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Mitigation of progressive collapse in steel structures using a new passive connection

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Abstract. If an alternative path would not be considered for redistribution of loads, local failure in structures will be followed by a progressive collapse. When a vertical load-bearing element of a steel structure fails, the beams connected to it will lose their support. Accordingly, an increase in span's length adds to the internal forces in beams. The mentioned increasing load in beams leads to amplifying the moments there, and likewise in their corresponding connections. Since it is not possible to reinforce all the elements of the structure against this phenomenon, it seems rational to use other technics like specified strengthened connections. In this study, a novel connection is suggested to handle the stated phenomenon which is introduced as a passive connection. This connection enables the structure to tolerate the added loads after failing of the vertical element. To that end, two experimental models were constructed and thereafter tested in half-scale, one-story, double-bay, and bolted connections in three-dimensional spaces. This experimental study has been conducted to compare the ductility and strength of a frame that has ordinary rigid connections with a frame containing a novel passive connection. At last, parametric studies have been implemented to optimize the dimensions of the passive connection. Results show that the load-bearing capacity of the frame increased up to 75 percent. Also, a significant decrease in the displacement of the node wherein the column is removed was observed compared to the ordinary moment resisting frame with the same loads.

Keywords: progressive collapse; alternative path; passive connection; rigid connection; ductility; strength

1. Introduction

Partial collapse of one of the Ronan Point Towers in London in 1968 attracted the attention of structural engineers to progressive collapse phenomena for the first time. A relatively small gas explosion in a corner of a building led to a series of failures in structural members that developed above and below of the explosion point almost as high as the building height (Griffiths 1968). Other successive events like demolition of the Alfred P. Murrah Federal building in Oklahoma in 1995, World Trade Center in New York in 2001, attracted more attention of researchers to this phenomenon (Fu 2016).

One of the main reasons of progressive collapse occurrence is lack of alternate path to redistribute and bear overloads. Several explicit design methods have been suggested in the General Services Administration (GSA 2013) and the Unified Facilities Criteria (UFC 20013) like the alternate path method (APM). In this approach, the structure shall remain stable after column(s) removal scenarios. At the failed column location, effective continuous facilities are required in order to form the plastic hinges and transfer axial load developed by the catenary action.

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As the structure meets the elimination of a vertical member, probability of progressive collapse will increase. In this situation, overloads will impose to the connections of the structure and ordinary connections cannot resist against these new loads. Researchers have been proposed several solutions to resolve this problem like retrofit the floor of the structure using cables (Tan *et al*. 2003).

The result of the column failing in a structure is a sagging beam that is spanned over two bays which must be capable to sustain large deflections and generated overloads by the catenary action. Large ductility of moment-resisting frames can help the structure to redistribute overloads after the local failure as a new load-bearing path. Though, for this purpose connections have to be strengthened (Kim and Park 2008). In order to resist against progressive collapse of structures, Crawford (2002) proposed some connections to resist structures against seismic loads such as Side-Plate. Some design methods are described and compared by Mirtaheri and Zoghi (2016) to resist structures against progressive collapse. Astaneh-Asl *et al.* (2001) investigated the strength of a one-story steel structure with steel deck, concrete slab floor, and wide flange beams and columns to resist progressive collapse. It was observed that after removing the middle perimeter column, the catenary action of the steel deck and girders was able to redistribute loads of the removed column to other columns.

Tan and Astaneh-Asl (2003) conducted an experimental investigation on the viability of steel cable-based systems to

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prevent progressive collapse of buildings. The test results showed that the system could prevent progressive collapse of the floor economically and efficiently. Khandelwal and El-Tawil (2007) carried out a finite element analysis to investigate the catenary action of beam–column sub assemblages in a steel special moment-resisting frame. Byfield and Paramasivam (2007) showed that the industry standard of beam-column connections possesses insufficient ductility to accommodate the large floor displacements that occurred during the catenary action. Kim and Kim (2009) investigated the reinforcing effect of the panel zone on the progressive collapse of the steel moment resisting frames. They implemented several nonlinear analysis to conduct their method. Tsitos and Mosqueda (2012) tested two frames including a special moment resisting frame and a Post-Tensioned Energy-Dissipating Frame (PTED). These models were subjected to quasi-static "push-down" tests to determine their resistance against progressive collapse. In the case of the PTED frame, the ultimate tensile capacity and failure mode of the tendons were proven to be important for the global performance during the push-down tests. Two-story full-scale steel frame with metal deck was tested by Chen *et al.* (2011). They emphasized that the catenary action of the slabs after the column removal redistributed the overloads and decreased the deformations due to the use of girder-metal deck composite beams. JalaliLarijani *et al.* (2013) investigated the vulnurability of two existing buildings against progressive collapse. They implemented an alternative path and linear elastic method to assess the progressive collapse in two different systems. Yi *et al.* (2008) investigated the effect of column removal from a lower story on the behavior of a four-bay and three-story, one-third scale experimental model. Rezvani and Asgarian (2014) investigated the effect of seismic design level for progressive collapse mitigation and reaching it to desired structural safety. Their study was on the concentric braced frame buildings. Zahrai and Ezoddin (2018) showed that using cap truss in the reinforced concrete structures, the axial force in the removed column transfers through an alternative path to adjacent columns to prevent local or general failure or to delay the progressive collapse occurrence. Adom-Asamoah and Ankamah (2016) investigated the effect of design ductility on the progressive collapse potential of RC frames that were designed based on Eurocode 8. Almusallam *et al.* (2010) investigated the effect of blast loads on the progressive collapse vulnerability of a 3D reinforced concrete building. Mashhadi and Saffari (2017) proposed a method to obtain Dynamic Increase Factor (DIF) in nonlinear static analysis of structures against sudden removal of a column. Khaloo and Omidi (2018) proposed a method in which a Vierendeel peripheral frame at roof level is used to redistribute the removed column's load on other columns of the structure. Zhuet *et al.* (2018) proposed an efficient method for calculating the dynamic increase factor to amplify the applied loads on the affected bays of a steel frame structure with semi-rigid connections. Shan *et al.* (2016) examined the interaction between the infill walls and the reinforced concrete frame members in the progressive collapse process. They were tested two 1/3 scaled, four-bay, two-story RC frame specimens to conduct their research. Ren *et al.* (2016) investigated the effect of column-removal on the progressive collapse-resisting mechanism of reinforced concrete floor systems. They were tested seven 1/3-scaled one-way substructure specimens under a middle-column-removal scenario. Gerasimidis *et al.* (2015) investigated the effect of column-removal from a corner section on a tall steel frame according to the alternative load path approach. Faridmehr *et al.* (2015) proposed a new steel connection to improve seismic behavior and performance of structures under progressive collapse loads. Numerical and experimental results showed that this connection is capable to achieve adequate rotational capacity and develops the full inelastic capacity of the connecting beam. Zoghi and Mirtaheri (2016) analyzed an existing seismically code design steel building with the alternative path method to assess its resistance against progressive collapse. They used an analytical macro-model based on the equivalent strut approach to simulate the effect of infill panels. Yang *et al.* (2016) conducted a series of experimental tests to estimate the structural behavior of composite frames under a middle column loss. Dinu *et al.* (2017) investigated the performances of four types of beam to column connection, namely, the welded cover plate flange connection, the haunch end plate bolted connection, the reduced beam section welded connection and the un stiffened extended end plate bolted connection, against progressive collapse. Kim *et al.* (2014) evaluated the effect of MR damper on the behavior of the structure against progressive collapse. They also suggested a preliminary design procedure for this type of dampers to prevent progressive collapse. Chen *et al.* (2016) proposed a method based on the energy principle to analyze the progressive collapse of steel moment framed structures. Lu *et al.* (2017) investigated the performance of RC beam-slab substructures against progressive collapse when an edge column has been removed. They were tested five 1/3-scaled edge span substructure specimens and reported the collapse mechanism of their models. Liu and Fung (2015) proposed a component-based model to predict the dynamic behavior of bolted-connections under progressive collapse scenarios. The capability of this model in predicting the connections' performance is investigated by using a comparative study. Application of the component-based model is investigated for cleat connection by developing a 4-story frame.

