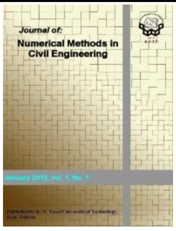

Numerical Methods in Civil Engineering

Investigation of different methods in strengthening braced steel frame under blast load

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Abstract:

Unpredicted loading caused by blast or impact may lead to severe damage in structures. Such damage may affect the whole structural performance. Connections are the key contributor to the integrity and energy dissipation capacity of the structural steel systems and play the most important role in mitigating such consequences. In the view of the importance of connections, finite element analysis is used in this paper to study the behavior of braced steel frame under blast loading using ABAQUS finite element software. This Frame is adopted from a hospital building. Firstly, the behavior of the frame is investigated under near-field blast load to locate the regions that are susceptible to damage. Then according to the damage, five different possible reinforcing scenarios are proposed, and the advantages and drawbacks of each one are investigated. It is observed that asymmetrical deformations and damages are notable due to the front members' severe local damage. The damage in the structure decreases rapidly as the height increases. In addition, the vertical web stiffeners and triangular plates on the top and bottom flanges eliminate the severe plastic strain and stress concentration in the connection zone causing the critical regions to move toward the middle of the beam that is desirable. It is also observed that continuous reinforcement of the beam webs with web sheets on both sides, significantly reduces the stresses on the web. This is a more effective way than the web stiffeners because of fewer stress changes in other members due to minor changes in the stiffness of the connected members.

1. Introduction

In recent decades, local structural damage caused by unusual loading of blast or impact has been of great importance to researchers as it plays a major role in the progressive collapse of framed structures. An explosion is the sudden release of energy, which can be in the form of a gas explosion, a nuclear explosion, or a bomb. In an explosion, the potential energy of the explosives is released in a very short time (within a few microseconds), and this released energy is converted into two separate phenomena, thermal radiation and shock waves (on the ground and in the air).

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Explosive waves in the air are the main cause of damage to buildings. These waves propagate at supersonic speeds, increasing the ambient pressure.One of the major events that may occur in structures after an explosion is the progressive collapse. Progressive collapse begins with the local failure of a primary structural component leading to the collapse of adjoining members. This failure may extend to the entire structure or a large area of it. Progressive collapse has been the subject of many pieces of research in recent decades [1-6].

Explosion loads are applied to the structure in a much shorter period of time than seismic loads. Therefore, the strain rate of materials becomes critical and should be particularly considered in the design process. In addition, compared to other hazards caused by earthquake, wind or flood, an explosive attack has the following distinctive features:

1- The intensity of the pressure on the building can be many times greater than other hazards. For example in an urban environment, the explosion of a car bomb on the sidewalk may put a maximum pressure of about 700 kPa on buildings.
2- The pressures caused by the explosion are greatly reduced by increasing the distance from the source of the explosion. So the damage to one side of the building that was exposed to the explosion may be much more severe than the back side.
3- Explosive loads are usually local loads that lead to local failures. Conversely, seismic loads are caused by ground movements that are applied uniformly throughout the base and foundation of the building, and as a result, all components of the building are involved with this load, when the structure is subjected to seismic load.
4- The time of this event is very short and is usually expressed in milliseconds, which is different from earthquakes and winds, which are expressed in seconds, or continuous floods, which are expressed in hours. For this reason, the mass of the building has a large reduction effect on the response of the structure, because it takes time to stimulate the mass of the building. Thus the mass reduces the response. This is in contrast to an earthquake in which the forces are almost simultaneous with the mass response of the structure and may intensify.

