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Evaluation of vierendeel peripheral frame as supporting structural element for prevention of progressive collapse

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Abstract. Progressive building collapse occurs when failure of a structural component leads to the failure and collapse of surrounding members, possibly promoting additional failure. Global system collapse will occur if the damaged system is unable to reach a new static equilibrium configuration. The most common type of primary failure which led to the progressive collapse phenomenon, is the sudden removal of a column by various factors. In this study, a method is proposed to prevent progressive collapse phenomena in structures subjected to removal of a single column. A vierendeel peripheral frame at roof level is used to redistribute the removed column's load on other columns of the structure. For analysis, quasi-static approach is used which considers various load combinations. This method, while economically affordable is easily applicable (also for new structures as well as for existing structures and without causing damage to their architectural requirements). Special emphasis is focused on the evolution of vertical displacements of column removal point. Even though additional stresses and displacements are experienced by removal of a structural load bearing column, the proposed method considerably reduces the displacement at the mentioned point and prevents the collapse of the structural frame.

Keywords: steel moment frames; progressive collapse; structural failures ; vierendeel frame; column removal

1. Introduction

Progressive collapse is defined as the spread of an initial local failure from element to element eventually resulting in collapse of an entire structure or a disproportionately large part of it (Izzuddin *et al*. 2008 and Fu 2010). The potential abnormal loads that can trigger progressive collapse are categorized as: aircraft impact, design error, construction error, fire, gas explosions, accidental overload, hazardous materials, vehicular collision, bomb explosions, etc. (Ellingwood *et al*. 2007). The interest in the field of progressive collapse has greatly increased after the collapses of the World Trade Center in New York in 2001 and the Alfred P. Murrah building in Oklahoma in 1995 (Wang *et al*. 2015). In the United States, the Department of Defense (Unified Facilities Criteria (UFC)-DoD 2007) and the GSA (The U.S. General Services Administration 2003) provide detailed guidelines regarding methodologies to resist progressive collapse of building structures. Both employ the alternate path method (Fu 2012). In this approach, the structure is designed such that if one component fails, alternate paths are available for the load and general collapse does not occur and a building structure be able to tolerate loss of any one column without collapse (Kim and Kim 2009). Kim and Park (2008) evaluated the progressive collapse potential of two special moment resisting frames by two analysis procedures: nonlinear static and nonlinear dynamic. A simple plastic design method for design of steel moment-resisting structures to prevent progressive collapse was presented upon which the plastic moment of beams required to stabilize a structure subjected to sudden loss of a column was computed from the equilibrium condition of the external work done by gravity load and the internal work done by plastic rotation of beams. It was shown that the increase of only the girder size for the purpose of preventing progressive collapse may result in weak story when the structure is subjected to seismic load. The formation of weak story can be prevented by increasing the column size in such a way that the strong column-weak beam requirement is satisfied. The nonlinear dynamic analyses results showed that the structures designed without considering progressive collapse did not satisfy the failure criterion required by the GSA guidelines; on the other hand, the structures redesigned by plastic design method to prevent progressive collapse turned out to satisfy the given failure criterion in most of the model structures.

Most of the published progressive collapse analyses are based on alternative load path (Ap) method with sudden column loss as recommended in mentioned guidelines. In this approach, the structure is designed such that if any one component fails, alternate paths are available and a general collapse does not occur. In most cases design for redundancy requires that a building structure be able to tolerate loss of any one column without collapse. In most of the published numerical studies of progressive collapse, open source or commercial nonlinear FE packages are used, such as Abaqus (Usmani *et al*. 2003 and Tavakoli and Kiakojouri 2013). In this study, current approaches for dynamic column removal is discussed in details and a