Suwondo *et al.* (2019) investigated the three-dimensional progressive collapse behavior of composite steel frames exposed to fire following earthquake.

Han *et al.* (2019) mitigated the behavior of steel frames against progressive collapse using cast steel stiffeners.

In this study, a novel connection is introduced which can bear additional loads in safety area named Passive Connection. It should be noted that the passive connection is assigned only to the critical connections of the structure at the lower stories. Furthermore, this connection takes part in load-bearing after the elimination of the column, and in normal loading conditions, the passive connection stays inactive. Two experimental studies were carried out on the half-scale, one-story, double-bay and prefabricated bolted steel frames in 3D space. The samples include four edge columns that have been connected to a central column by four beams. These experimental studies were conducted to compare the ductility, strength, and durability of the ordinary rigid connection and the novel passive connection. The middle column in both frames has been eliminated, and subsequently, a monotonic quasi-static vertical load is applied above the point in which the column has been removed. Accordingly, the results depict that the strength and durability of the structure with the passive connection were more than the frame with an ordinary rigid connection. Besides, the vertical displacement of the column-removal point in the frame with the passive connection was observed to be less than that of the same frame with ordinary rigid connection. Finally, parametric studies carried out to optimize the dimensions of the passive connection and measure the sensitivity of the passive connection's response to the variation of its characteristics.

2. Introduction to the passive connection

Due to the fact that in progressive collapse procedure the column would be eliminated, obviously, the lengths of the spans increase, so that this change gives rise to the appearance of plastic hinges. Fig. 1 shows the moment distribution alongside with the deformed shape of the beam, before and after removing the column. Furthermore, the overloading caused by the removal of the column exerts additional stress to the connections. These two scenarios contribute to intensifying the potential of progressive collapse in structure. Taking the strong column-weak beam concept into account, it is neither economical nor logical to reinforce all the connections. Hence, within this investigation, we come up with the passive connection method to manage this phenomenon. In this method, while designing the structural elements of the structure, except for the critical zone of the collapse growth, standard design steps have been established. In such conditions, the design of the connections is based on the moments which have been created after the column removal. These moments were not active in normal conditions. After column destruction, these moments will become active by intensifying the deformations of the beams. Designing the connections based on these moments increases the moment of inertia of the beams in the region that the column has been detached. Accordingly, while the moment of inertia of these beams increases, the formation of the plastic hinges, and consequently the large deformation would be restrained. Subsequently, it leads to a decrease in the degree of indeterminacy of the structure, and the progressive collapse possibility as well.

The especial interest here, to increase the moment of inertia of the beams that are directly connected to the column which is considered to be detached, is to use the tapered section idea. In this method, considering the new span lengths and also redistribution of moments, we redesign the beginning, middle, and the end of the beam to determine its resistance.

As it is shown in Fig. 2, the upper part of the new element is connected to the beam, while its lower part is free. However, when the deformations exceed the allowed value, the lower parts would also be engaged. Depending on the position of the connection, it acts as tension or as compression member in different stages of the collapse. Designing the length of the lower part is based on the allowed deflection analysis, and the lower part is designed so that it would avoid engaging with other sections before eliminating the column. In what follows, some of the merits of such a connection are listed:

- Ease of implementation
- Meager execution cost due to exploiting waste materials
- Not generating any interfering with installing risers
- Simplicity in design
- Innovative while simple technology
- Not violating strong column- weak beam due to being passive

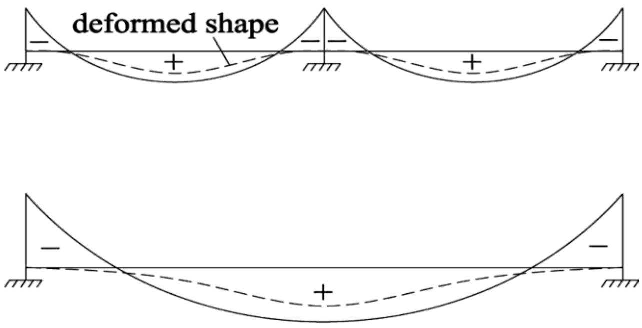


Fig. 1 Moment distribution and deformed shape of a beam before and after column removal

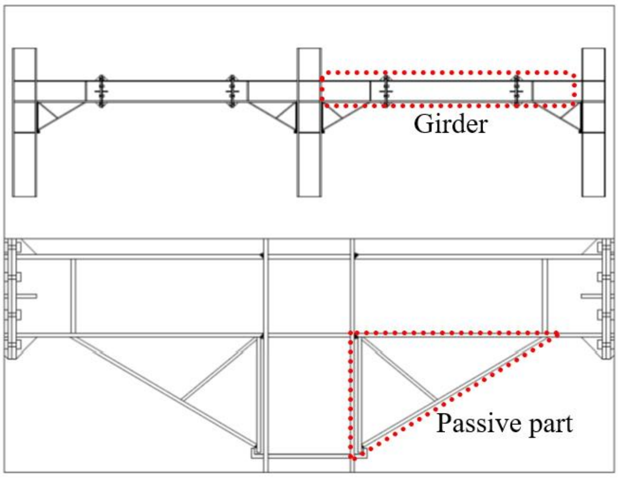


Fig. 2 General view of passive connection

3. Designing the laboratory frames

In this research, two experimental models have been elaborated, each of which is a model of a real eleven-story steel building that is built and located in Tehran, Iran. Seismic resistance design of this building was based on Iranian Code of Practice for Seismic Resistant Design of Buildings (BHRC 2004). The lateral resisting system of this building is column-tree especial moment resisting

