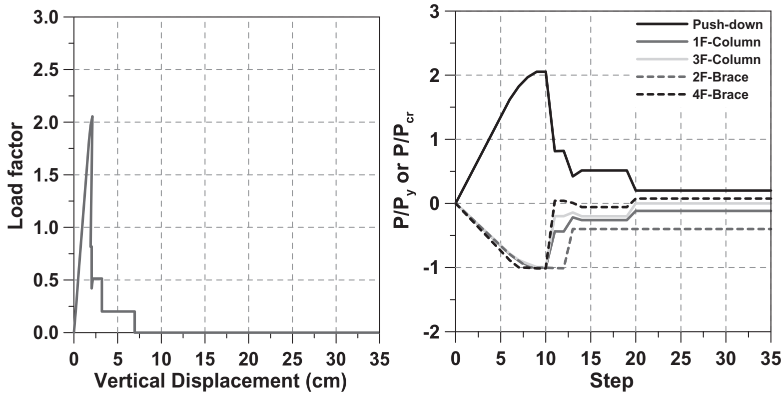
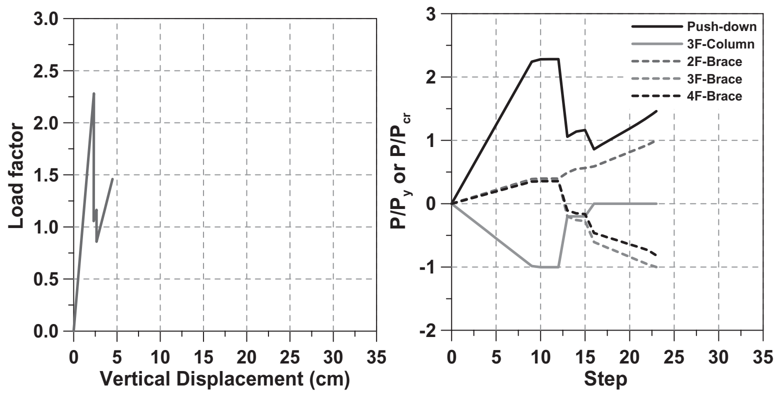


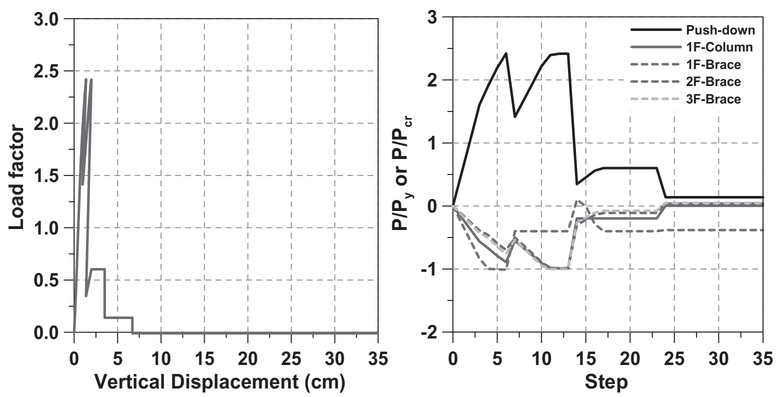
(a) Pushdown curve (b) Performance of members

Figure 6. Load–displacement relationship of the Diagonal-type 2 frame.



(a) Pushdown curve (b) Performance of members

Figure 7. Load–displacement relationship of the Diagonal-type 3 frame.



(a) Pushdown curve (b) Performance of members

Figure 8. Load–displacement relationship of the Diagonal-type 4 frame.

nected to the buckled braces resist the load and transfer it to the adjacent braces and columns which are still capable of resisting additional load. This leads to more ductile pushdown behaviour compared with other bracing types even though the maximum strength is slightly smaller than those of structures with diagonal braces.

