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Progressive collapse-resisting capacity of framed structures with infill steel panels

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ABSTRACT

In this study the effect of infill steel panels on enhancing the progressive collapse resisting capacity of moment frames is evaluated, and a simple design procedure for infill steel plates is proposed to enhance the progressive collapse resisting capacity of steel moment frames. The progressive collapse potentials of model structures are evaluated by arbitrarily removing a center column and carrying out nonlinear static pushdown analyses using the nonlinear finite element analysis code ABAQUS. The performances of structures with partial or perforated infill panels are also studied. Then a preliminary design procedure for infill steel panels is proposed based on the equivalent single brace simplification of steel panels. The analysis results show that the infill steel panels, even the partial infill panels or panels with perforation, are effective in reducing the progressive collapse potential of moment frames. It is also shown that the proposed design procedure may be effective in preliminary design of infill steel panels to prevent progressive collapse of steel moment frames.

1. Introduction

Infill steel plates consist of vertical steel panels connected to the surrounding beams and columns and installed mainly to resist lateral load. According to previous research, they exhibit high initial stiffness and dissipates significant amounts of seismic energy. Timler and Kulak [1] and Tromposch and Kulak [2] found that the steel plate wall with unstiffened thin plates had high ductility as well as high strength even after the local buckling of the thin infill plate. Driver et al. [3] found that the infill steel panel showed a ductile behavior without brittle failure at the connection. Park et al. [4] found that to achieve large ductility, the boundary columns must resist the combined axial force and transverse force developed by the tension-field action of the infill plates. Formisano et al. [5] investigated the use of steel and aluminum shear panels as seismic retrofitting systems of existing RC structures. They concluded that the thin plates could be considered as effective strengthening devices of existing RC framed structures.

The phenomenon that local damage of structural elements results in global collapse of a structure is referred to as progressive collapse. Collapse behavior of steel moment-resisting frames caused by sudden loss of columns has been investigated by many researchers [6–8]. In this study the effect of infill steel panels on enhancing the progressive collapse resisting capacity of steel moment frames was investigated. To this end the progressive collapse resisting capacity of steel moment frame structures with infill steel panels was investigated by nonlinear static analyses using the general purpose nonlinear finite element program code ABAQUS [9]. Parametric study was performed with the thickness of infill panels varying from 2 mm to 8 mm. The progressive collapse resisting capacities of partial infill panels and panels with various rates of perforation were also studied. Finally the validity of the simplified equivalent single brace modeling of a steel panel was investigated, and a preliminary design procedure of infill steel panels to prevent progressive collapse of moment frames was developed using the simplified modeling technique.

2. Design and analysis modeling of example structures

Two-, three-, and five-story steel moment frames with a uniform story height of 3.6 m were designed as prototype structures with dead and live loads of 4.0 kN/m^2 and 2.5 kN/m^2, respectively, based on the Load and Resistance Factor Design procedure of the AISC Specifications [10]. The exterior frames were designed as moment-resisting frames and the interior frames were pin-connected to each other and to the exterior frames. Fig. 1 depicts the plan shape and elevation view of the two-story 9 m span and five-story 12 m-span analysis model structures. Only the two dimensional exterior moment frames marked in Fig. 1(a) and (c) were separated and analyzed to investigate the progressive collapse potential. Table 1 shows the sizes of the structural elements used in the design of the analysis model structures. The dimensions of the H-shaped sections are presented in the order of depth × width × web thickness × flange thickness. The columns and beams were designed using SM490

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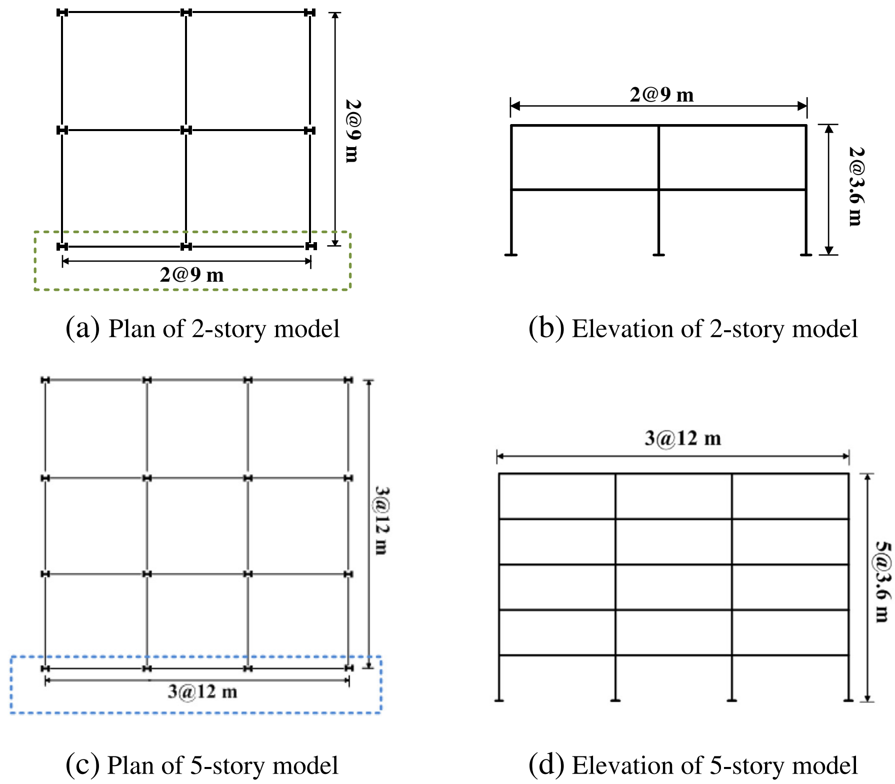