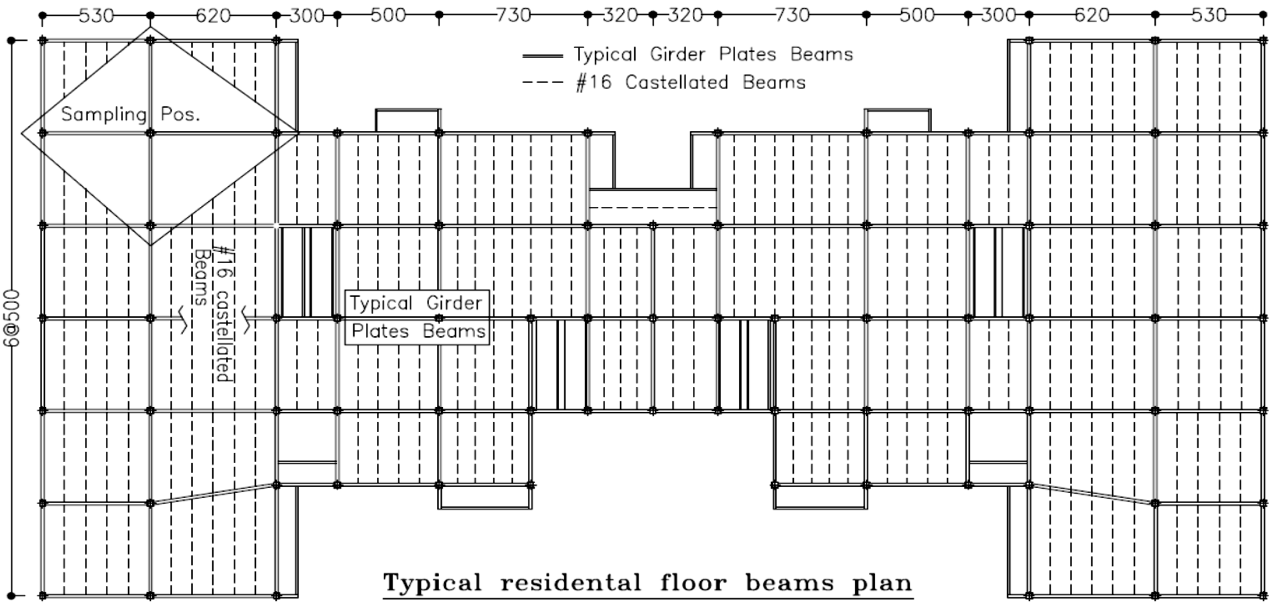


Fig. 3 Sampling position

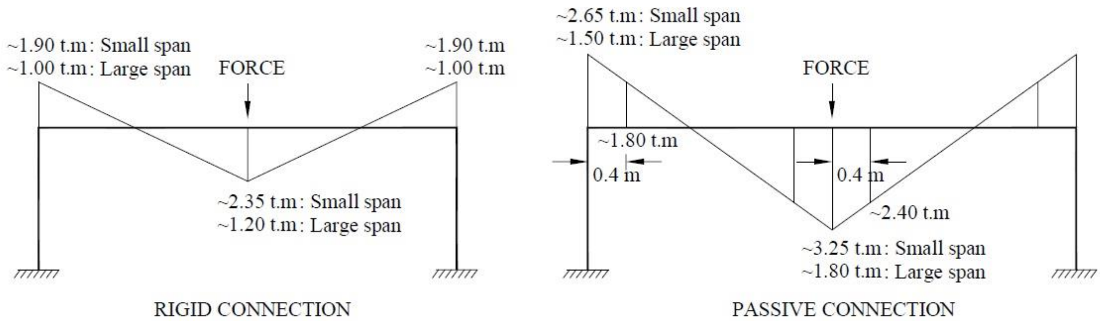


Fig. 4 Moment diagram of the frames in initial analysis.

frames (SMRFs). The columns and girders of this building are box shape and H shape, respectively, and its connections are made up of bolted joints.

To conduct this experimental study, the critical elements that have many criteria for comparison are selected from the first floor and are designed in a half scale. Sampling position is shown in Fig. 3. These experimental models are created in a 3D frame system with screw connections including one story and two spans. Here, our experimental models are designed by using equalization of modal periods and mode shapes of the whole structure. Another noteworthy point about this study is that the critical spans with two different lengths are selected to evaluate span length's effect on the beams and the corresponding connections.

4. Numerical modeling of the structure

Utilizing the ABAQUS software, numerical simulations of the experimental models are presented in this section. Numerical models have been manipulated under a 10tons in magnitude quasi-static monotonic load, the same as the experimental conditions. The flange's location in the rigid and passive connection is determined according to the moment diagram and designing codes. Due to the limitations regarding the scaling of the model, a 40 centimeters distance from the column is selected to place the flange. In Fig. 4, one can find the moment diagram of the frames.

All of the members are made up of St37 steel type. The modules of elasticity, the mass density, and the Poisson's ratio of the used steel is equal to 2.1e6 $\mathrm{Kg/Cm^2}$, 7.85e-3 $\mathrm{Kg/Cm^3}$, and 0.3 respectively. Furthermore, the nonlinear behavior of the steel is modeled based on the stress-strain diagram that is shown in Fig. 5. It should be noted that this diagram is obtained from the experiment and is calibrated for parametric study. The panel zone has also been reinforced. To manage this phase, the beams' flanges are welded to the column. Subsequently, the columns are reinforced by using steel backing plates where the tension or the compression forces transferring by the passive connection would be applied to the column. Fig. 6 shows details of the screw flange used in both frames.

Shell elements with 6 degree of freedoms have been used to model the entire structures. It should be noted that the contact surface between the screws and the connection plate are tied together. Also, the main element of the passive

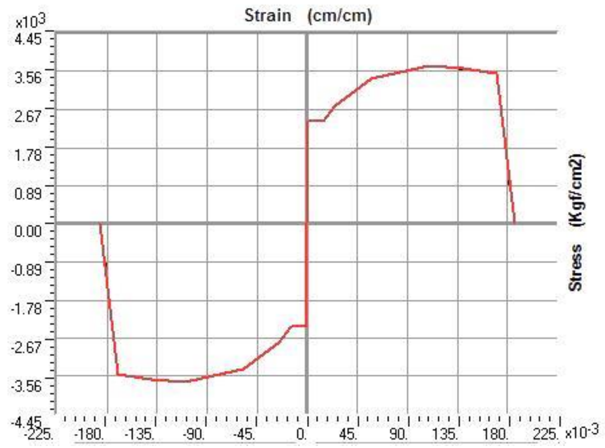


Fig. 5 Stress-strain diagram of the steel which is used in this research

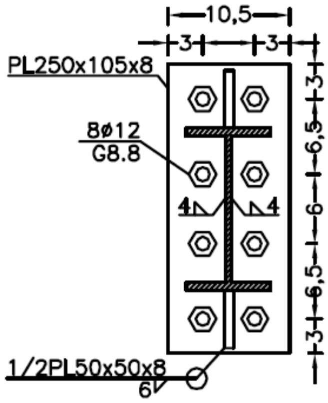
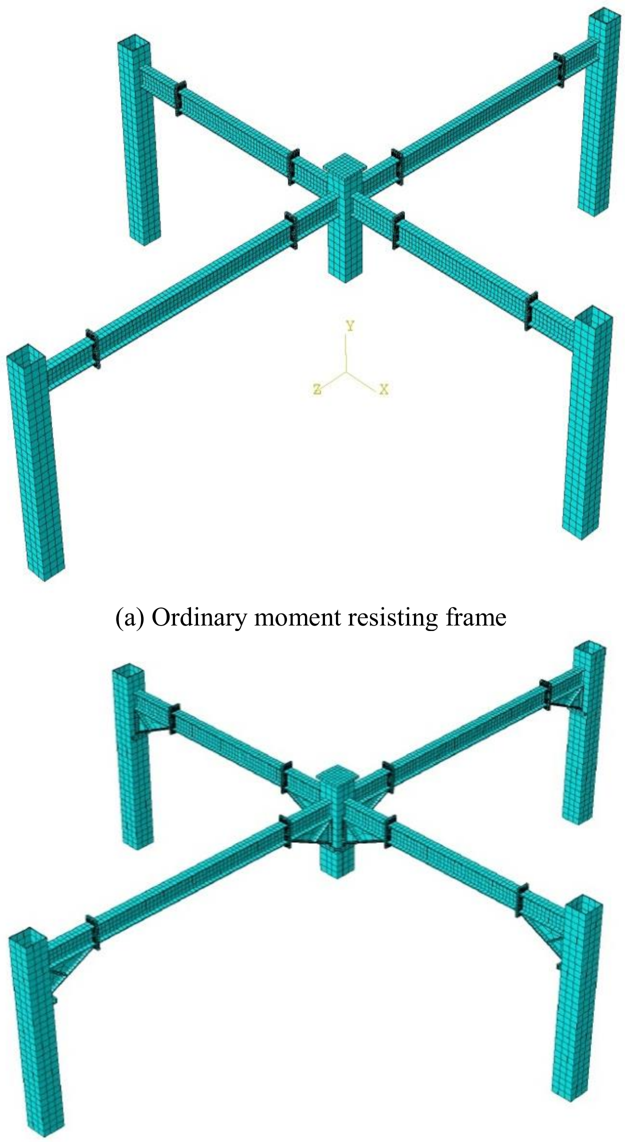


Fig. 6 Details of the screw flange used in both frames



(a) Ordinary moment resisting frame

(b) Frame with passive connections

Fig. 7 General view of modeled frames in ABAQUS

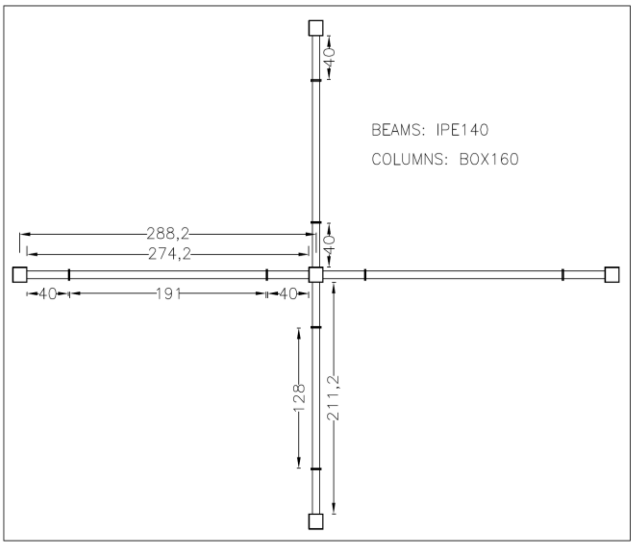


Fig. 8 Workshop plan of the frame with rigid connections

connection (the vertical member) has not any contact with other elements at the initial condition. Thus, this vertical member must be modeled so that the contact of the vertical member with other elements due to the movement of the passive connection would be defined, and the force that is resulted by this contact will be considered. Therefore, here the surface to surface contact pairing technique is used in the present modeling. As the columns are vertical in this study, it is essential to consider a state in- which the mandatory boundary conditions have not significant effect on the results. Due to several analyses, it was found that in a 20 centimeters distance from the upper flange of the beam, the column does not endure significant tension. Particularly, the column does not have any significant effect on the frame stiffness and accordingly on the results. Hence, the columns continued up to 20 centimeters above the upper flange of the beams. Fig. 7 shows the modeled frames in ABAQUS software.