Following an explosion, the waves propagating at supersonic speed, increase the ambient pressure which drops to the initial amount after a short time. Fig. 1 shows the general shape of a time-pressure curve of a shock wave of an air explosion. The shock front reaches the target at time t_A . After t_r seconds the pressure reaches its maximum value P_{so}. Since the time interval between t_A and t_r is very short, it is common to assume that reaching the maximum pressure value occurs immediately after the shock front arrives. The maximum pressure of P_{so} decreases during time t_o to its initial level, defined as the positive phase of the pressure shock. After this step, the negative phase occurs, and will continue for t_o^- during which, the amount of pressure will be less than the initial ambient pressure. In this phase, the wind direction (particle motion) reverses. The negative phase is not very important in the design process and is usually neglected [7]. The positive impulse of the impact is equal to the area below the positive phase curve of the time-pressure curve and is displayed by i_{so}.

Various methods proposed for applying blast load to the structures divide into two general categories: experimental relations and the methods based on fluid dynamics analysis. Despite the accuracy of the methods based on fluid dynamics analysis, the experimental relations are more common in design practices because of their simplicity.

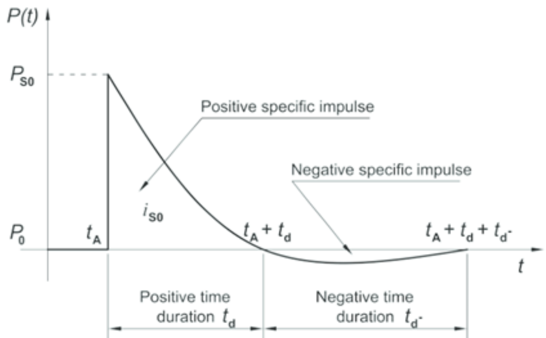


Fig. 1: General shape of a time-pressure curve

In most experimental relations presented for blast loading, it is common to scale the characteristics of an explosion wave where, based on experimental results, the characteristics of the blast wave can be estimated for a desired explosive charge and distance. The most common form of blast scaling is the Hopkinson–Cranz [8, 9], or cube-root scaling. It is customary to characterize the pressure loadings in terms of scaled range Z as:

$$Z = \frac{R}{W^{1/3}} \tag{1}$$

where W is the explosive weight in TNT, and R is the radial distance from the center of the explosion.

A blast at a distance R from the center of an explosive source of characteristic dimension d will produce a blast wave with an amplitude of P, duration t_d, and impulse i. The Hopkinson–Cranz scaling law states that, at a source-to-center distance ZR of a similar explosive source with characteristic dimension Zd, the blast wave will be in similar form with amplitude P, duration Zt_d and impulse Zi as shown in Fig. 2. Numerous experiments conducted over a vast range of explosive charges support the Hopkinson–Cranz scaling law.

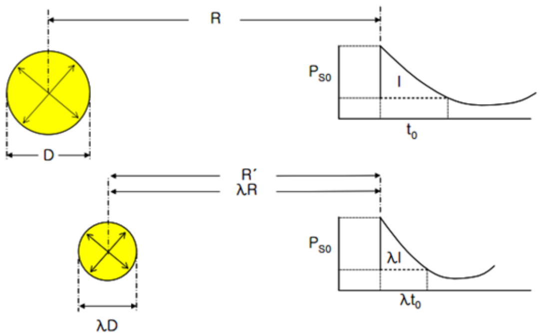


Fig. 2: Hopkinson–Cranz scaling law [10]

Explosion parameters, as well as suitable equation for modelling of blast pressure on objects and structures, have been the target of many scientific studies over the past half a century [11-14].

For example, the peak incident pressure P_{so} was firstly expressed using the scaled distance (Z) based on numerical modelling by Brode in 1955 as follows:

$$P_{SO} = \frac{0.975}{Z} + \frac{1.455}{Z^2} + \frac{5.85}{Z^3} - 0.019\ bar \qquad (2)$$
$$0.1\ bar < P_{SO} < 10\ bar$$

$$P_{SO} = \frac{6.7}{Z^3} + 1\ bar \qquad (3)$$
$$P_{SO} > 10\ bar$$

This relation was subsequently reviewed by Smith [7] who compared Brode's model with results obtained from more recent experimental studies. The comparison displays excellent agreement between the models in the far-field. However, Brode's model tends to be conservative in the near field (Z<<1). Another well-known relation is proposed by Henrych [12]:

$$P_{SO} = \frac{14.072}{Z} + \frac{5.54}{Z^2} + \frac{0.357}{Z^3} + \frac{0.00625}{Z^4} \qquad (4)$$
$$0.05 < Z < 0.3$$

$$P_{SO} = \frac{6.194}{Z} + \frac{0.326}{Z^2} + \frac{2.132}{Z^3} \qquad (5)$$
$$0.3 < Z < 1$$

$$P_{SO} = \frac{6.662}{Z} + \frac{4.05}{Z^2} + \frac{3.288}{Z^3} \qquad (6)$$
$$1 < Z < 10$$

Unlike Brode's relation, the equations recommended by Henrych has good agreement with experimental results in the near-field blast.

Kingery and Bulmash [13] provided equations for predicting the parameters of air explosion due to spherical and semi-spherical charges ranging from 1 kg of TNT to 3 tons of TNT. In this method, the blast pressure reduces according to the following equation which is used in this study:

$$P(t) = P_{S0}\left[1 - \frac{t - T_A}{T_0}\right] e^{\left[\frac{A(t-T_A)}{T_0}\right]} \qquad (7)$$

Where P(t) is the pressure at time t, P_{so} is the maximum impact pressure, T_0 is the duration of the positive phase and A is the reduction coefficient. In newer versions of ABAQUS software, the feature known as CONWEP blast loading is added. To use it, it is necessary to point out the location of explosion, the levels on which the explosion pressure should be applied, and the weight of the explosive charge to the software. In this paper the aforementioned method is used to apply the blast loading.

Since the behavior of connection has significant effects on the overall stability and efficiency of the structures, studying the behavior of different structural connection is of great interest [15-17]. Many studies have emphasized the vulnerability of steel connections, especially in the field of seismic design of structures [18-24]. Early investigations generally focused on the behavior of connections under cyclic loading. In one of the earliest studies, Popov [20] performed experimental tests to investigate the cyclic behavior of steel moment-resisting connections. Following the January 17, 1994, Northridge Earthquake and observing extensive damage in beam-to-column moment connections, a great deal of testing and research has been done to gain a better understanding of the behavior of connections [18, 22-25] leading to new paradigm in seismic design codes. Although suggestions on improving the seismic behavior of steel connections can also be helpful in blast-resistance structures, in these studies, many critical factors in the nonlinear behavior of connections such as severe torsion, lateral bending and the effects of high strain rate on materials have not been considered. Since the September 11th terrorist attack on the World Trade centers, there has been a growing interest all around the world over the safety of buildings subjected to blast loads. In addition, structural retrofits of buildings has been of great interest to increase the protection level of occupants from potential terrorist bombing attacks. Generally retrofit strategies can be categorized into three groups: (i) strengthening concepts, (ii) shielding concepts, and (iii) concepts to control hazardous debris [26]. Since the beam-to-column connections do the important task of transferring load between different structural members, understanding of their behavior under blast loading is decisive. Nevertheless, studies investigating the response of the building components under blast load are quite limited and include some case studies based on past blast attacks on buildings focusing on the pattern and intensity of blast damages formed in the structures, [27, 28], experimental tests [2], and finite element modeling [4, 29, 30]. There is also very little test data available on the behavior of steel connections loaded by blast directly. On the other hand, some studies put the focus on the macro-scale pattern and intensity of damages in past blast attacks rather than identifying the behavior of individual structural components under blast loads [31].