Fig. 1. Structural plan and elevation of analysis model structures.

steel (F_y = 325 MPa and F_u = 490 MPa) and SS400 steel (F_y = 240 MPa and F_u = 400 MPa), respectively. The infill steel plates were made of SS400 steel. The yield strength and the idealized stress–strain relationship obtained from the experiment of infill steel panels were used for nonlinear analysis. Fig. 2 shows the simplified stress–strain relationship of structural steel obtained from the experiments of Park et al. [4]. The Young's modulus is 2.05×10^5 MPa and the Poisson's ratio is 0.3.

In practice the connection of the panels to the boundary members is generally achieved with welds on the plate to thicker fishplates that are also welded to the boundary elements. Typical connection details are presented in Choi and Park [11]. In this study it was assumed that the infill plates were rigidly connected to the boundary elements and the fish plates were ignored in the finite element model. The infill steel panels were modeled with ABAQUS S4R elements which are four node doubly curved shell elements that accounts for finite membrane strains. The element has six degrees of freedom at each node: three displacement components and three in-surface rotation components. The cross-sectional behavior of the S4R element was integrated at five points across the thickness. The beams and columns were modeled with eight node solid elements. The effect of geometric nonlinearity and large deformation was considered by selecting NLGEOM option. The finite element analysis program ABAQUS has been successfully employed to verify the experimental results of structures with infill steel panels subjected to seismic load [4,12,13].

Table 1
Member size of model structures (depth × width × thickness of web × thickness of flange in mm).

Story	Span	Columns	Girders
2	6 m	250 × 250 × 9 × 14	400 × 400 × 8 × 13
	9 m	400 × 400 × 13 × 21	582 × 300 × 12 × 17
3	12 m	450 × 450 × 18 × 24	648 × 302 × 15 × 28
5	12 m	550 × 550 × 20 × 28	650 × 420 × 20 × 28

Purba and Bruneau [14] also investigated the behavior of unstiffened thin steel plate shear walls having a regular pattern of openings using ABAQUS.

The analysis model structures with various configurations of infill panels are shown in Fig. 3. Four types of infill steel panels were applied for analysis: (i) a full steel panel spanning a whole span; (ii) a partial infill panel located between stud columns; (iii) a partial steel plate with a height of 1/3 of the story height; and (iv) a full steel plate with various rates of perforation. Fig. 3(f) depicts the location of holes in the infill panel. Regardless of the perforation rate, the total of 73 holes were evenly distributed throughout the infill plate.

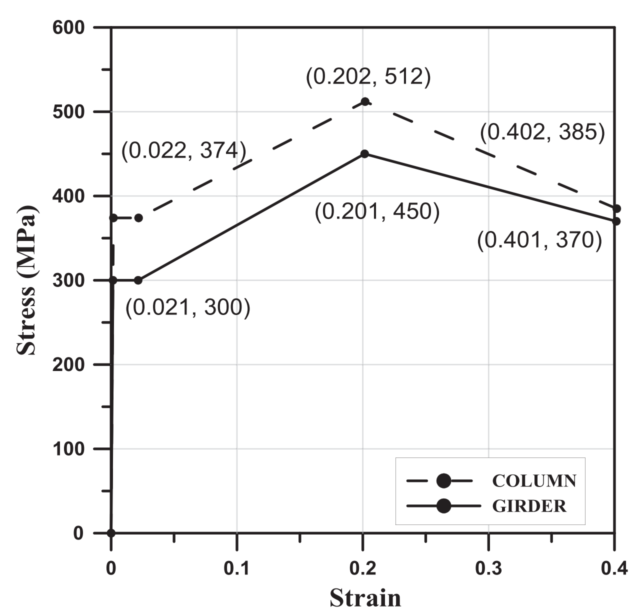


Fig. 2. Idealized stress–strain relationship for structural steel.