5. Setup procedure

5.1 Frame with rigid connections

All of the welds were implemented in corner intrusive weld shape in the factory. The connections of the first frame are constructed rigidly with Welded Unreinforced Flanges (WUF). Figs. 8 to 10 show the geometry of the first frame. After constructing the model and moving it to laboratory,

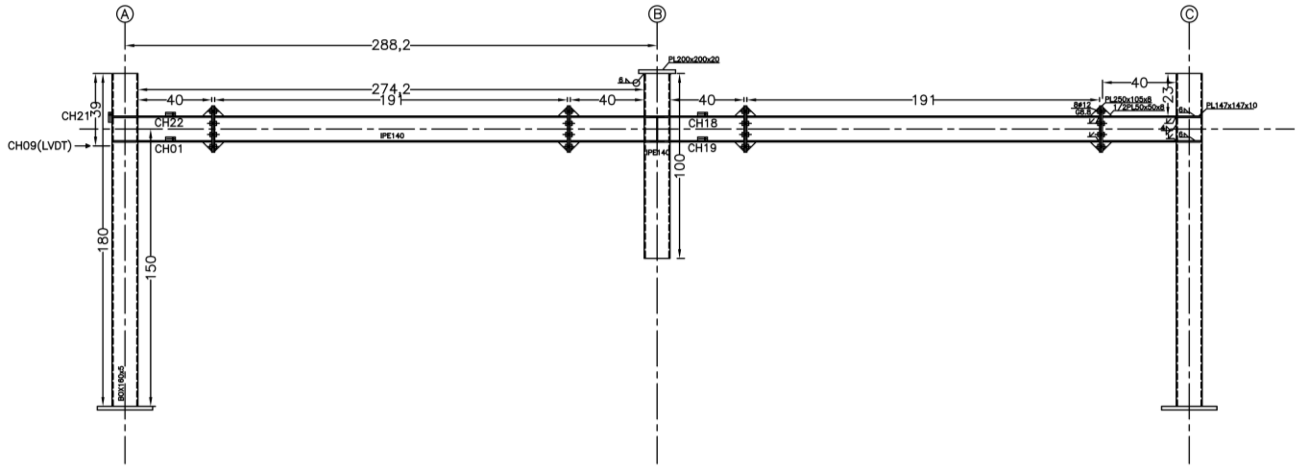


Fig. 9 Workshop map of longer span for the frame with rigid connections

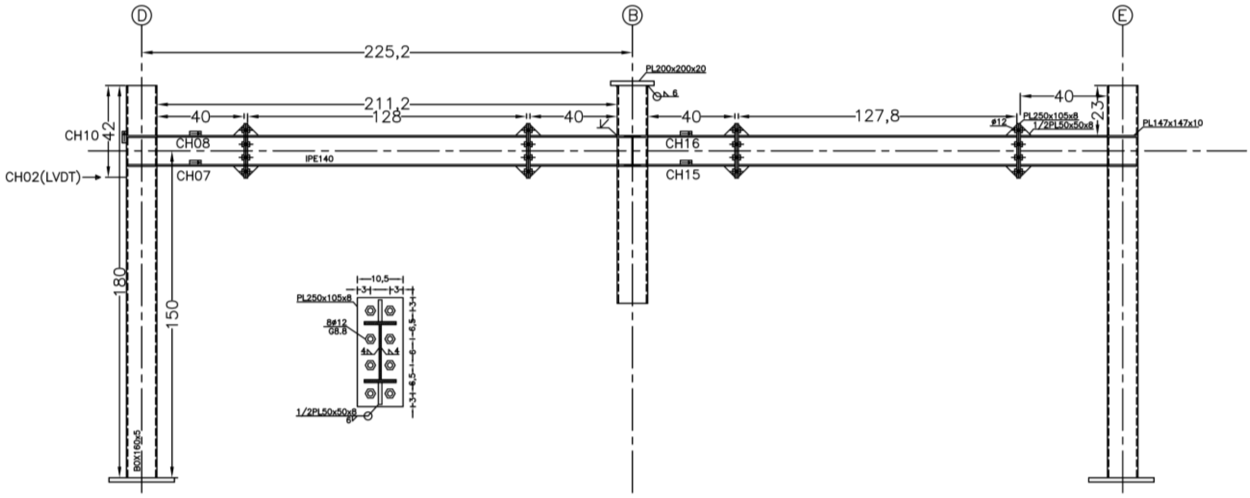


Fig. 10 Workshop map of longer span for the frame with rigid connections

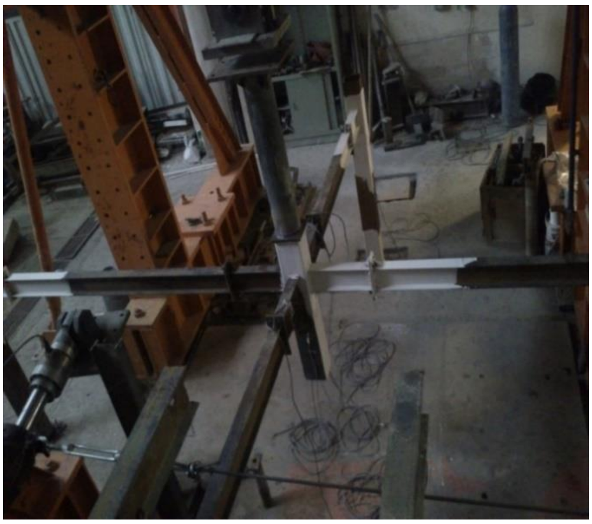


Fig. 11 Setup details of the frame with rigid connections

Fig. 13 Exploitation of the passive connection in the laboratory

the bases of the column were welded to the rigid floor of the laboratory. Subsequently, two adjacent columns were restrained with barriers to prevent the model from twisting. Then, the actuator was placed on top of the middle column (Fig. 11). Considering the numerical results, ten uniaxial strain gauges with 5 millimeters lengths and 120-ohms resistant were used to record the strains. Furthermore, two LVDTs were used to record the displacements. These strain gauges and LVDTs were placed on the structure as shown in