Krauthammer et al. conducted one of the leading studies to investigate the blast effect on steel connections [32]. In their study, they noticed brittle failures and significant deformations in connection components caused by high-rate loading. So they investigated the strain rate effect on numerical examples [32, 33]. They [19] also investigated blast-resistant steel and concrete connections using a numerical method. In their research, the criteria mentioned in the TM5-1300 code [34] and their accuracy were examined. The results of this study proved the importance of considering the effect of strain rate on the behavior of materials. It was also observed that in the models considering the dead load, the vertical deformations of the beams and columns are significantly higher than the case where the dead loads were omitted, and therefore the need for considering the dead load in the dynamic analysis was emphasized. It was also concluded that the provisions of Code TM5-1300 are not sufficient for extremely dynamic loads.

Sabuwala et al. [29] carried out one of the most important studies on investigating the effects of blast load on the behavior of connections. They investigated the effects of the blast on the welded beam column connections using the finite element method. The purpose of this study was to analyze and evaluate the behavior of steel joints against blast load by finite element simulation and the adequacy of seismic design for blast loading. They also provide suggestions for modeling steel connections under blast loading. Three different types of connections were studied with different details, one of which was related to the details of the pre-Northridge earthquake design, and the rest were in regard to the post-Northridge earthquake resistant design provisions. ABAQUS software was used in this research for performing cyclic static tests on the connection, and the finite element model was verified by comparing the numerical and test results. Explosion loading was obtained using TM5-1300 code. To obtain the failure mechanism, stress and strain control were used at critical points of the structure. The results of this simulation showed that the criteria presented in the code are conservative for the members of the structure under the blast load, because the maximum values of displacement and period obtained from modeling were lower than the limit values specified in the code. It was also found that the reinforced connections have better performance than the connections designed according to the pre-Northridge earthquake design process, and the seismic design recommendations proposed following the Northridge earthquake to improve the blast performance of connections. In addition, low groove welding in the unreinforced connections was identified as a critical point that was consistent with the test observations. The use of flange cover plates in the reinforced connections effectively reduce the stress concentration in the fillet welds. They also shift the plastic joints away from the connection to the end of the cover plates.

This paper investigates the behavior of a braced frame structure under blast loading. The studied structure is selected from a hospital building in Tabriz, Iran. Firstly, the frame is investigated under blast load and critical regions with considerable stress and plastic strain are found. Then according to the stress and plastic strain contour, five different possible reinforcing scenarios are proposed. These scenarios include combination of using web stiffeners, triangular stiffeners at the top and bottom flanges, using continuous sheet in web and flexural reinforcement of connected beam. The effects of adding different components in connections on stress and the plastic strain contours are investigated. Finally best scenarios are proposed.

2. Finite Element Model

This paper investigates five different blast rehabilitation scenarios for a six-story braced frame subjected to near-field blast load. Fig. 3 shows the studied frame. The building is a six-story braced steel structure adopted from a hospital building. The selected frame is the most vulnerable frame against the most likely scenario of an explosion in a nearby street. In the first step, before applying the blast load, a gravity load of 28 KN/m was applied on the beams of all floors, which is a suitable load for a hospital structure. The studied frame with all the details of the connections is modelled in ABAQUS. The details of connections are presented in Fig. 4.

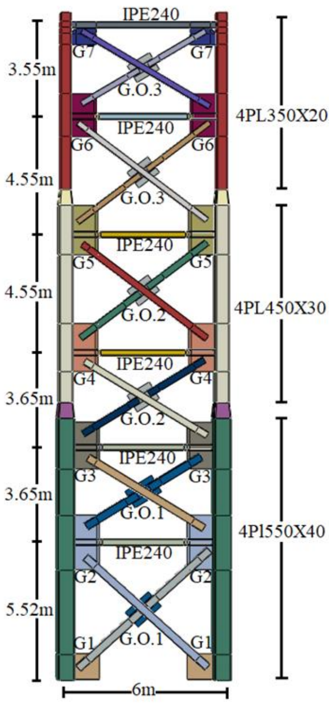
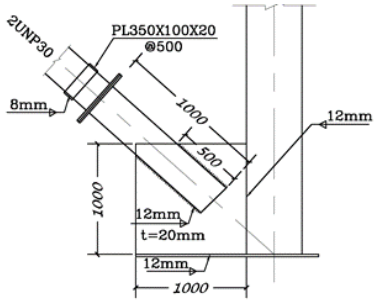
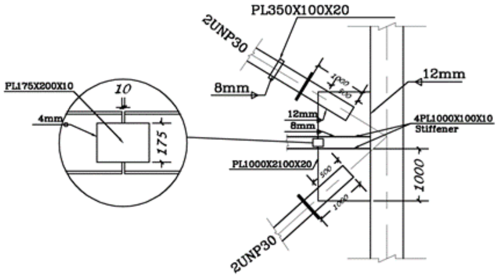