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widely used method for removing columns will be used. Special emphasis is focused on the evolution of vertical displacements of column removal point. Considering the consequences of the removal of a structural load-bearing column, a solution is presented that favorably reduce the displacement of the mentioned point significantly and prevent the collapse of the entire structure. Recently Dusenberry and Hamberger (2006) explained the phenomenon of progressive collapse based on energy balance concept and mentioned that the energy-based method have strong potential to be developed into simplified procedures for collapse potential assessment. The methods presented so far have problems, including that they are not economically justified or change the expected structural ductility as a result of increase the girders and columns size. Arshian and Morgental (2017) presented a sequential nonlinear time-history analysis algorithm alongside with a macro modeling approach to predict the dynamic redistribution of the gravity loads. The efficiency of such a numerical framework was verified through comparison of computational results with the available experimental data from a past 3D half-scale test. Good agreement was observed for the global and for the local response quantities. Furthermore, a practical strengthening technique was applied into the computational model of the structural system for artificially activating the catenary mechanism. Analysis results showed that strengthening of peripheral beams with externally bonded steel plates significantly increases the rotational ductility at beam sections and in turn, enables the damaged structure to accommodate larger deformations. Seethalakshmi *et al.* (2016) investigate the comparative behaviour of four bay, five storey RC bare frame, infilled frame and infilled frame with openings to assess the effect of infill to resist the progressive collapse. A linear static analysis was carried out and maximum moment, shear force, axial force and deflection for both beams and columns generated before and after middle column removal were studied and compared. There is an average of 30% and 34% decrease in moments for infilled frames when compared to a bare frame. The percentage of decrease in moments increases to an average of 71% when the column is removed. The results showed that the presence of infilled frames will delay the progressive collapse when compared to bare frames. Nezamisavojbolaghi *et al.* (2017) presented an appropriate procedure for numerical modeling of infills in steel asymmetric moment frames in progressive collapse analysis. The proposed procedure has been applied to a series of 6 story moment frames with different span size, once without the infills and once more with them. Various scenarios have been considered for column removal to trigger the progressive collapse. Results of dynamic analysis show that the amount of vertical deflection of the studied frames with contribution of infills is significantly less than those obtained without infills.

The effect of viscous dampers on reducing progressive collapse potential of steel moment frames was evaluated by nonlinear dynamic analysis and according to the parametric study, the vertical displacement generally decreased as the damping ratio of the system increased. The analysis results showed that the viscous dampers, originally designed to reduce earthquake-induced vibration, were effective in reducing vertical displacement of the structures caused by sudden removal of a first-story column, and the effect was more predominant in the structure with longer span length (Kim *et al*. 2010). The time length required for disabling the failed column is defined as the rise time. A time length less than one tenth of the column-loss vibration period is suggested by some guidelines for disabling the failed column in the numerical analysis. The external loading for the progressive collapse analysis grows within a finite rise time. Increase of rise time can lead to a larger collapse resistance under a specific ductility demand or a smaller ductility demand under a specific normalized loading (Tsai 2017). Rezvani and Asgarian (2014) investigated the effect of seismic design level as a practical approach for progressive collapse mitigation and reaching desired structural safety against it in seismically designed concentric braced frame buildings. By sensitivity analysis it became possible to introduce the equation of structural safety against progressive collapse for concentrically braced frames.

Up to now, most researchers have investigated the progressive collapse due to explosion, fire or impact loads. But new research has shown that the seismic load could also be a factor for initiation of the progressive collapse. The plastic deformation of ductile beams is characterized by strain hardening, which is responsible for the development of bending moments larger than the plastic bending strength (Güneyisi *et al*. 2014). Therefore, according to hierarchy criteria, nondissipative elements (namely connections and columns) should be designed to resist the maximum bending moment experienced by the beams. Cassiano *et al.* (2016) investigated effectiveness of seismic detailing for steel Moment Resisting Frames (MRF) in limiting the progressive collapse under column loss scenarios. With this regard the following cases were examined: (i) MRF structures designed for wind actions according to Eurocode 1; (ii) MRF structures designed for seismic actions according to Eurocode 8. Results show that structures designed according to capacity design principles are less robust than wind designed ones.

The progressive collapse potential of steel moment framed structures due to abrupt removal of a column was investigated based on the energy principle (Chen *et al.* 2016). A simplified beam damage model was proposed to analyze the energies absorbed and dissipated by structural beams at large deflections, and a simplified modified plastic hinges model was developed to consider catenary action in beams. Kordbagh and Mohammadi (2017) investigated the influences of building height and seismicity level on progressive collapse resistance of steel special moment resisting frames buildings. The obtained results indicated that taller buildings are safer against progressive collapse and the structures designed for greater seismic base shears are more resistant against progressive collapse.