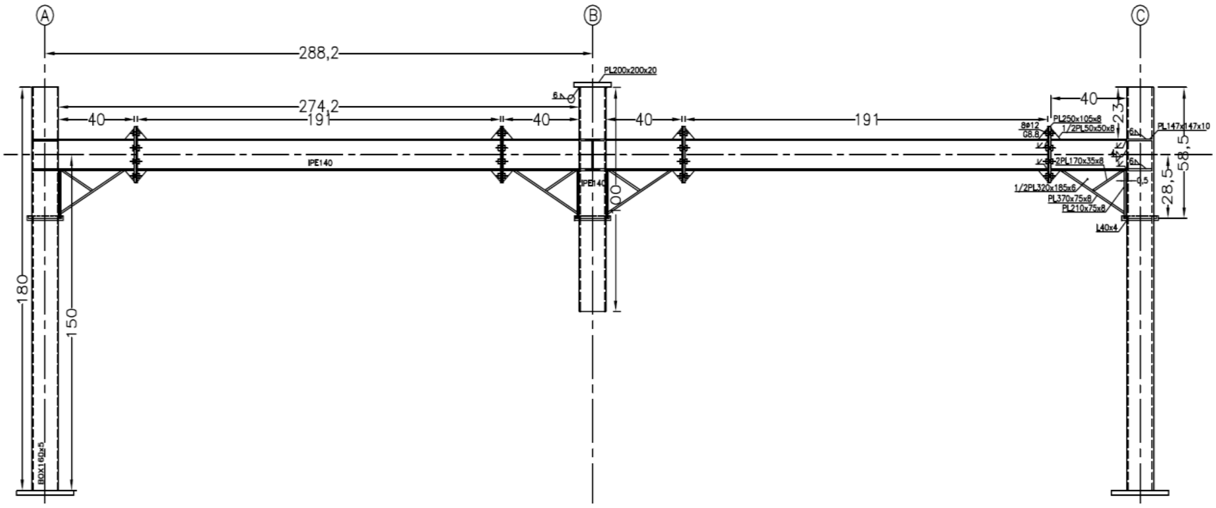


Fig. 12 Workshop map of longer span for the frame with passive connections

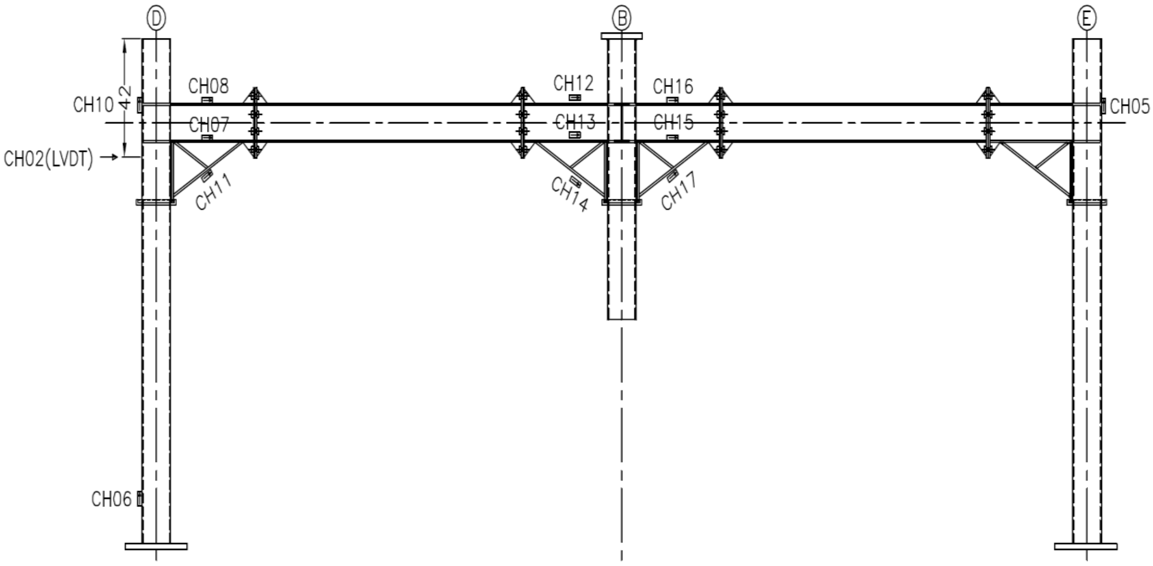


Fig. 14 Location of instruments for the smaller span of the frame with passive connections

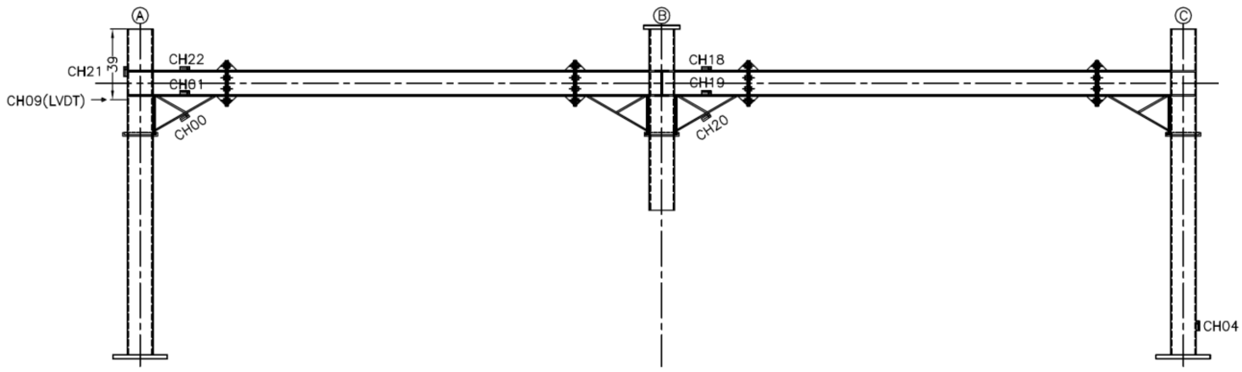


Fig. 15 Location of instruments for the longer span of the frame with passive connections

Figs. 9 and 10. Note that the actuator is also equipped with an LVDT to record its displacement during the loading process. Finally, a datalogger is used to record the outputs. It should be noted that all the tests are implemented at structure laboratory of civil engineering faculty of Amir-Kabir University of technology, Tehran, Iran.

5.2 Frame with passive connections

All construction procedure stages are similar to those of the frame with rigid connections, except at building and installing passive connection stages. Since the passive connection will become activated only during large

Fig. 16 Setup procedure of the frame with passive connections

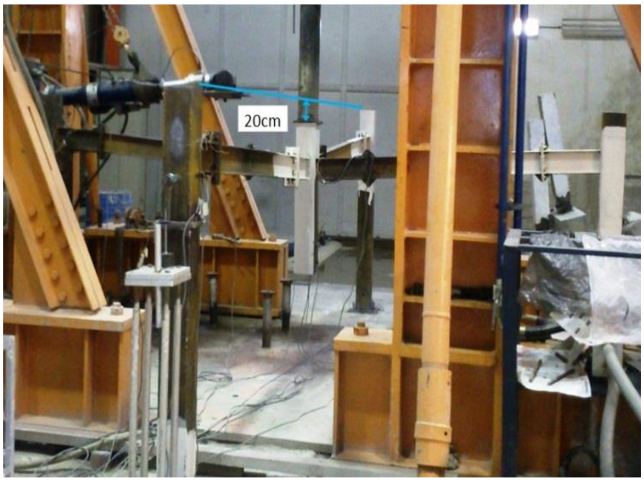


Fig. 17 Deformed shape of the frame with rigid connections

deformation which occur in the event of possible potential progressive collapse, therefore, it will act like a WUF connection in operational state and seismic loads. It should be noted that the bulge part of the passive connection should be placed inside the roller part. Moreover, it must have a 5millimeters distance from the column. In this regard, a No. 4 angle is welded to the column. To make sure of the correct performance of the roller and the passive connection, the aforementioned angle is placed and welded all the around the column. Fig. 12 shows a schematic representation of the second frame for the large span. Also, Fig. 13 illustrates the exploitation of the passive connection in the laboratory.

As shown in Figs. 14 and 15, twenty uniaxial strain gauges with 5 millimeters lengths and 120-ohms resistant were utilized to record the strains. In addition, two LVDTs were used to keep track of displacements. Fig. 16 demonstrates the setup procedure of the second frame in the laboratory.

5.3 Loading

As stated in previous sections, loading is exerted by a dynamic actuator. The rate of the quasi-static load is considered to be equal to 0.15 mm/s. The first frame's experiment took about 15 minutes, and the second frame's experiment lasted roughly 21 minutes.

Fig. 18 Deformed shape of the frame with passive connections

6. Experimental results

Regarding the sensitivity of the analysis, and to afford accommodation for an appropriate engineering judgment to determine the frame's behavior under the progressive collapse event, several different parameters have been sensibly documented. These parameters include the vertical displacement of the point wherein the column is removed, the lateral displacement of the columns, the ultimate strength and ductility of the columns, the process of load distribution in different parts of the passive connection, and finally the frame's failure procedure.