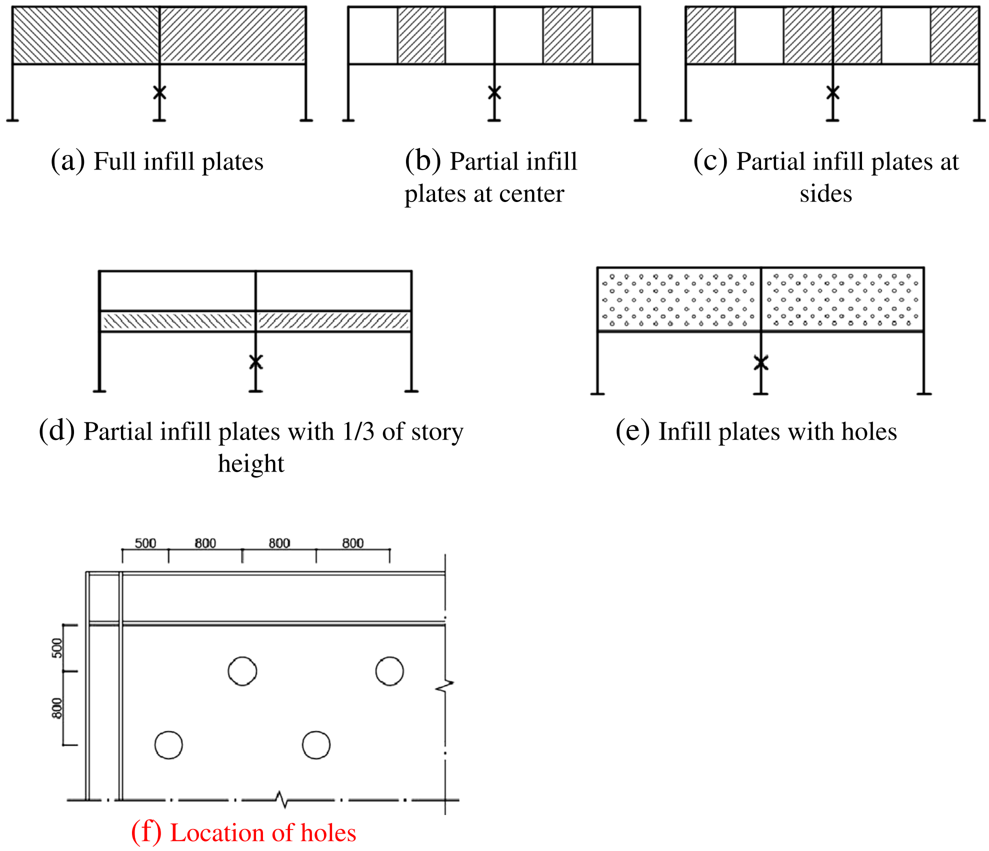


Fig. 3. Configuration of infill steel panels used in the analysis.

The diameter of a hole was increased from 107 mm in the case of 10% perforation rate to 213 mm in the 40% perforation rate. The distance between the beam or column flanges to the centers of the holes nearest to the boundary elements is 500 mm, and the center to center distance between two holes is 800 mm. In practice the partial steel panels shown in Fig. 3(b) and (c) may be applied to accommodate openings such as doors or windows, respectively. The steel panels with holes may be used to provide openings or to save steel tonnage.

For simulation of progressive collapse, nonlinear static pushdown analyses were carried out by removing the first story center column and gradually increasing the vertical displacement in the location of the removed column. For nonlinear static analysis the amplified gravity load of 2.0(Dead load + 0.25 × Live load) was applied following the recommendations of the GSA guidelines [15]. The factor 2.0 is applied to consider the dynamic effect associated with sudden removal of a column. The steel plates are expected to buckle along compressive diagonals under relatively small shear forces. After buckling, the story shear forces are resisted by the plates through formation of a tension field. To simulate the buckling of the compression diagonal in the plates, slight out-of-plane deformation in the shape of first buckling mode was provided prior to pushdown analysis.

3. Pushdown analysis results of model structures with and without infill panels

In this section pushdown analyses of the model structures with two-story 9 m span length and five-story 12 m span length were carried out to investigate their progressive collapse resisting capacity when the first story center column was removed from each structure. Fig. 4 depicts the pushdown curves of the two-story model structure installed with full infill plates with their thickness varying from 2 mm to 8 mm. The first story middle column was arbitrarily removed and the applied force, normalized by the imposed gravity load specified in the GSA Guidelines, vs. the vertical displacement relationship was plotted. The pushdown curve of the bare frame without steel panels was also plotted for comparison. It can be observed that the maximum load factor of the bare frame fails to reach 1.0, which implies that the structures lack enough strength to resist the GSA specified load when a column is suddenly removed. When full steel panels with thickness of 2 mm were installed, the maximum strength exceeded the load factor of 1.0, implying that the structure can resist the specified load combination. The strength further increased significantly as the thickness increased to 4 mm.

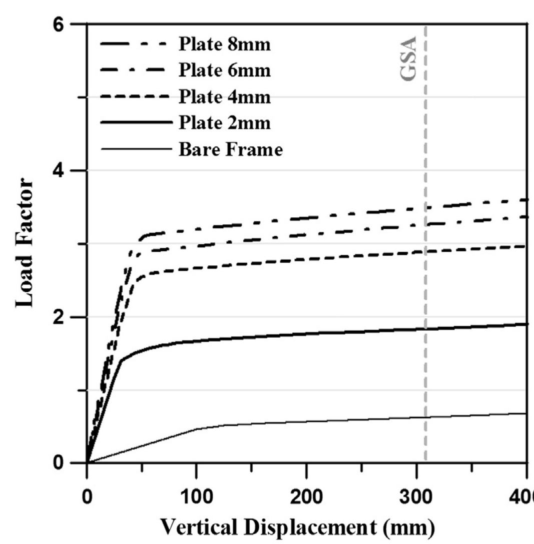


Fig. 4. Pushdown curves of the two-story 9 m span model structure with full infill panels.