Fig. 3: 6-Story studied frame

A 1,000 Kg explosive charge of TNT, which is a conventional load for vehicle carrier, was detonated 5 meters from the structure. Considering the high pressure of blast, (28 kg/cm2), and the details of the wall-to-frame connections, the walls will be completely detached from the structure and destroyed at the very first moment. So they are not existent in the numerical model.



G.1



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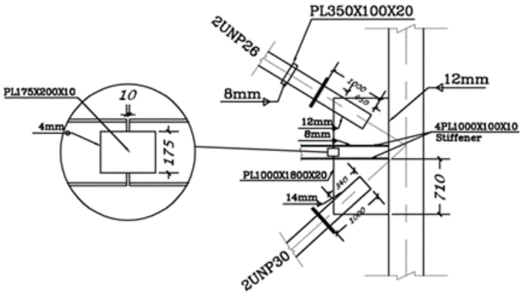

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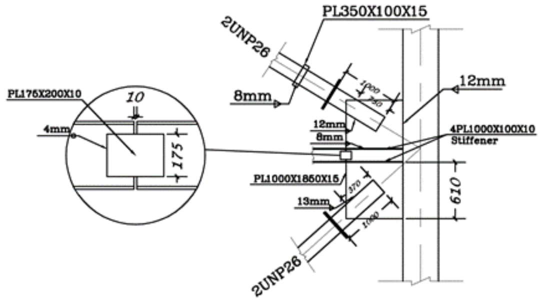

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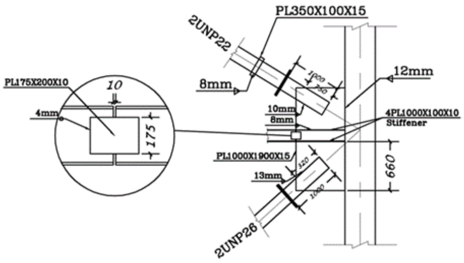

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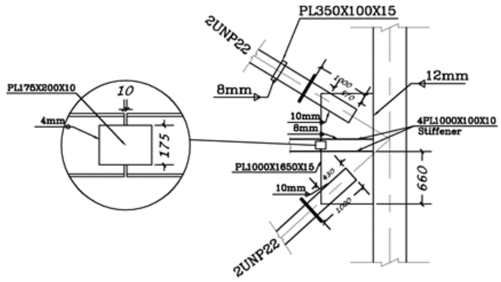

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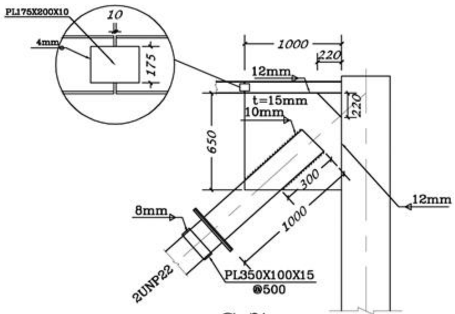

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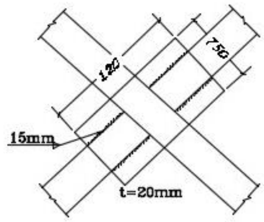
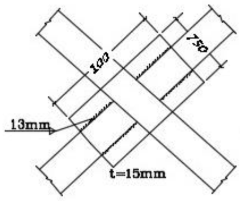
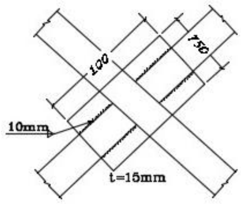