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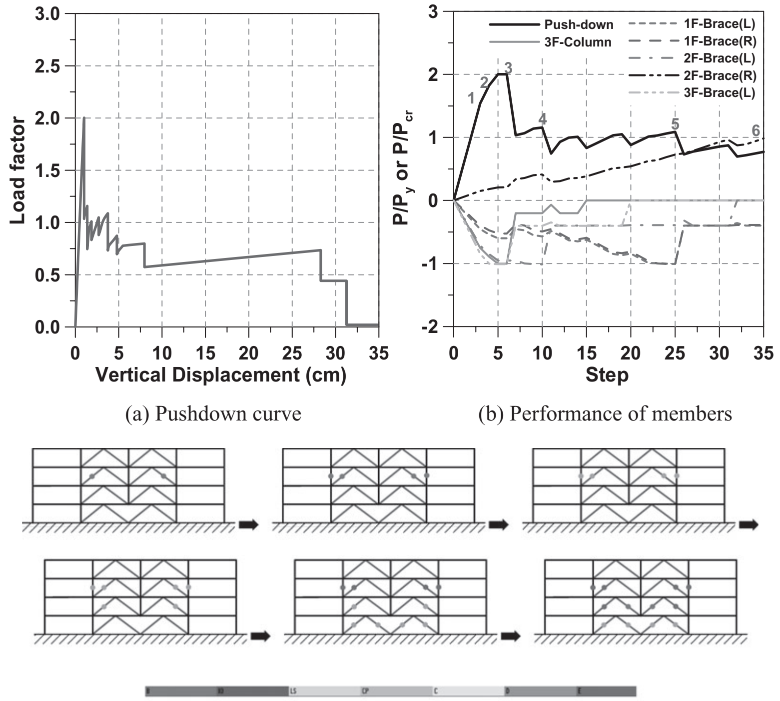
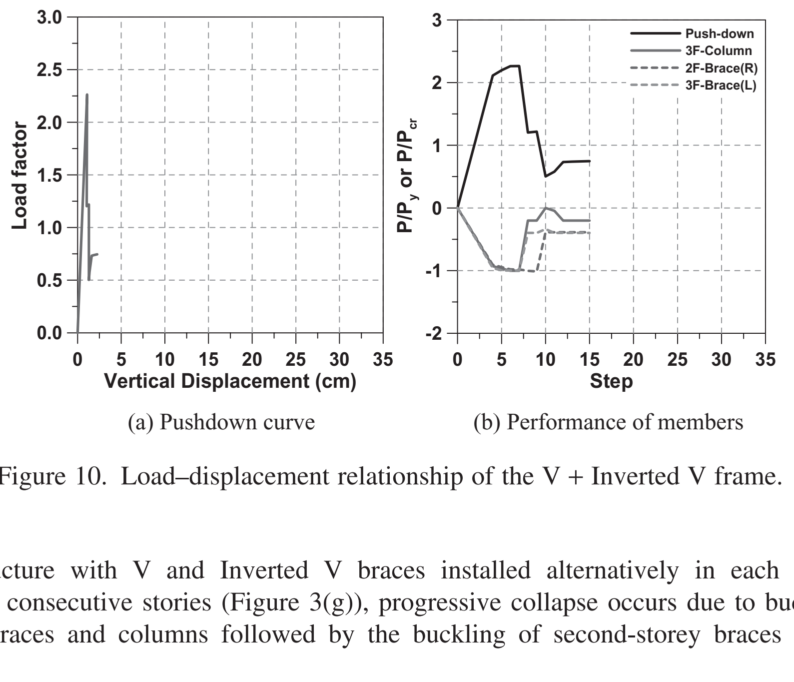


(a) Pushdown curve (b) Performance of members

(c) Distribution of plastic hinges at each loading state

Figure 9. Load–displacement relationship of the Inverted V frame.



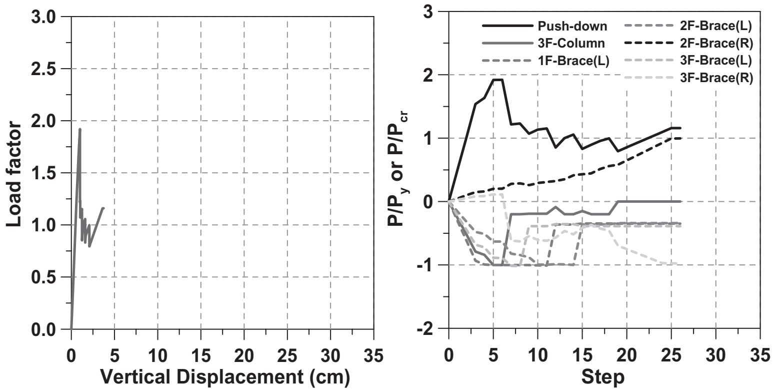
(a) Pushdown curve (b) Performance of members

Figure 10. Load–displacement relationship of the V + Inverted V frame.