2. Applied approach for dynamic column removal

Some studies have been carried out for investigating the rise-time effect on the structure response under column loss.

It has been indicated that the time length for completely removing the column can affect the structural response and the rate of removal is insignificant in the final results as the rise time is less than 10 ms (Liu *et al*. 2005). Several factors led to the removal of columns and dynamic nature of this phenomenon for different scenarios of remove columns must be appropriately considered. To carry out dynamic analysis, different approaches are suggested by many researchers. These approaches are categorized into two main groups; either direct element deletion and reaction approaches. Direct element deletion is not applicable in all FE packages and if possible additional subroutine is required. In so-called reaction approaches, the reaction forces acting on a column are computed before its removal and then the column is replaced by concentrated loads equivalent to its forces (Ellingwood *et al*. 2007). As a third method, Tavakoli and Kiakojouri (2013) have proposed a method called material degradation. In present study, the second method as shown in Fig. 1 will be used. The loads increased linearly for 5 seconds until they reached their full amounts, kept unchanged for 2 seconds, and the concentrated forces were suddenly removed at seven seconds to simulate the column removal (Ellingwood *et al*. 2007).

2.1 Numerical study

In this study finite element analysis is performed using the general purpose finite element package Abaqus/Explicit version 10.6. Explicit dynamics is a mathematical technique for integrating the equations of motion through time. The explicit dynamic integration method is also known as the forward Euler or central difference algorithm. Unknown values are obtained from information already known. Combining the explicit dynamic integration rule with elements that use a lumped mass matrix is what makes an explicit finite element program work. The explicit procedure performs the analysis using a large number of inexpensive, small, time (load) increments.

2.2 Analytical model

The model structure is a 2D five story steel moment resisting frame, the floor height is 3.2 m and span length is 5 m (Fig. 2). This steel moment frame is designed to resist both gravity and lateral loads due to strong earthquake according to AISC 2010 code. Modeling details are available

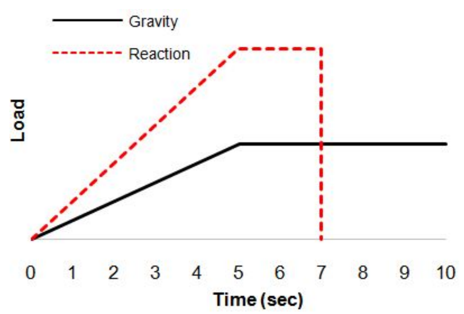

Fig. 1 Reaction approach, sudden column removal and linear increased gravity load

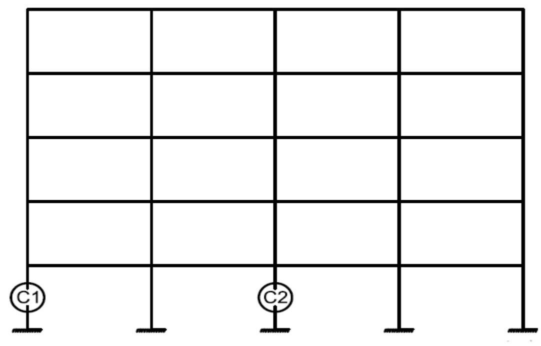

Fig. 2 Elevation of model and column removal cases

in (Tavakoli and Kiakojouri 2013 and Ellingwood *et al*. 2007). Two conditions for column removal are considered. In the first case, column 1 and in the second case, column 2 is removed. At this stage, vierendeel peripheral frame at roof level redistributes the removed column's load on other columns of the structural frame.

For evaluation of behavior, changes in vertical displacement versus time in second for top end of the removed columns are drawn and the performances are compared.