G.O.1



G.O.2



G.O.3

Fig. 4: Details of connections

Explicit solver is a finite element analysis product that is particularly well-suited to simulate brief transient dynamic loads which is used in this study. 8-node cubic solid elements mesh geometry are used to model the steel components of the structure. The average size of mesh is taken to be 2cm. The interaction between the different components (welds, columns, beams) is of “tied” type which makes the displacement degrees of freedom, U_x, U_y, U_z tied together.

2.1 Material Properties

The elasto-plastic isotropic model is used in this study to define the material properties. In isotropic hardening, the shape of the yield surface does not change but expands in the post-yield region as shown in Fig. 5. The von-Mises yield criterion is also used suggesting the yielding of materials begins when the second deviatoric stress invariant J_2 reaches a critical value K, which leads to a yield function as:

$$f(\sigma_{ii}.K_i) = \sqrt{3J_2} - \sigma_Y - K = 0 \qquad (3)$$

Table 1 represents the material properties used in this study.

Table 1: Material Properties

Type	Yield Stress (Mpa)	Ultimate Stress (Mpa)	Ultimate Strain
Steel A36	250	420	0.15
Weld	360	515	0.09

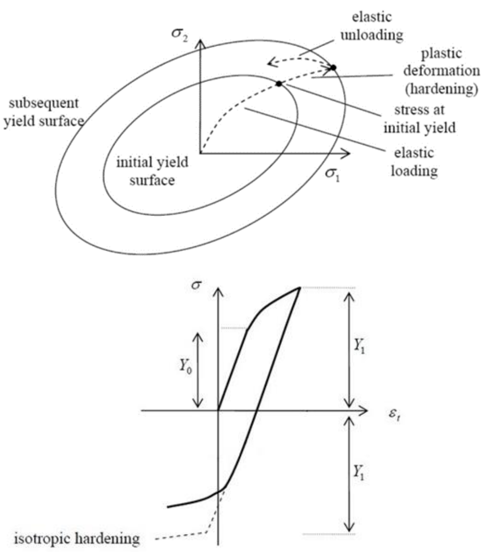


Fig. 4: Yield surface in isotropic hardening (first) and typical stress-strain curve (second)

The mechanical properties of structural materials change in high loading rates. The effect of high-rate strain on the mechanical properties of steel is an essential part of a blast-resistant design. It is shown that as the strain rate increased, the yield and ultimate stresses increases, while ductility decreases. The most common way is employing dynamic increase factors (DIFs) to address the increase in yield stress due to high-rate loading. A DIF is a ratio representing the increased strength (due to high-rate loading) to the static strength (e.g., the dynamic to static yield stresses). Krauthammer et al. [35, 36] investigated the recommended DIF values in TM 5-1300 and the effects of high rate dynamic loadings on structural responses for both the design and the numerical simulation and validated the factors proposed by the codes for modeling the behavior of connections under blast loading. In this study the values of 1.3 and 1.1 are considered for yield stress and ultimate stress.

3. Rehabilitation Scenarios

Fig. 6 displays the von-Mises equivalent stress for the studied frame and the connections of the first floors. Fig. 7 represents equivalent plastic strain contour. It is worth mentioning that in this paper the standard system (SI units) is used in modeling and presenting the results in all figures and tables.

The stress contour indicates that stresses in the braces of the first two stories almost exceed the yielding stress. Other story braces are also under significant tension. Nevertheless, these stresses are still far from failure. The welds of the connections are in their ultimate capacity. However, it cannot be the decisive point in the structure because of elastic behavior of some sections along the welds. So it is unlikely that these connections experience failure due to welds fracture. Plastic strain contour also reveals significant local plastic strains in the web of the first and second story beams and upper and lower regions of the gusset plates alongside the column. There is also a significant local stress concentration on the side of the column towards the explosive charge. In addition, asymmetric damage and displacement field of far and near sides of the structure is noticeable.