6.1 The effect of passive connection on vertical displacement of the removed column

Figs. 17 and 18 show the deformed shape of the frame, and the vertical displacement of the loading point, at the end of the experiment. The values of the displacement that are shown in Figs 17, 18 are obtained as compared to undeformed shape of the structure which is shown in Fig 16. Subsequently, Fig. 19 represents the force-displacement-diagrams made from noticed data during the test.

It should be remarked that the following conclusions can be drawn from Figs. 17, 18, and 19:

- The ultimate strength for the frame with passive connections is about 7.3 tons, and for the one with rigid connections, it is about 4.2 tons which manifests an increase up to 74 percent to the ultimate strength of the frame.

- As the stiffness of the frame with passive connections is more than the one with rigid connections, the ductility of the former is less than that of the latter. In regard to the dissipation energy, the results are reversed. Considering the relation of ductility as $\mu = \frac{\text{Ultimate displacement}}{\text{Elastic displacement}}$, the values for the ductility of the frames are obtained in what follows: $\mu_{Passive} = \frac{225}{65} = 3.92$ and $\mu_{Rigid} = \frac{170}{40} = 4.25$.

- The ratio between the ultimate displacement of the frame with passive connection and that of the frame with rigid connection at loading point is equal to $\frac{225}{170} = 1.5$.

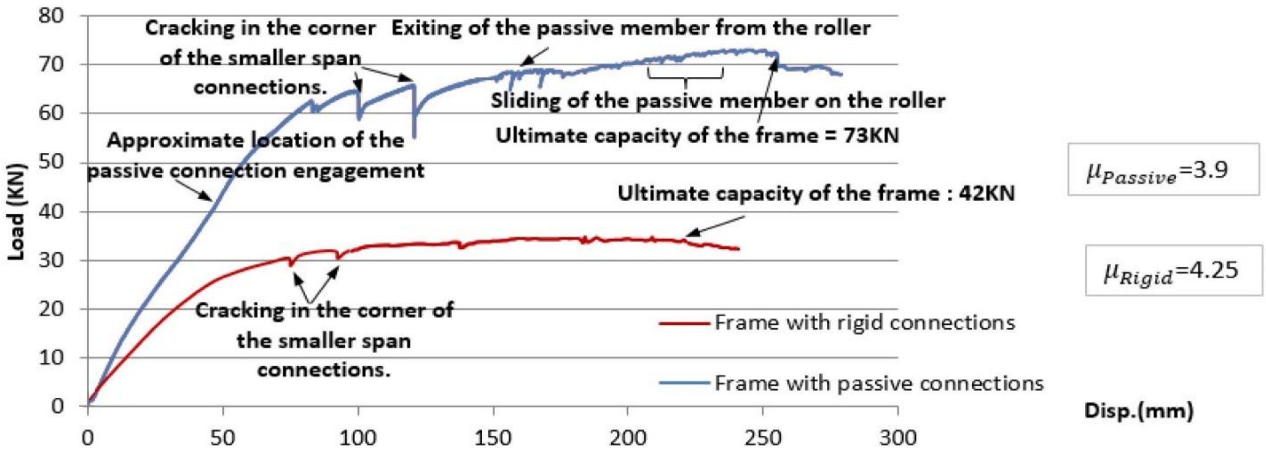


Fig. 19 Force-Displacement diagram of the frames at loading point

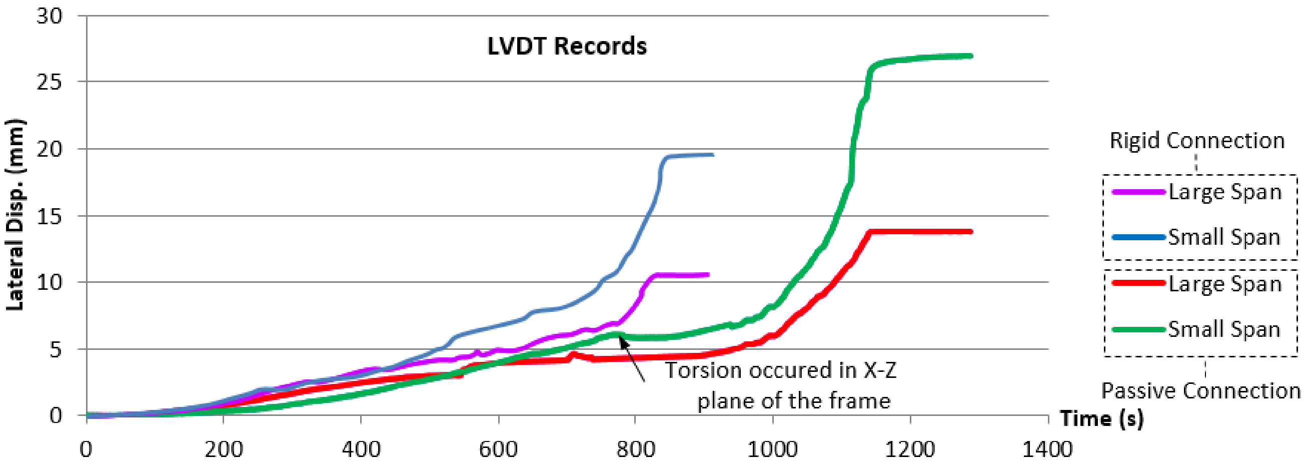


Fig. 20 Lateral displacement-Time diagrams of specimen's columns

Furthermore, this value is equal to $\frac{80}{40} = 2$ while the experiments reach the yield point.

- The amount of plastic rotation angle in frame with passive connection for the small and large span is equal to $\frac{5}{210} = 0.0238$ *and* $\frac{7}{270} = 0.0259$, respectively. These values are smaller than the acceptance criteria which is introduced in Table 5-2 of UFC 4-023-03. The amount of rotational angle of the beam within the frame is computed as follow:

0.0284-0.0004d → 0.0284 - 0.0004 × 5.5=0.0262

Where d is depth of the beam in inches.

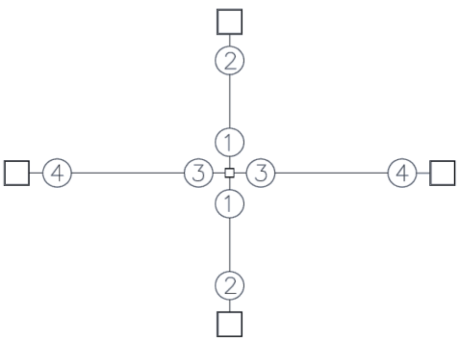


Fig. 21 Sequence of the plastic hinge formation

6.2 The effect of the passive connection on the lateral displacement of the frame's columns

Another behavior that was expected from the frame with the passive connection in comparison with the frame with the rigid connection is a lessening in the lateral displacement of the column adjacent to the collapse location at the yield point. Therefore, the lateral displacement of a column in the large span, and that of a column in the small span are recorded by using two LVDTs which were located above the mentioned columns. As shown in Fig. 20, generally, the lateral displacement of the small span columns was more significant than other columns. As a result, while the frame with rigid connections was in its ultimate loading (4.2 tons), the one with passive connections remained in the elastic range. At this point, for the frame with passive connections, a 68 percent reduction in columns' lateral displacement was observed in the small span (from 19mm to 6mm); For the large span, this value was equal to 71 percent (from 11mm to 3.2mm).