3. Vierendeel frame

The vierendeel frame, or truss as it is more popularly but inappropriately called, is a series of rectangular frames which achieves stability by the rigid connection of the vertical web members to the top and bottom chord. Contrary to the typical pin-connected truss in which all members are axially loaded and shear is transferred axially through diagonals, the vierendeel transfers shear from the chords by bending moments at the joints and finally by bending moments in the vertical webs. As a result, all members are combined stress members in which axial, shear, and bending stresses exist (Fig. 3). The vierendeel's origin dates back to 1896. The Belgian engineer Arthur Vierendeel, then professor at the University of Louvain, unveiled the concept in his book Longerons en Treillis et Longerons a Arcades (Wickersheimer 1976). A vierendeel frame used in the construction of a bridge is illustrated in Fig. 4. At first little appears to be gained by this system. The vierendeel frame will be heavier than an equivalently loaded truss. Even though the diagonals are eliminated, bending in all members results in chord sizes and vertical webs significantly larger in cross-sectional area. Shop fabrication of the gussets is usually complicated without again increasing member sizes, or the system's depth. As in most structural systems, the vierendeel gains tremendous rigidity with increased depth. At that time steel trusses required extremely large gusset plates to accommodate rivet groups; members were generally oversized and rarely did the center lines of all joined members intersect.

Therefore, the pin-jointed theory which ignored moments, due to these eccentricities, led to errors on the critical side, approaching fifteen percent when office calculations were compared to field measurements. Methods and reality are what led Professor vierendeel to

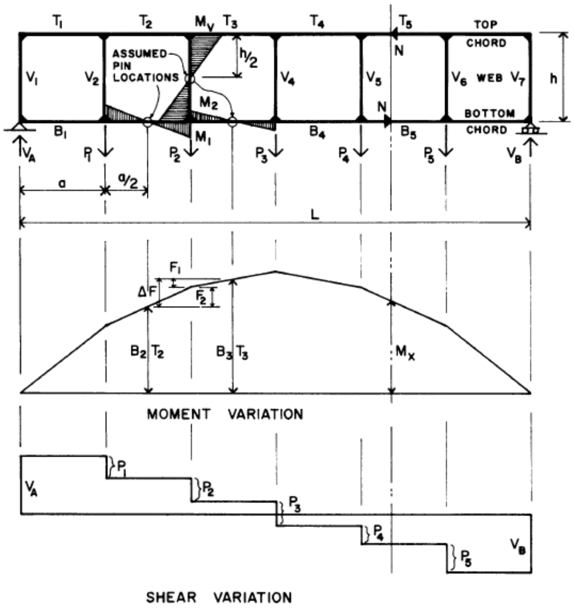

Fig. 3 Vierendeel analysis

Fig. 4 Vierendeel frame used in the construction of a bridge

propose the rectangular rigid-jointed system where these eccentricities could be eliminated and accuracy between analysis and reality kept in close accord. A smaller factor of safety could be used, due to this improved accuracy, so that in the early I9oos vierendeel bridges did weigh less than alternative truss solutions.

4. Analysis and discussion

The adopted material properties used here, were: Young's modulus, E = 210 GPa, Poisson ratio, u = 0.3, and density ρ = 7850 kg/m^3 The static yield stress was f_y = 240 MPa and the plastic properties of steel materials presented in Fig. 5.

The geometric dimensions of the case study frame sections, are shown in Fig. 6 and their corresponding values are presented in Table 1. Box sections are used for columns and the beams are made of I-shaped sections. Nonlinear dynamic analysis is performed using the general purpose finite element package Abaqus/Explicit version 10.6 in order to evaluate the effect of the vierendeel frame on the structural response (i.e., the vertical displacement of the top of the removed column).

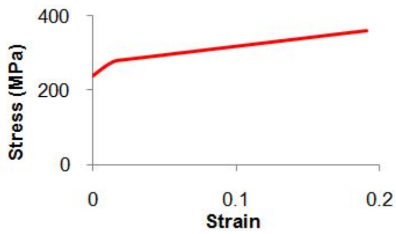

Fig. 5 plastic properties of steel materials

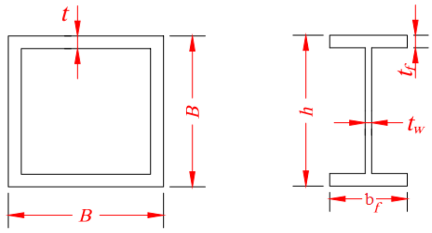

Fig. 6 Geometric dimensions of the frame sections

Table 1 Columns and beams sections (all units are in cm)