In the structure with V and Inverted V braces installed alternatively in each storey or X braces in two consecutive stories (Figure 3(g)), progressive collapse occurs due to buckling of the third-storey braces and columns followed by the buckling of second-storey braces as shown in Figure 10.

Figure 11 shows the pushdown curve of the structure with X-braces shown in Figure 3(f) and the variation of member forces, where it can be observed that buckling occurs in both the braces and the columns. It is interesting to note that when the third-storey columns buckle, the third-storey tension braces start to be subjected to compression.

Figure 12 shows the pushdown curve of the K-braced frame shown in Figure 3(h), where it can be observed that the maximum load factor is about 1.5 which is the smallest among the model structures.



(a) Pushdown curve (b) Performance of members

Figure 11. Load–displacement relationship of the X-braced frame.

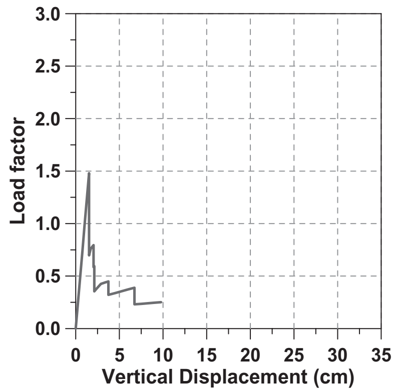


Figure 12. Pushdown curve of the K-braced frame.

After buckling occurs in compression braces, the columns to which the braces are connected fail due to the unbalanced force between the buckled compression braces and the tension braces. The K-bracing is not recommended for seismic design of structures mainly because of the possibility of column buckling followed by premature failure of the structure. The same phenomenon was obtained in the pushdown analysis for progressive collapse.

Figure 13(a) shows the pushdown curve of the SMRF depicted in Figure 3(i), where it can be observed that the maximum strength reaches 2.0 and the structure has higher ductility than the braced frames. This is reasonable considering the fact that the resistance of the SMRF to the progressive collapse is by the bending deformation of girders and therefore no buckling is involved, and that the structure is designed not by strength but by the inter-storey drift limitation for seismic load and therefore retains enough residual strength. Moreover, as the structure is designed to satisfy the strong column and weak beam requirement, plastic hinges form only in girders while columns remain in elastic range. Figure 13(b) shows the pushdown curve of the moment–resisting frame with knee-braces as shown in Figure 3(j). The structure is designed for the same design loads using the response modification factor of 3. The special requirements of the seismic provision (AISC, 2002) applied for the SMRF are not applied in this case. Figure 13(b) shows that the MRF with knee braces has strength and ductility against progressive collapse even larger than the SMRF structure designed for the same design loads.

Figure 14 shows the pushdown curves of selected four-storey model structures with the first storey second column suddenly removed. In this case, the structures deform unsymmetrically. It is observed that the failure mode is similar to that of the previous case of centre column removal; compression

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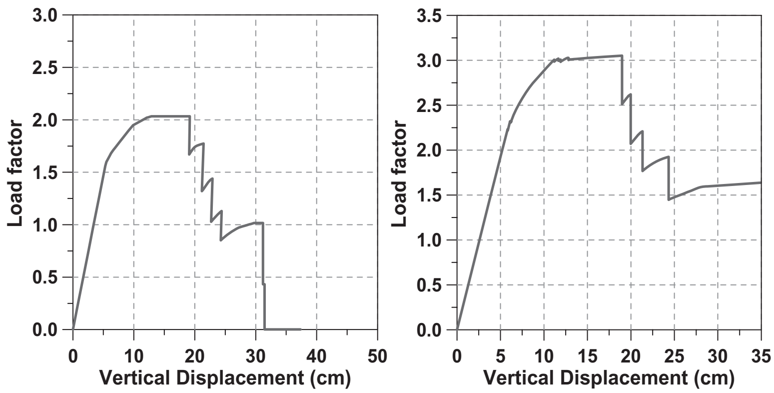