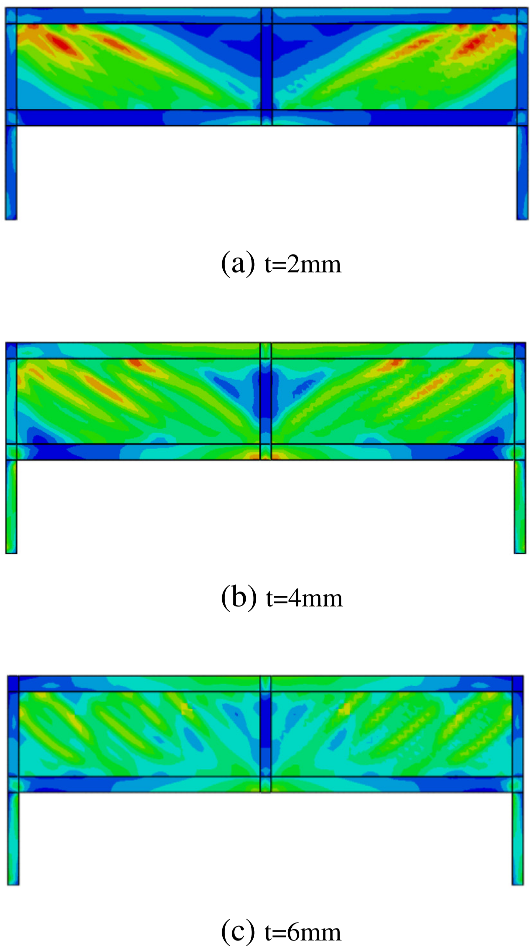


Fig. 5. Von-Mises stress contours in the infill steel plate when the vertical displacement reached 50 mm.

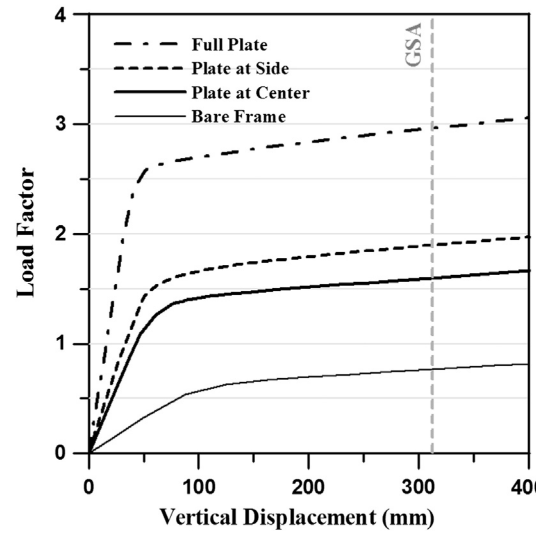


Fig. 6. Pushdown curves of the two-story 9 m span model structure with 4 mm thick partial infill panels.

However as the thickness increased to 6 mm and 8 mm the increase in strength became only minute. Fig. 5 shows the Von-Mises stress contours in the steel panels when the vertical deflection at the top of the removed column reached 100 mm. The formation of tension field is clearly visible in the infill panels. It also can be observed that as the thickness of infill plate increases the stress concentration in the steel plates decreases gradually while the stress in the columns increases.

The pushdown analysis results of the two-story structure with 4 mm thick steel plate located only at part of the span as shown in Fig. 3(b) and (c) are depicted in Fig. 6. It can be observed that the strength of the structure with partial steel plates having only 1/3 of the full width located at the center of the span is almost twice that of the bare frame and is enough to resist the progressive collapse. When two partial plates were installed at both sides of a span, the strength increased slightly; however considering the doubled amount of steel plate, the increase in strength is not significant. Fig. 7 shows the Von-Mises stress contours in the partial steel plates at the vertical deflection of 100 mm. It can be observed that the partial infill plate placed at the center of the span is more highly stressed than those placed at the sides of the span when the structure is subjected to the same vertical deflection. In addition to the possibility of providing openings for windows or doors, the partial steel infill wall may be effective in preventing progressive collapse with reduced steel. Fig. 8 depicts the pushdown curve of the 9 m span structure with partial infill plates having 1/3 of story height ($h = 1.2$ m) and those of the structure installed with full infill plates. The added horizontal beams are H-shaped sections with the size of $175 \times 90 \times 5 \times 8$ (mm). It can be observed that, even though the overall strength is significantly smaller than that of the structure with full infill panels, the maximum strength exceeds the load factor of 1.0.

In case the minimum thickness of infill plates required to prevent progressive collapse of a structure is smaller than the minimum thickness of plates produced by a manufacturer, an economic solution can be achieved by perforating holes in the infill plates and thus reducing overall weight [16]. Fig. 9 shows the pushdown curves of the two-story 9 m span model structure installed with 4 mm thick steel infill plates with regularly spaced circular holes. It can be observed that the strength of the model structure keeps decreasing as the perforation rate increases. However even in the structure with infill panels having 40% perforation rate, the yield strength exceeded the imposed load specified in the GSA Guidelines. Fig. 10 shows the Von-Mises stress contours in the steel plates with holes when the vertical displacement reached 10 cm. It can be observed in Fig. 5(a) that stress is concentrated along the tension fields when there are no holes in the plates. In the infill plates with holes, the formation of tension field is not as distinct as in the infill panels without holes, especially when the perforation rate increases to 40%.