Story	Column (box) $(B\times B\times t)$	Beam (H) $(h\times b_f\times t_f\times t_w)$
1	25×25×2.3	45×19×1.46×0.94
2	25×25×1.9	45×19×1.46×0.94
3	25×25×1.6	45×19×1.46×0.94
4	20×20×1.6	40×18×1.35×0.86
5	20×20×1.3	40×18×1.35×0.86

As one of the perimeter frames of the structure, the distributed load on the beams (proportional to the tributary areas) is considered as

$$(1.2Dead+1.6Live)*w_{eff}$$
$$=(1.2*500+1.6*200)*2.35=21.6\,KN/m,$$

Where w_{eff} is half the length of the span perpendicular to the frame. As an initial trial, a vierendeel frame made with IPE140 sections with a height (d) of 70 cm where the distance between the vertical frame members (s) considered to be one meter (d and s as shown in Fig. 13, respectively) and will be shown that although the dimensions of the vierendeel frame were selected based on engineering judgment, for this case, proposed height is closed to the optimum height required. When the corner column in first story was removed suddenly (case 1), the node on the top of the removed column vibrated and reached a maximum vertical displacement of 101 mm. For case 2, when the middle column in first story was removed suddenly, the node on the top of the removed column vibrated and reached a peak vertical displacement of 78 mm. Based on these results, it can be understood a two-dimensional frame is suffering more damage by removal of corner column. Structural model stability and load carrying capacity, after removing the corner column has been maintained by presence of vierendeel frame and results of analysis in form of stress contours is presented in Fig. 7.

Vertical displacement changes of the point at top of the removed column versus time is presented (Figs. 8 and 10) and in order to make better comparison, the results for both resistant and non-resistant structure are plotted on a graph. Also, free vibration of the structure for 7 seconds after removing the column is shown.

As seen in Fig. 8, while reducing vertical displacement by 52 mm (Reduction in the amount of displacement in this case is (101-52)/101 = 48.5%), severe vertical vibration of the check point of the study (as in real condition, due to fatigue phenomenon, has an effect on the structural connections like what happened in the Northridge earthquake (1995)), after retrofitting the frame, has completely removed.

After the mentioned earthquake, connections design criteria, fundamentally changed. Before removing the column, vierendeel frame is free of stress and is designed

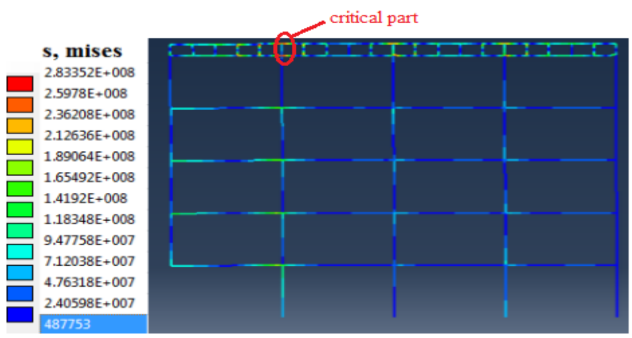

Fig. 7 Stress contour after column removal

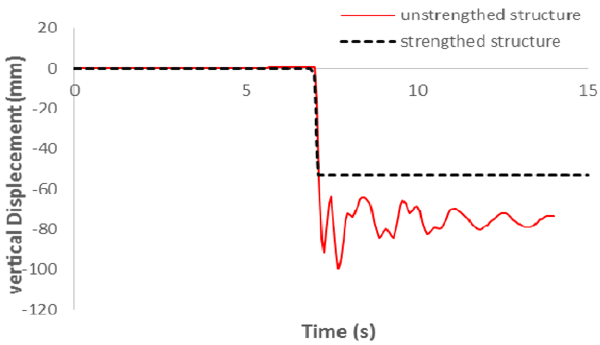

Fig. 8 Vertical displacement of the point at top of the removed exterior column versus time