(a) SMRF (b) MRF+knee braces

Figure 13. Pushdown curves of the moment-resisting frames.

braces buckle first followed by the buckling of columns. As soon as columns buckle, the overall strength immediately drops and the structure collapses. It was observed that, as the structures deform unsymmetrically, only a part of the braces participates in resisting collapse. This results in smaller load factors compared with those obtained when the centre column is removed. In the SMRF structure, however, the overall behaviour is quite similar to the case when the centre column is removed and the load factor reaches as high as 2.0. As in the case of the centre column removal, progressive collapse is initiated by plastic hinge formation in girders and all columns remain elastic throughout the process.

Figure 15 shows the pushdown curves of the four-bay four-, six- and eight-storey Inverted V braced frames, where it can be observed that as the number of story increases, the maximum load factor also increases. The ductility, however, does not increase proportionally to the number of stories.

Figure 16(a) depicts the three-bay structure with Inverted-V braces located in the centre bay. The structure becomes unsymmetric when any of the first-storey interior columns is suddenly removed. This leads to plastic hinge formation in the remaining first-storey columns followed by occurrence of collapse mechanism. In this case, all the braces remain in elastic range regardless of bracing configuration. Figure 16(b) shows the pushdown curve of the structure, where it can be observed that the load factor barely reaches 0.5 due to unsymmetric failure mode and the structure is vulnerable to progressive collapse.

4.2. *Nonlinear dynamic time history analysis*

Nonlinear dynamic analyses are carried out by suddenly removing the member force of the lost column as depicted in Figure 2. Figure 17(a–i) shows the vertical deflection time histories of the analysed structural models subjected to sudden loss of the first-storey centre column, where the horizontal dotted lines indicate the linear static analysis results when the structures are subjected to the gravity load of DL+0.25LL. The analysis results show that the dynamic responses gradually approach to static responses (obtained without applying the dynamic amplification factor). This implies that no permanent deformation occurs and the structures behave elastically when the centre column is suddenly removed. This can be expected from the pushdown analyses results with the maximum load factors exceeding 2.0 in most cases. It can be noticed that the amplitude of vibration is smaller when the braces connected to the beam-removed column joint are under compression than when they are under tension. The maximum deflections of the moment frame and the moment frame with knee-braces are larger than those of braced frames. Figure 18 shows the vertical deflection time-histories of selected model structures caused by sudden loss of the first-storey second column. In this case, unsymmetric vibration mode is involved in the vibration and therefore the maximum deflections are significantly larger than those obtained by removing the centre column. However, as can be expected from the pushdown analysis results, in which the load factors are