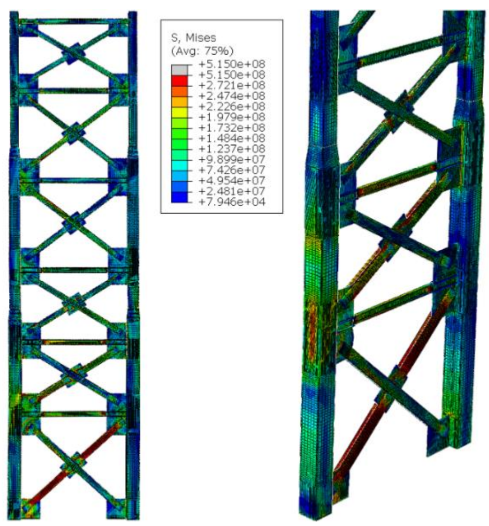


Fig 6: von-Mises equivalent stress for the studied frame and the connection of the first floor (SI Units)

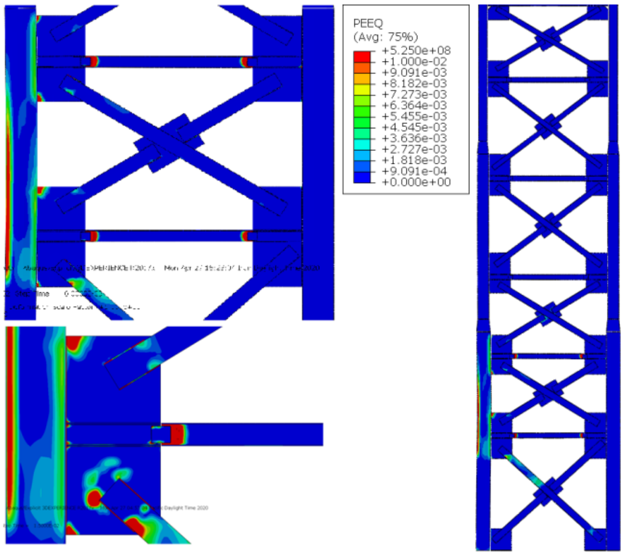


Fig. 7: equivalent plastic strain contour

The connections are strengthened with five scenarios presented in Fig. 8. Each scenario is chosen according to the results observed in the previous ones to try to eliminate the drawbacks by providing new and different suggestions. Using the results and observations of this study will provide an excellent view of the effects of the different methods in connection rehabilitation. It also demonstrates how applying changes in the connections or the connected members affect the overall behavior of the frame members. The specifications of stiffener plates are presented in Table 2. These reinforcing plates are applied in first and second stories connections as they are critical regions. The numbers of used plates of each kind are presented in Fig. 6 as well.

Table 2: stiffeners and plates specifications (mm)

Triangular Stiffeners (P1)	20*20*200 + 282*106*6.2
Web Stiffeners (P2)	220*57*20 & 220*40*20
Gusset stiffener (P3)	412*220*20
Continuous Web Plates (P4)	3162*220*10
Flange Plates (P5)	3350*160*15

4. Results and Discussion

Table.3 summarizes all the scenarios with their pros and cons. Fig. 9 and Fig.10 display the von-Mises equivalent stress and equivalent plastic strain contours for all proposed scenarios.