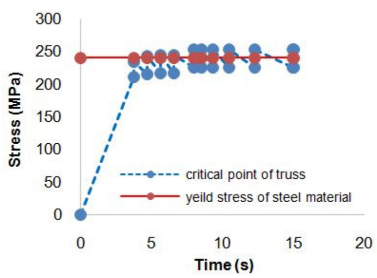

Fig. 9 Stress in critical part (which is shown in figure 6) of vierendeel frame

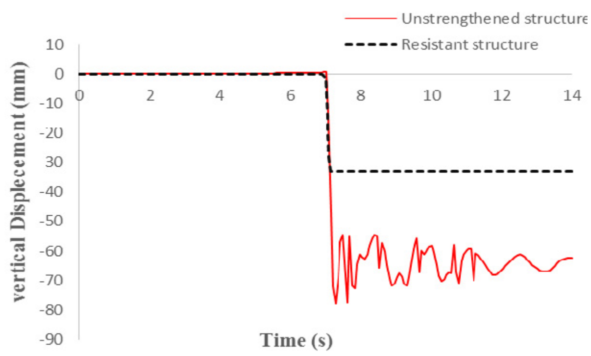

Fig. 10 Vertical displacement of the point at top of the removed middle column versus time

for impact load that occurs at the time that column is removed suddenly. If the removed column belongs to the first floor, the amount of impact load will be the maximum possible (Due to maximum gravity load). Now considering the yield stress of 240 MPa for all the structural members, the stresses in the most critical part of the vierendeel frame, during and after removal of column is plotted in Fig. 9. In this case, the critical stresses in the vierendeel frame go slightly beyond the yield stress. There is an increase of about 10 percent and to overcome this problem, height of the vierendeel frame could be slightly increased.

4.1 Removing a middle column

If one of the middle columns of the structure removed suddenly, it is expected that the redistribution of the load on other columns by vierendeel frame, be better than the case in the previous paragraphs. For this case, vertical displacement of the point at top of the removed middle column versus time is presented in Fig. 10.

Reduction of the amount of vertical displacement in this case is (77-33)/77 = 57%. Also, because of the vierendeel frame's high stiffness, severe vertical vibration at the check point of the study as stated earlier, after retrofitting the frame, has completely removed.

Although the height of vierendeel frame, is selected on an empirical basis, however the result (Fig. 11), shows that height of the vierendeel frame, according to the resistance criteria, is approximately the optimal height.

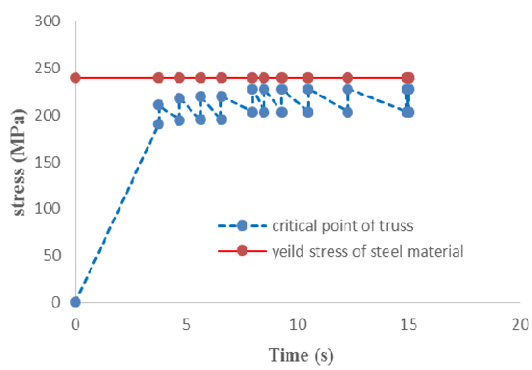

Fig. 11 Stress in critical part of vierendeel truss frame while middle column is removed

(a) Side column removal (b) Middle column removal

Fig. 12 Column removal scenarios

5. The optimum height of the vierendeel frame according to the acceptance criteria

In nonlinear dynamic analysis of column removal, most reputable codes, specify maximum plastic hinge rotation (Which obtained by dividing the maximum displacement to the length of the member) as a limit value. The acceptance criterion for plastic hinge rotation of steel beam and column (flexural member) is 0.02 in radian. After removing columns in the first or second scenario, as shown in Fig. 2, considering the impact of the sudden column removal, vierendeel peripheral frame behaves as two cantilever beams with common end point (Fig. 12(a)) or a continuous beam (Fig. 12(b)), respectively under concentrated load equivalent to the removed column's load. Assuming an average width of structure's spans by s_{avr} , number of floors by n, average gravity load on the studied column from each floor by w_{avr}, the equivalent concentrated load can be calculated as p_{eq} (Eq. (1)).