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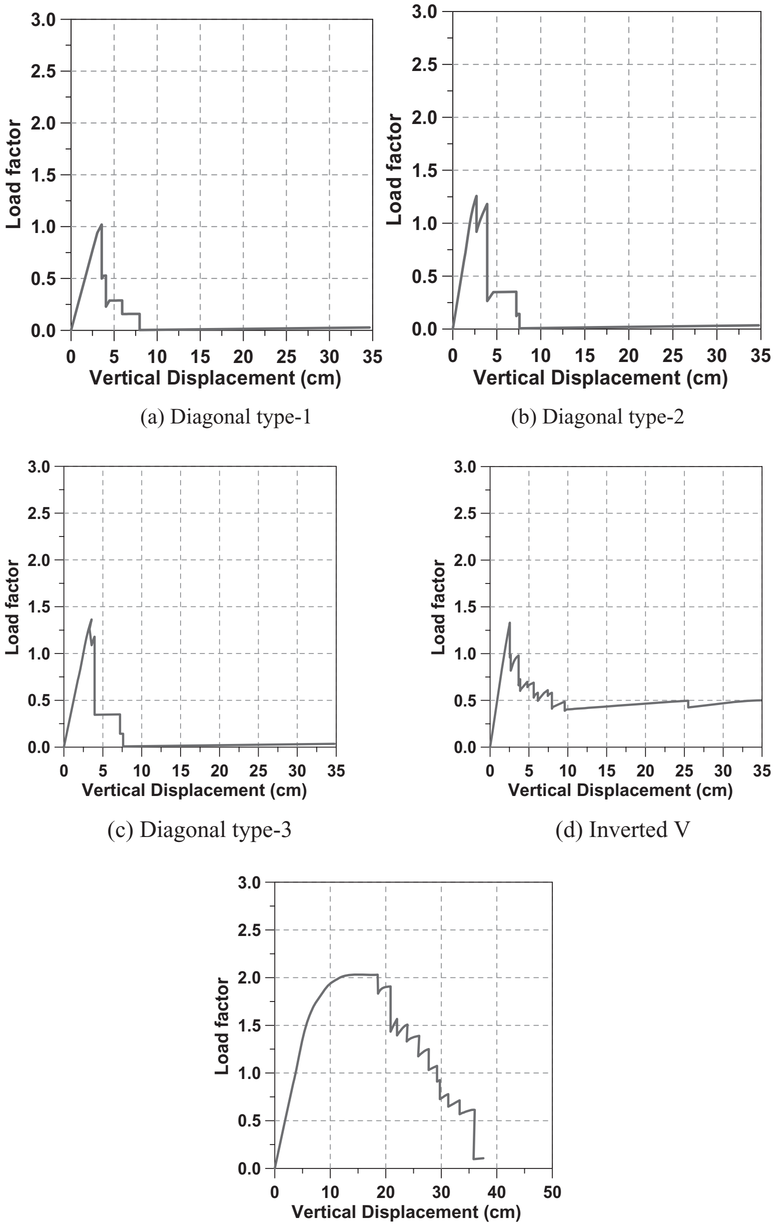

(a) Diagonal type-1

(b) Diagonal type-2

(c) Diagonal type-3

(d) Inverted V

(e) SMRF

Figure 14. Pushdown curves of the model structures subjected to sudden loss of the first-storey second column.

larger than 1.0, the nonlinear dynamic analysis results for maximum deflection gradually approach the linear static analysis results. This implies that the structures remain elastic and stable after the first-storey second columns are suddenly removed.

5. CONCLUSIONS

This study investigated the progressive collapse resisting capacity of framed structures with steel braces using nonlinear static and dynamic analysis methods. The structures were designed with various types of braces located symmetrically.

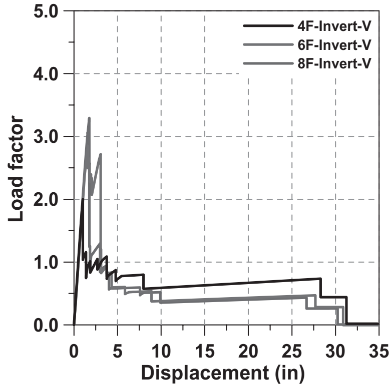

Figure 15. Pushdown curves of the 4-bay inverted-V braced frames with varying number of storeys

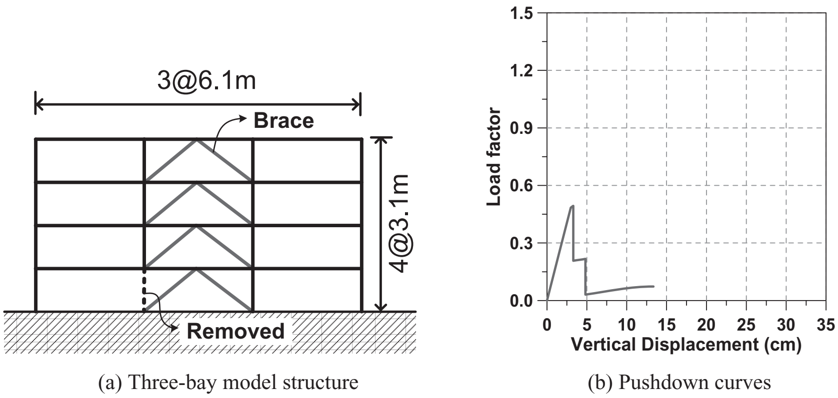

Figure 16. Pushdown analysis result of the four-storey three bay Inverted-V braced frame subjected to the loss of a first-storey internal column.