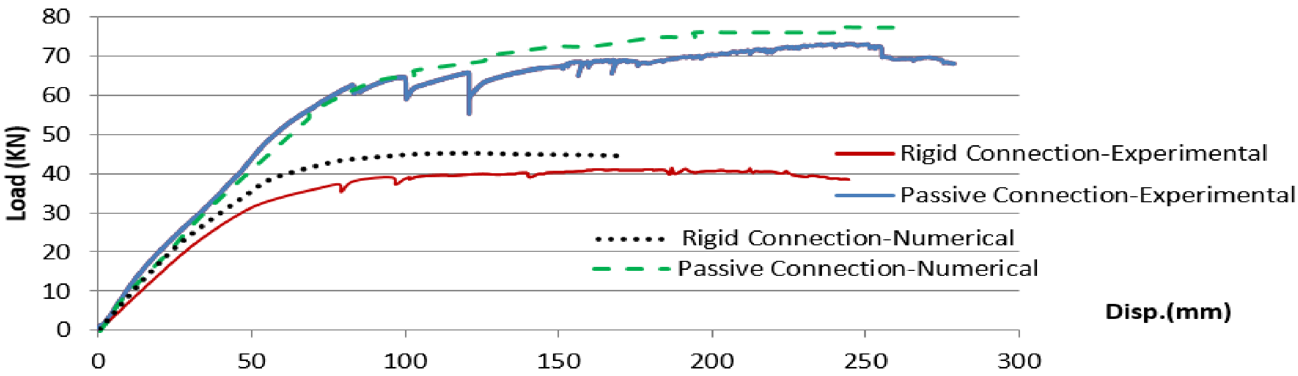


Fig. 22 Numerical and experimental force-displacement diagrams

It should be noted that the steady slope of these curves is originated from performing the failure mechanism to the frames. In such conditions, the plastic hinges are made in beams. Then beams are deformed without any contribution from the column's resistance. In this study, the welding is defined to be of full penetration groove type. Therefore, the capacity of the rigid connection's welding is considered to be equal to the plastic capacity of the beam section which is not allowed to be formed in the design procedure of the beams. Thus, plastic hinges are not made at the upper, and the lower flanges of the beams. On the other side, due to the column exclusion in passive connection, it is assumed that the elements undergo large deformations. Accordingly, the applied moments increase until the plastic hinges are formed. Another point is that recalling weak beam-strong column concept, the moment of inertia of the beams must not exceed that of the columns in rigid connections, while this concept is not valid for passive connections. In passive connections, the moment of inertia of the beams must be greater than that of the columns. Considering the two-sided operation of the passive connections and the belt which is welded all around the column, the tension or compression load of the diagonal member is transferred to the next connection by the column. Therefore, this force must be considered in design procedure.

6.3 Comparison between numerical and experimental results

As it is shown in Fig. 21, the plastic hinges first form in connections of the small span, and then in the large one. In each span, the first plastic hinges form in connections which are at the vicinity of the removed column, and subsequently in connections of the other side of the beams. For the comparison purpose, Numerical and experimental force-displacement diagrams of both frames are presented in Fig. 22. In this regard, the following results could be extracted from these two diagrams.

- Force design of the frame with rigid connections is equal to 40 KN. After a 5 centimeters displacement of the node in which the column is removed, the frame will yield. For the frame with passive connections these values are equal to 60 KN, and 6.5 centimeters. Favorably, the numerical models have given acceptable approximations of these values.

- When the plastic hinges begin to form, the applied loadings to the frame with passive connections and rigid connections were reported to be equal to 40 KN and 70 KN, respectively.

- The ultimate loading of the frame with passive connections and the frame with rigid connections was reported to be equal to 46 KN and 80 KN, respectively.

- The minor discrepancies between the numerical and the experimental results at the first stage can be attributed to the practical inaccuracies, indeterminacy of the mechanical behavior of the materials, the heterogeneity in the experiment's conditions. On the other hand, it can be originated from the solving method and the parameters of the nonlinear equations in numerical analysis.

7. Parametric studies

After calibrating the numerical model, a parametric investigation has been implemented to optimize the passive connection's geometry. The parameters that are considered in this study include characteristic strength of the connection's elements, the distance between the flange and the column, and at last the length and the height of the cripple.

7.1 Effect of steel's strength

Here, the effect of characteristic strength of the used steel in the passive connection has been studied. In this regard, a comparison between the usage of passive connection made from st37 and st52 steels has been done. Obviously, when the passive connection is made from st52, the strength of the frame increases because of the higher strength of the used steel. Comparing these two especial cases yield the following results:

- Vertical displacement of the node in which the column is removed, is reduced by 3.5 percent that seems not remarkable.

- The stress in the beam of the small span reduces approximately up to 10 percent. Then again, the stress in the beam of the large span decreases up to 20 percent. As the connection would be attached to the beam and increases its stiffness, the stress reduction of the beam would be more than the column.

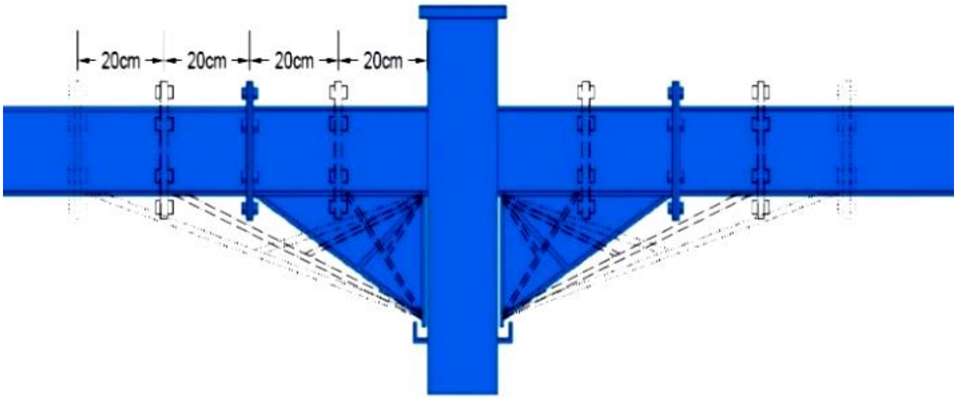


Fig. 23 Change in the length of the passive Connection's cripple

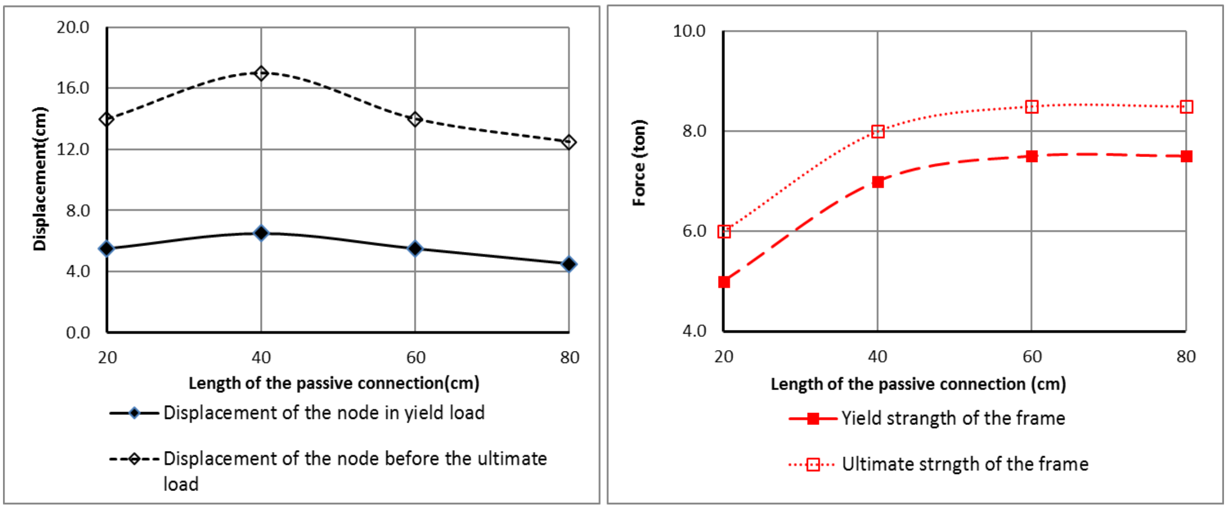


Fig. 24 Effect of the cripple length on different parameters of the frame

Table 1 Effect of the passive connection's length on the frame behavior

Parameter	Rigid connection	Cripple length (cm)			
		20	40	60	80
Yield strength of the frame (ton)	4	5	7	7.5	7.5
Displacement of the column removal node (cm)	5	5.5	6.5	5.5	4.5
Ultimate strength of the frame (ton)	4.6	6	8	8.5	8.5
Displacement of the column removal node before the ultimate load (cm)	9	14	17	14	12.5

- There was no significant change in the yielding strength of the frame. Based on the aforementioned results, as the passive connection holds a small part of the frame's resistant system, an increase in the characteristic strength of the used steel has no significant effect on improving the frame's behavior.