$$p_{eq} = s_{avr}.w_{avr}.n \tag{1}$$

In this case, the maximum gravity load is applied to the vierendeel peripheral frame. If the *i*'th story column is removed, n variable in Eq. (1), must be replaced by (*n*–*i*). Using the structural analysis equations, maximum vertical displacement (Δ_v) of the outlined points in Fig. 12 can be calculated as follow

$$\Delta_v = ie \times \max \begin{cases} (\dfrac{p_{eq}.s^3{}_{avr}}{6EI_{vfe}}) & \text{Fig. 12(a)} \\ (\dfrac{p_{eq}.(2s_{avr})^3}{192EI_{vfe}}) & \text{Fig. 12(b)} \end{cases} \tag{2}$$

In which, I_{vfe} is the required equivalent moment of inertia of the vierendeel peripheral frame (as given in Eq. (3)) and *ie* is to consider the impact effect.

$$I_{vfe} = 1.2\left(2I + 2A\left(\frac{d}{2}\right)^2\right) \tag{3}$$

In Eq. (3), *I* is the moment of inertia of the section about its main axis which is used in assembling of vierendeel frames, *d* is shown in Fig. 13 and *A* is cross sectional area of I section. By considering the allowable beam plastic rotation as the vierendeel frame design criterion, optimal height of the vierendeel frame is determined as follow

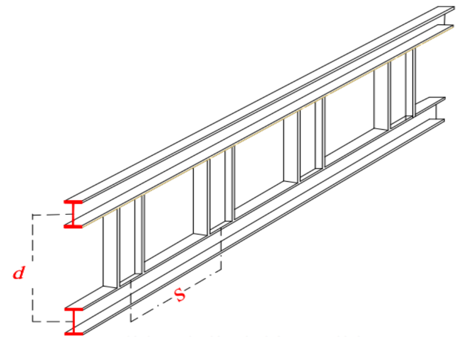

Fig. 13 Vierendeel frame dimentions

$$\frac{\Delta_v}{s_{avr}} < \theta_{p(allowable)} \tag{4}$$

For example, as is carried out in this study if the vierendeel frame is built with IPE140 sections, assuming that gravity load on beams is 21.6 KN/m, it's optimal height (considering impact effect, *ie* = 1.5) is calculated as follow

$$p_{eq} = s_{avr}.w_{avr}.n = 5\times 21.6\times 5 = 540KN$$

$$I_{vfe} = 1.2\times\left(2\times 541 + 2\times 16.4\times\left(\frac{d}{2}\right)^2\right) = 1298.4 + 9.84d^2$$

$$\Delta_v = 1.5\times\max\begin{cases} \dfrac{540\times 500^3}{6\times 20594\times I_{vf}} = \dfrac{819410}{I_{vf}} \\ \dfrac{540\times(2\times 500)^3}{192\times 20594\times I_{vf}} = \dfrac{204850}{I_{vf}} \end{cases} = \frac{819410}{I_{vf}}$$

$$\frac{\Delta_v}{s_{avr}} = \frac{819410}{500.I_{vf}} < \theta_{p(allowable)} = 0.02 \Leftrightarrow I_{vf} > 81941\,\text{cm}^4$$

$$I_{vfe} = 1298 + 9.84d^2 > 81941\,\text{cm}^4 \rightarrow d > 90.5\,\text{cm}$$

With regard to the hardening behavior of steel material, the height of vierendeel frame can be considered smaller

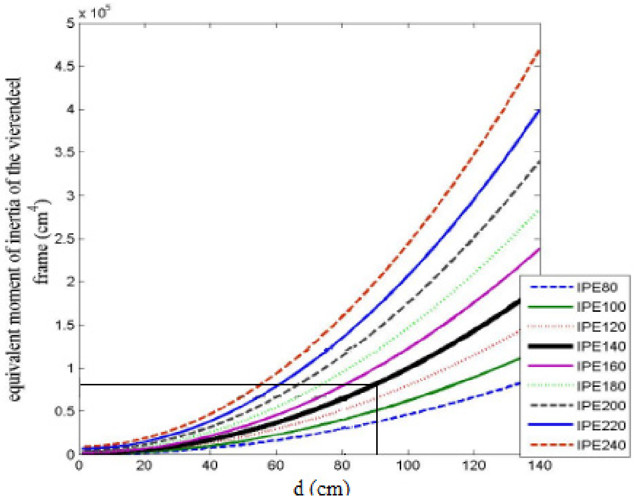

Fig. 14 Relationship between variable d and required equivalent moment of inertia of the vierendeel peripheral frame (I_{vfe}), for 9 types of rolled sections