According to dynamic analysis results, the model structures generally remained stable after the first-storey centre column was suddenly removed. The nonlinear static pushdown analysis results showed that the model structures had inherent strength twice as high as the strength required by the GSA guideline except the K-braced frame in which premature failure occurred due to column buckling. However, after the maximum values were reached, the strengths sharply dropped. It was observed that after buckling of compression braces, some columns buckled before tension braces yielded, resulting in brittle failure modes. Only in the Inverted-V braced frame the girders resisted the unbalanced force between the elastic tension braces and the buckled compression braces, and thus collapse was somewhat delayed compared with other model structures with different bracing configurations. Therefore, to prevent brittle failure of braced frames, it would be necessary to reinforce columns connected to braces. The structures with braces only in single bay turned out to be very vulnerable when a column adjacent to the braced bay was lost. In this case, the progressive collapse can be prevented by designing the frame as moment frame stiffened by braces.

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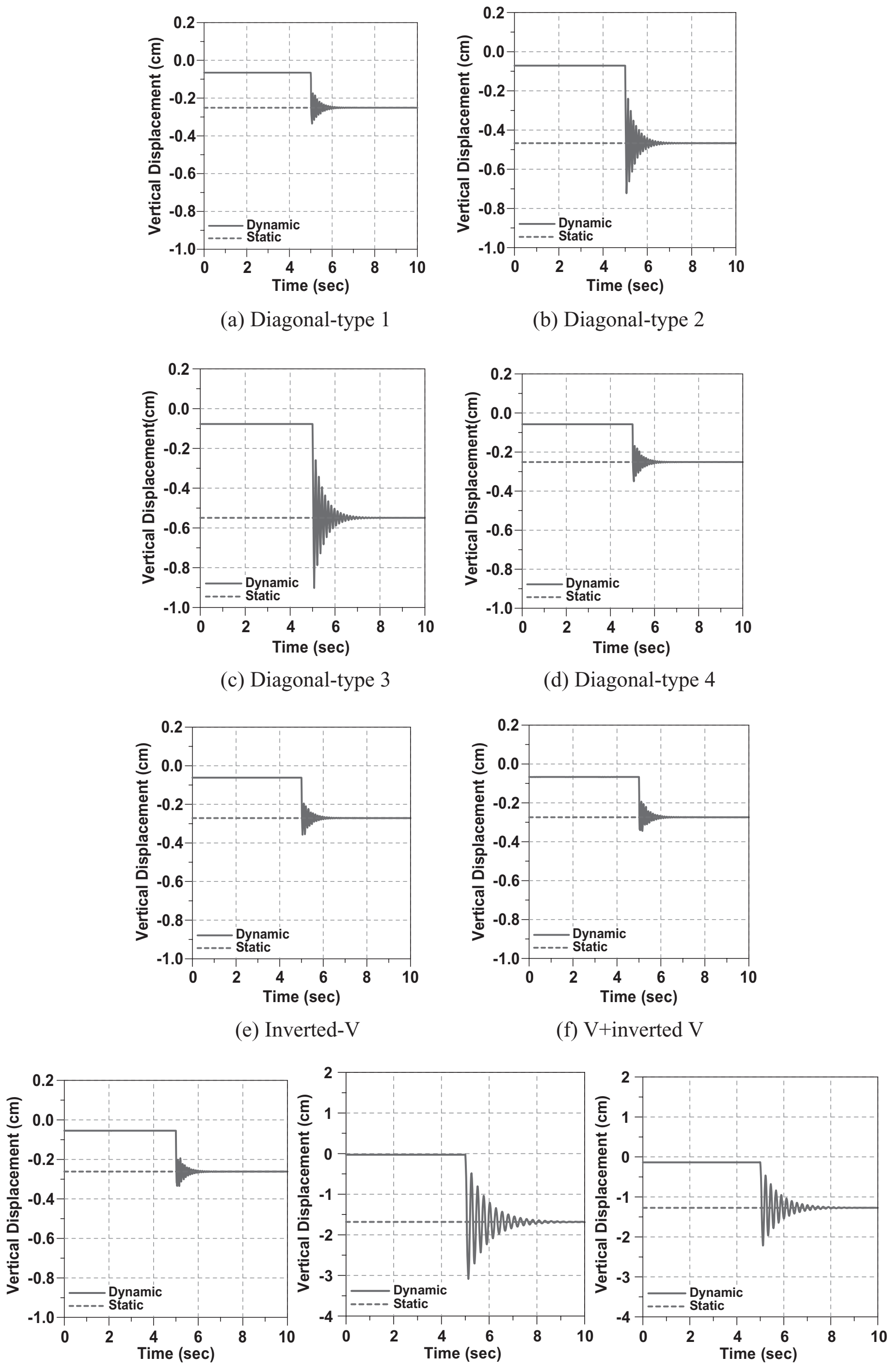