Fig. 11 shows the pushdown curves of a 5-story 12 m span model structure with and without infill steel plates when a first story interior column was removed. It can be seen that the progressive collapse resisting capacity of the bare frame is not satisfactory as in the case of the two-story structure analyzed previously. When 2 mm thick infill panels were placed only in the second story, the maximum load factor at the vertical displacement specified in the GSA guidelines as the limit state did not reach 1.0. The maximum load factor reached 1.0 when the steel panels were installed in the second and the fourth stories. Fig. 12 depicts the Von-Mises stress contours in the infill panels installed in the five-story structure at the vertical displacement of 100 mm. The formation of tension field can be observed in the panels located in the spans from which a column is removed.

4. Simplified modeling of infill plates

For analysis of a structure with infill steel plates, the finite element modeling may produce the most accurate results. However as sophisticated modeling technique and significant amount of computation time are required in the finite element modeling and analysis, simplified modeling techniques such as a strip model or an equivalent single brace model are frequently used for structural analysis and design of structures with infill steel plates. In the strip model, the

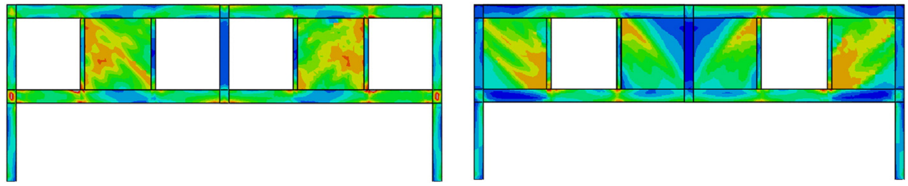


Fig. 7. Distribution of stress at partial steel plates when vertical displacement reached 100 mm.

steel plate is replaced by a series of diagonal elements called strips [1,17]. Another simplified model for an infill steel plate, an equivalent single brace model, was proposed by Thorburn et al. [18]. In this model a steel plate is modeled as a single diagonal brace with an equivalent property to the steel plate. Using the elastic strain energy formulation, they derived the area of the equivalent brace based on fully developed tension field theory as follows:

$$A = \frac{th \sin^2 2\alpha}{2 \sin\phi \, \sin 2\phi} \quad (1)$$

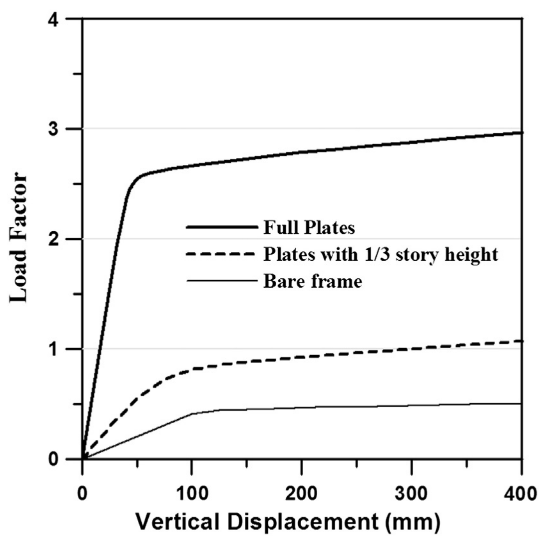


Fig. 8. Pushdown curves of the two-story frame installed with partial steel plates with 1/3 of story height.

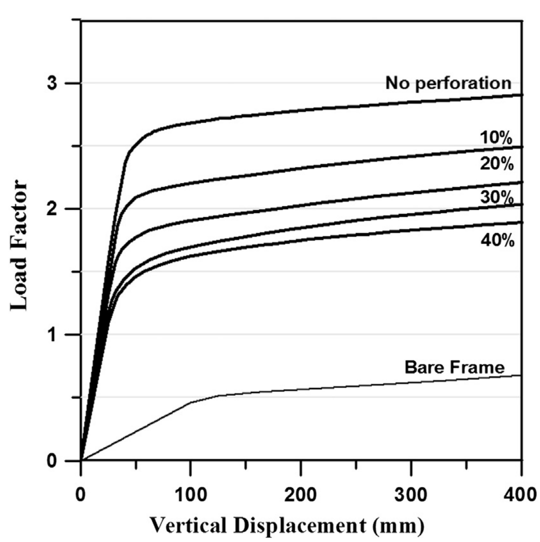


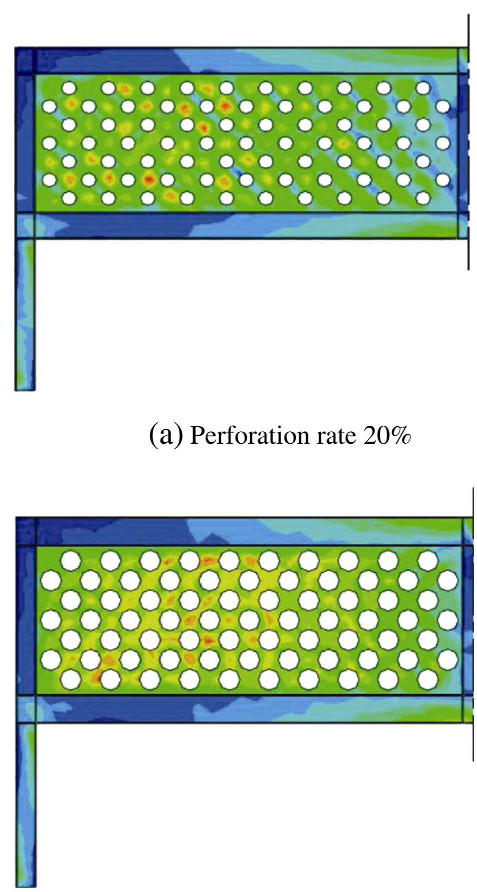
Fig. 9. Pushdown curves of the two-story structure with 4 mm thick infill plates with various perforation rates.