than the calculated value. Nevertheless, it requires conducting laboratory tests or very advanced nonlinear finite element analysis for further verification. If the structure designed according to reputable regulations with appropriate safety factor (where uncertainty factor of the structure, ρ, is equal to 1), other structural columns can endure the removed column's load, which according to the stress contour of the structural frame shown in Fig. 9 is justifiable.

To avoid a trial and error calculations, the relationship between variable d and required equivalent moment of inertia of the vierendeel peripheral frame (I_{vfe}), is presented graphically in Fig. 14 for 9 types of rolled I sections. For example, if the IPE140 section is used, the required center to center distance of horizontal vierendeel I sections, *d*, according to the above calculations, is 90 cm which is shown in the figure by two horizontal and vertical lines.

In this study, after removing corner column in the first story, maximum plastic hinge rotation in the corresponding beams is 101/5000 = 0.020 radian and for the case of middle column removal, the rotation is 77/5000 = 0.0154. In this case although the structure is designed according to recent valid regulations (considering the ideal connections), the safety factor for stability of the structure is close to 1, that is not appropriate but after strengthening the structure by vierendeel frame, these values reduce by over so percent to 47/5000 = 0.0094 and 33/5000 = 0.0066 radian, respectively, which are much lower than the permissible limit.

For structures with a greater number of floors, engineer can use a vierendeel peripheral frame for a certain number of floors to reduce the height of the vierendeel frame from the occupancy Kingston benchmark (okb) on the corresponding floor.

6. The relative cost of the proposed method

For the five story (symmetric in plan) structure investigated in this study, the relative costs of steel material needed to construct the Vierendeel frame in relation to the costs of structural frames (beams and columns), is calculated and the result can be extended to similar structures. The total weight of the beams and columns for this structure are 57.43 and 25.52 tons, respectively and the weight of the Vierendeel frame designed for this structure is 3.1 tons. Therefore, in terms of the amount of steel material used, the relative costs of the Vierendeel frame to structural frames is (3.1/(57.43+25.52)) ×100 = 3.7% which is insignificant. Also, if the roof level Peripheral beams be used as the lower edges of the Vierendeel frame, the relative cost will be reduced to two thirds of the calculated value.

7. Conclusions

In this study, a method is proposed to prevent progressive collapse phenomena in structures subjected to removal of the single column. The results of numerical study can be summarized as follow:

- A two-dimensional frame is suffering more damage by removing corner column than middle column. Also in the case of three-dimensional structures, it is shown that this claim is true which was confirmed by hand calculations.
- Because of high stiffness of vierendeel frame, the column removal at a higher level will not induce larger vertical displacement than a column removal at first story as is expected in ordinary structures.
- Maximum vertical displacement in column removal point decrease up to 50 percent either for corner column or middle column removal. By selecting the appropriate sections and the height of the vierendeel peripheral frame according to the formulas 1 to 4 and Figs. 13 and 14, the amount of this displacement can be controlled.
- Severe vertical vibration of the check point of the study (that in real condition, due to fatigue phenomenon, has an adverse effect on the structural connections like what happened in the Northridge earthquake (1995)), after retrofitting the frame, have diminished.
- By spending a little cost towards the total cost of construction, structural behavior can be improved and catastrophic collapse can be avoided.
- In case of existing structures, without damage to architectural requirements, this strengthening method can be easily implemented. This is an important parameter in the rehabilitation of existing structures that are in operation.
- Performing practical experiments (though with smaller scale) in accordance with the method presented in this study is recommended to confirm the modeling results.
- The relative costs of steel material needed to construct the Vierendeel frame in relation to the costs of structural frames (beams and columns), is insignificant.

Acknowledgments

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