(a) Diagonal-type 1
(b) Diagonal-type 2
(c) Diagonal-type 3
(d) Diagonal-type 4
(e) Inverted-V
(f) V+inverted V
(g) X-braces
(h) SMRF
(i) SMRF+Knee-braces

Figure 17. Time history analysis results of the 4-bay model structures subjected to loss of the first-storey centre column.

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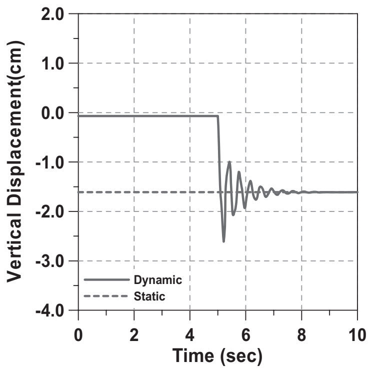
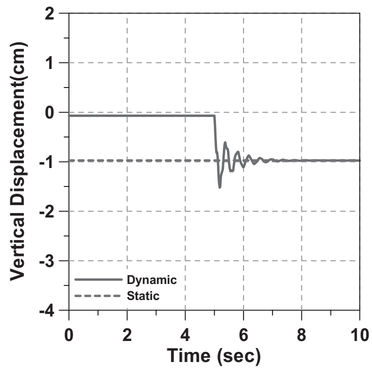
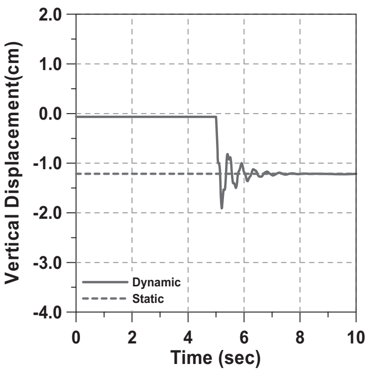


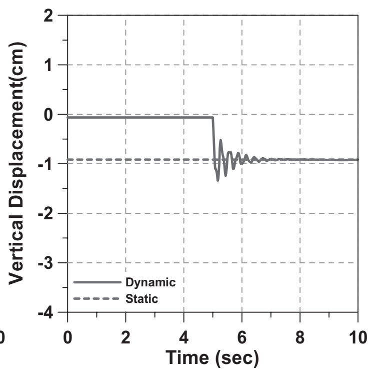
(a) Diagonal-type 1



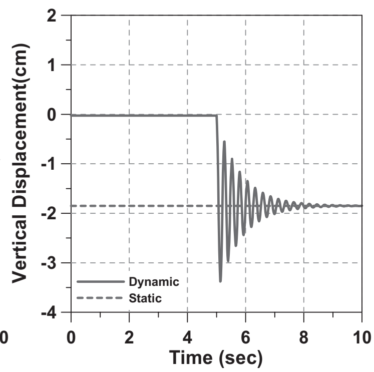
(b) Diagonal-type 2



(c) Diagonal type-3



(d) Inverted V



(e) SMRF

Figure 18. Time history analysis results of the 4-bay model structures subjected to loss of the first-storey second column.

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