where t is the thickness of a steel plate, h is the story height, ϕ is the angle between horizontal axis and the diagonal tension brace, and α is the inclination angle of the tension field to the horizontal axis. For progressive collapse, the inclination angle of the tension field to the horizontal axis, α, is obtained as follows based on the principle of least work [1]:

$$\tan^4\alpha = \frac{1 + \frac{th}{2A_b}}{1 + tL\left(\frac{1}{A_c} + \frac{L^3}{360 I_b h}\right)} \quad (2)$$

where L is the center-to-center distance between the boundary columns; A_b and A_c are the cross-sectional areas of the beam and the column, respectively; and I_b is the moment of inertia of the boundary beam.

Fig. 13 depicts the pushdown curves of the two-story 6 m and 9 m span model structures with infill steel panels modeled by finite



(a) Perforation rate 20%

(b) Perforation rate 40%

Fig. 10. Von-Mises stress contours of steel infill plates with perforations at the vertical displacement of 100 mm.

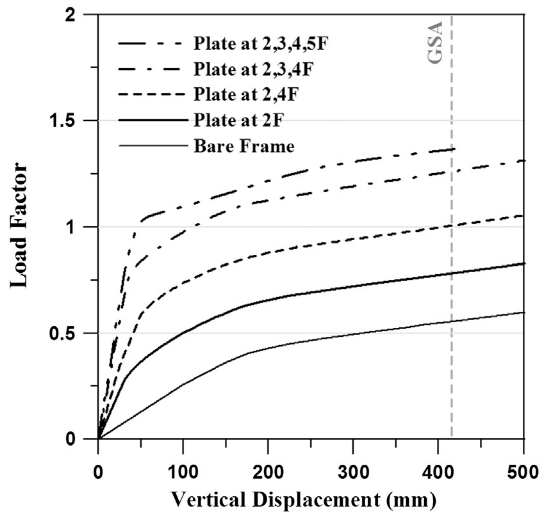


Fig. 11. Pushdown curves of five-story model with and without steel plates.

elements and the equivalent single brace. Infill steel plates with thickness of 2 mm and 4 mm were placed in the 6 m and 9 m span structures, respectively. It can be observed that the overall vertical stiffness and strength of the 6 m span structure with infill panels modeled by the equivalent single brace are similar to those obtained by FE modeling of the steel panels. However the vertical stiffness of the 9 m span structure, in which the aspect ratio of the steel panel is higher, turned out to be significantly smaller when obtained by the simplified model. The strength of the 9 m span structure computed by the simplified model is more or less smaller than that of the model structure obtained by the FE analysis; however the difference is gradually reduced as the vertical displacement increases. The increased difference between the two modeling methods in the structure with longer span length may be due to the fact that as the span length increases, that is as the aspect ratio of the infill panel exceeds a certain point, the representation of the behavior of steel panels with only the inclined tension field may become less precise. However as the difference in the ultimate strength is relatively small and is less affected by the change in span length, a preliminary design process of infill panel to prevent progressive collapse was developed using the equivalent single brace model in the following section.

5. Design of steel plates to prevent progressive collapse

In this section a preliminary design procedure for infill steel panels was derived using the equivalent single brace modeling of the infill

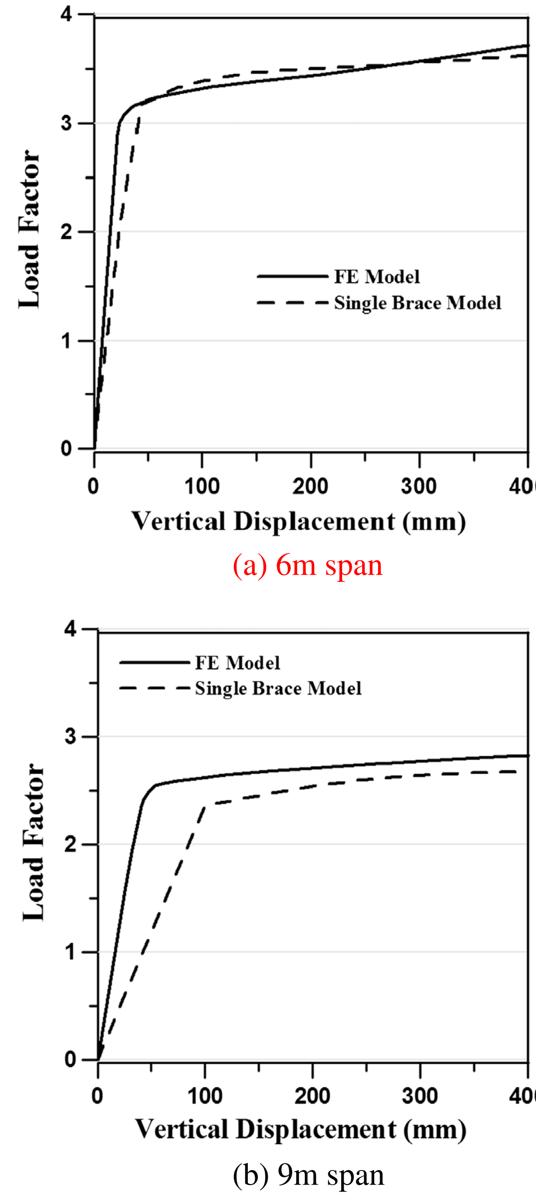


Fig. 13. Pushdown curves of the two-story model structure with infill steel plates obtained by the FE analysis and the equivalent single brace analysis.