7.2 Effect of cripple's length

As the length of the connection is a function of several parameters like design process, practical problems, installation procedure, and transportation dilemmas, it was not easy to change this length. Nevertheless, by neglecting these restrictions, a parametric study is conducted to investigate the effect of this length on the structural performance of the frame. For this purpose, as shown in Fig. 23, the length of the cripple is changed in the range of 20 to 80 centimeters, and the results have been put under discussion. Accordingly, a decrease or an increase more

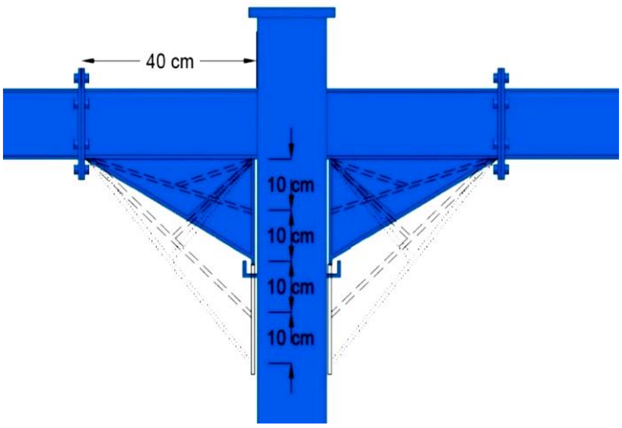


Fig. 25 Change in the height of the passive Connection's cripple

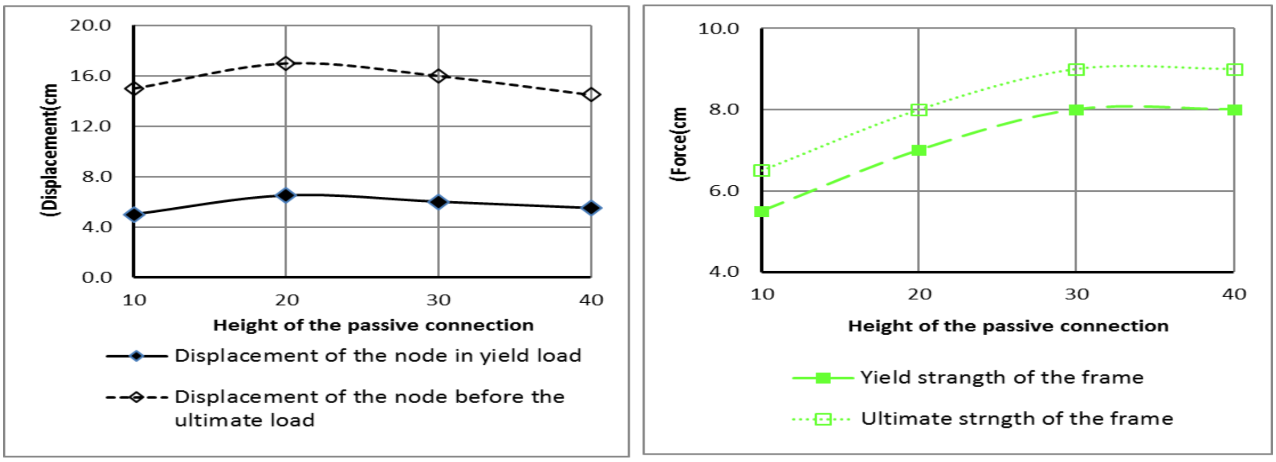


Fig. 26 Effect of the cripple height on different parameters of the frame

Table 2 Effect of the passive connection's length on the frame behavior

Parameter	Rigid connection	Cripple height (cm) 10	20	30	40
Yield strength of the frame (ton)	4	5.5	7	8	8
Displacement of the column removal node (cm)	5	5	6.5	6	5.5
Ultimate strength of the frame (ton)	4.6	6.5	8	9	9
Displacement of the column removal node before the ultimate load (cm)	9	15	17	16	14.5

than the mentioned values in the length of the cripple is not practical to accommodate a solution for the problems that arise at the design procedure of the passive elements. At last, it should be noted that crippling and local buckling of the passive elements is checked in every state.

Table 1 shows the effect of the change in the cripple length on the structure behavior under the column removal condition.

According to Table 1, when the length of the passive connection increases, especially for the larger than 50 centimeters lengths, the beams become stronger and the resistance of the frame will depend mainly on the resistance of the columns. Apparently, the formation of the plastic hinges in the column is not desirable. Moreover, increasing the length of the passive connection more than 50centimeters does not significantly affect the frame's resistance. It just increases the displacement of the node in which the column is removed. Then, the stresses in the beams will decrease and consequently the ductility and the energy absorption of the frame will decrease. These results are summarized in Fig. 24. As a result of the aforementioned discussion, the optimized length of the passive connection is reported to be equal to 50 centimeters.

7.3 Effect of the cripple's height

The stated limitations about the cripple's length are also valid for the cripple's height. In this regard, increasing the cripple's height may not be practical. Along with the practical problems, a parametric study is implemented to evaluate the effect of the cripple's height on the results. For this purpose, Fig. 25 demonstrates different values for the height of the cripple between 10 to 40 centimeters within which the variable parameters have been studied, and the results have been recorded. It should also be noted that if the cripple's height reaches out of this range, no noticeable effect on the performance of the passive connection, and more generally, on the performance of the frame would be observed. In all the aforementioned stages, the crippling and local buckling of the passive connection elements has been checked. At last, the effect of the cripple's height on the performance of the frame is shown in Table 2. In this Table, the value of each parameter is compared to the corresponding parameter of the frame with rigid connection.

Considering the results of the analysis in accordance with Table 2, it was concluded that by increasing the cripple's height, the rigidity of the passive connection increases, and the connection becomes stronger and stiffer than the mere beams and the columns. Furthermore, by this change the stresses decrease in the passive connection and it makes the beams and the columns take control of the frame's strength. When the height of the cripple increases, the diagonal member exerts a concentrated force to the column and it inserts more stress and subsequently more moment to the column. Therefore, increasing the height of the cripple, on the other hand, leads to a reduction in the frame strength. According to Fig. 26, in cripple heights greater than 30 centimeters, the frame's strength does not increase. Moreover, the ductility of the frame and also the displacement of the node in which the column was removed, diminished. To wrap up, it can finally be stated that the optimum height for the cripple in this study is equal to 30 centimeters.

Comparing the results of this section with results of the previous section, the following conclusions can be obtained:

- Increasing the length of the cripple have more effects than rising of the cripple's height on the displacement reduction of the frame.

- The ultimate and yielding strengths of the frame are more significant when the increase is implemented in the cripple's height than the case of cripple length increase.

- Generally, according to the results of the analysis, design, and also practical issues, increasing the cripple's height is recommended rather than its length. However, the optimum values of its length should be considered.

8. Conclusions

Increasing the resistance and the ductility of connections in every structure play an essential role in decreasing the damage stem from the progressive collapse phenomenon. In this study, in order to prevent the occurrence of progressive collapse in steel structures with moment resisting frames, the passive connection was introduced and tested. As this connection is easy to analysis and have low construction cost, therefore, it can be implemented in the structures in large scale. This connection stayed inactive during the normal loading conditions of the structure. After eliminating the column during alternative paths procedures, this connection got engaged with other elements and became active. This connection was designed including the resisting cripple and based on the relations of the beams with variable cross-sections. In this regard, two sample models, the first one a normal frame with moment resisting connections, and the second one a frame with passive connections have been made and subsequently tested.

The results of the tests presented a noticeable increase in both strength and the energy absorption in the frame with passive connection in comparison with the frame with moment resisting connections. Accordingly, load-bearing capacity of the frame increased up to 75 percent. Also, a significant decrease in the displacement of the node wherein the column is removed was observed compared to the ordinary moment resisting frame with the same loads.

Furthermore, the parametric studies on passive connection showed that the increase in the height and the length of the passive cripple is limited and has an optimum value. As the final point, taking note all the design and construction issues in this optimum value, the strength and the ductility of the connection have been improved, and the displacements have been lessened.

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