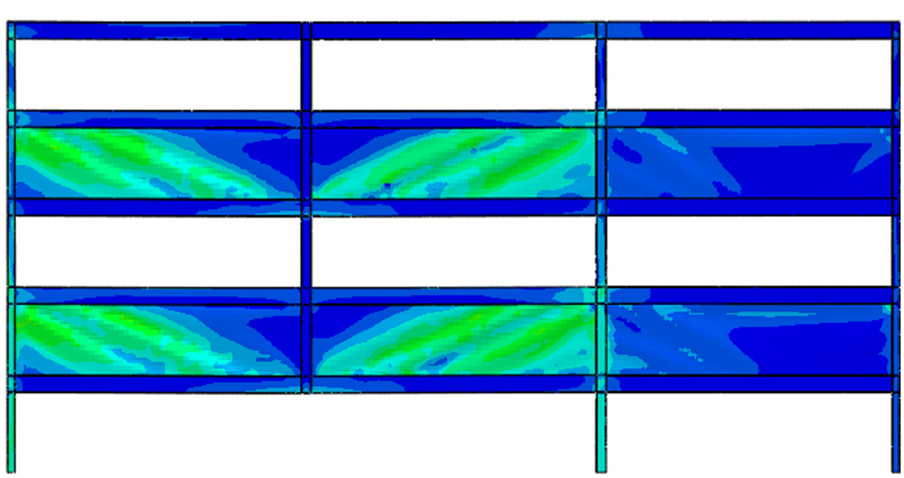


Fig. 12. Von-Mises stress contours in the five-story structure at vertical displacement of 100 mm.

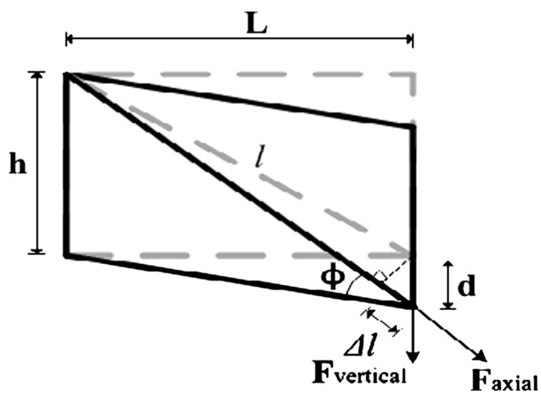


Fig. 14. Relation of vertical deflection and element force of an equivalent single brace.

panels. The design procedure was applied to the three-story 12 m span model structure and the validity of the procedure was investigated by finite element analysis. The design objective for retrofit is to determine the required thickness of steel panels to achieve the maximum load factor of 1.0 when a column is removed. This implies that the structure is retrofitted using infill panels in such a way that it can resist the imposed load 2(DL + 0.25LL) specified in the GSA guidelines. Fig. 14 shows the configuration of an equivalent single brace subjected to vertical displacement d at one side. From the equivalence between the vertical deflection of the plate and the elongation of the equivalent diagonal brace, the vertical force acting on the equivalent single brace subjected to vertical displacement d can be obtained using the following relationship:

$$F_{vertical} = \frac{EA\sin^3\phi}{h}d \tag{3}$$

where E is the modulus of elasticity, A is the cross-sectional area of the equivalent brace, h is the height of the steel plate, and d is the vertical deflection for which the collapse limit state specified in the GSA guidelines can be used. The rotation limit state for moment frames specified in the GSA guidelines is 0.035 rad. The thickness of the steel plates is determined in such a way that the total strength of the plates at a limit state is equal to the difference between the global strength of the bare frame and the target strength of the retrofitted frame, which is the GSA specified imposed load. To achieve this design objective it is required that the vertical strength of the equivalent brace is equal to the required strength of the system:

$$F_{\text{vertical}} = \frac{nEA\sin^3\phi}{h}d = \Delta Load \tag{4}$$

where n is the number of steel plates installed in the bays from which a column is removed, and the required strength $\Delta Load$ is obtained

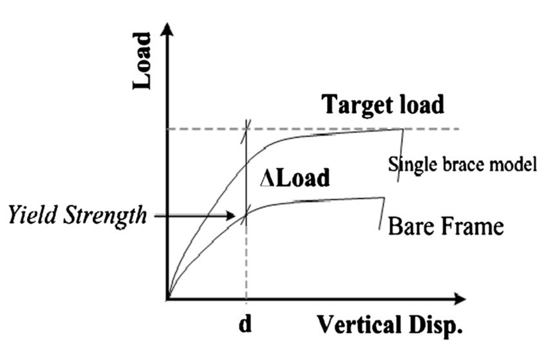


Fig. 15. Target strength of the frame for design of infill steel panels.

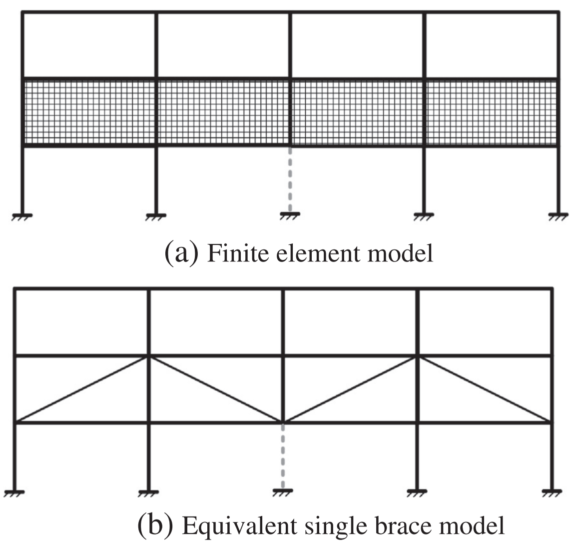


Fig. 16. Configuration of three-story analysis model structure with infill panels for validation of simplified modeling of infill steel panels.

from the pushdown curve as depicted in Fig. 15. From Eqs. (1) and (4), the required area of each equivalent brace can be obtained as follows:

$$A = \frac{(\Delta Load)h}{ndE\sin^3\phi} = \frac{th\sin^2 2\alpha}{2\sin\phi\cdot\sin 2\phi}. \tag{5}$$

Then the required thickness of each infill plate to prevent progressive collapse is computed as:

$$t = \frac{2(\Delta Load)\sin 2\phi}{ndE\sin^2\phi(\sin^2 2\alpha)}. \tag{6}$$

The above procedure for design of infill steel plates was applied to the three-story 12 m span structure shown in Fig. 16. The sizes of the structural members were shown in Table 1. According to the pushdown curve of the bare frame shown in Fig. 17, the structure is vulnerable for progressive collapse when a first-story center column is removed. To enhance progressive collapse resisting capacity, infill steel plates were to be installed in the second story. From the pushdown curve of the bare frame and the given target strength, the

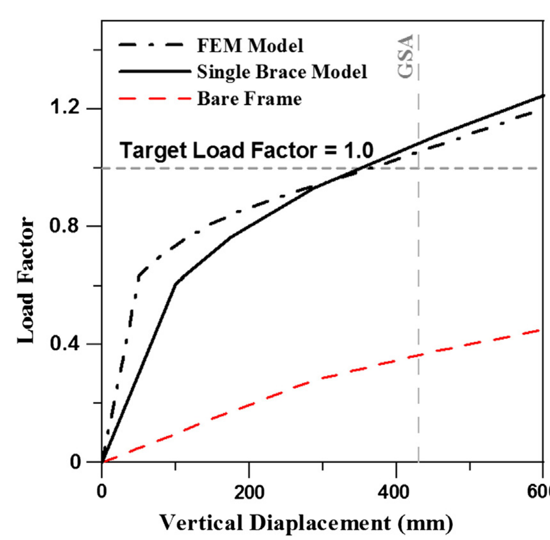


Fig. 17. Pushdown curves of the three-story model structure with infill plates designed using Eq. (6).

thickness of infill steel plates required to achieve the maximum load factor of 1.0 was computed using Eq. (6). In order to reach the target load factor, the thickness of the infill plates was computed as 4.4 mm. The model structure was retrofitted with the 4 mm-thick steel infill plates in the second story and was analyzed with their first story center column removed. Fig. 17 shows the pushdown curves of the model structure before and after the retrofit. The steel plates in the retrofitted structures were modeled by both the finite and the equivalent single braces in the pushdown analysis. It can be observed that the maximum load factor of the model structure retrofitted with steel plates reached the target load factor of 1.0 at the displacement specified in the GSA Guidelines as a limit state. Even though the stiffness of the structure predicted by the equivalent single brace model is somewhat smaller than that obtained by the FE analysis, as also observed in Fig. 14 in the case of the two-story 9 m span structure, the maximum strength of the structure generally corresponded well with that of the FE model.

6. Conclusions

In this study the effect of infill steel panels on enhancing progressive collapse resisting capacity of steel moment frames was investigated, and a simple design procedure for infill steel panels was proposed to ensure safety against progressive collapse caused by sudden removal of a column. The progressive collapse potentials were evaluated based on arbitrary column removal scenario. The accuracy of the equivalent single brace modeling techniques of steel panels was investigated in comparison with the analysis results of finite element modeling.

The analysis results showed that the infill steel panels were effective in reducing the progressive collapse potential of moment frames caused by sudden removal of a column. It was observed that as the thickness of the steel panels increased the progressive collapse resisting capacity also increased. However when the thickness of the steel panels increased higher than a certain level the increase in the progressive collapse resisting capacity did not increase proportionally because of yielding of columns. Even the partial infill panels or panels with perforation were somewhat effective in protecting the structures against progressive collapse. The simplified modeling of steel panels utilizing an equivalent single brace generally corresponded well with the finite element model, and the preliminary design procedure of steel panels using the single brace model turned out to be effective in estimating the minimum thickness of steel panels required to ensure safety against progressive collapse.

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