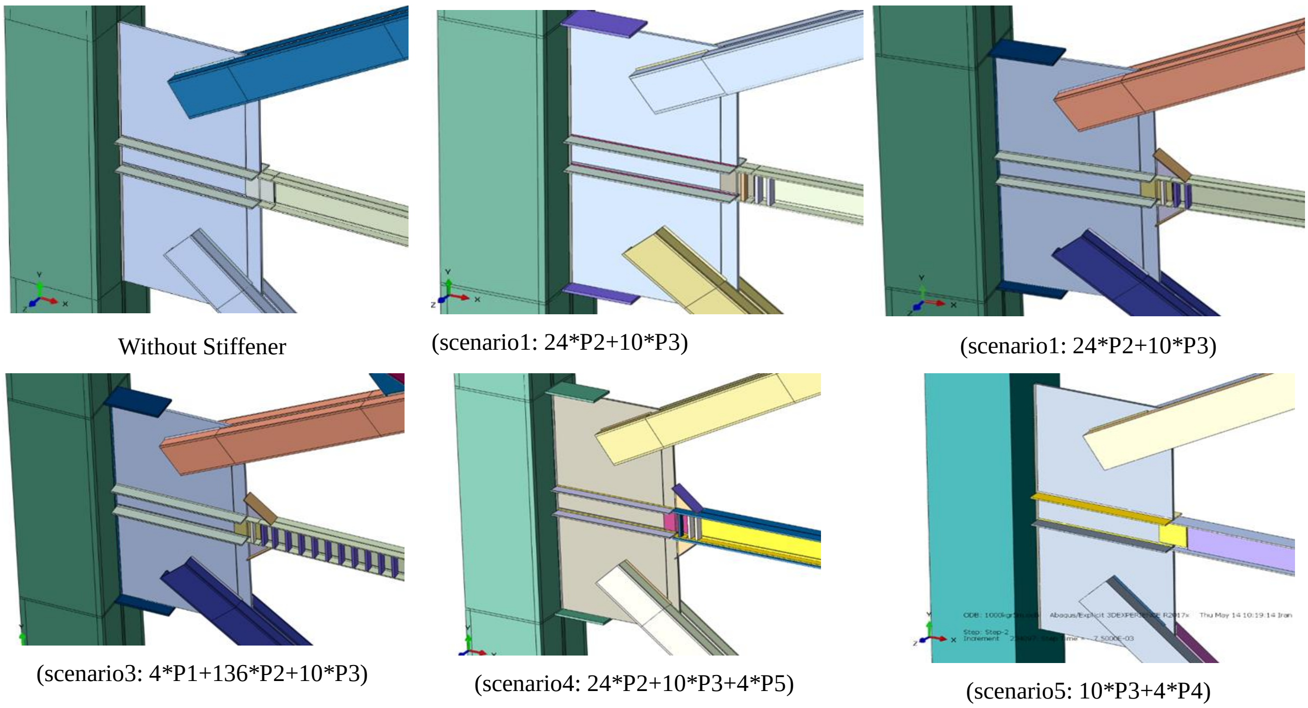
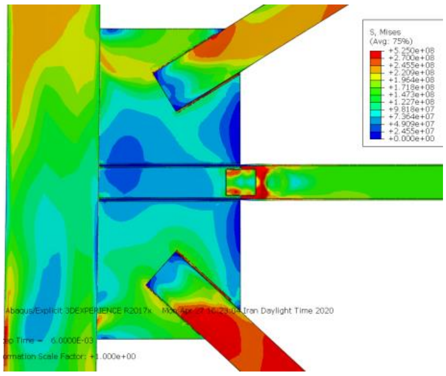


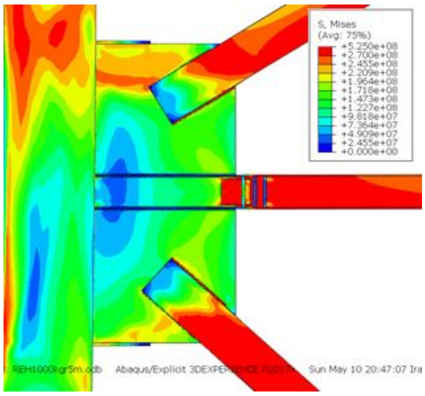
Fig. 5: rehabilitation Scenarios

Table 3: studied scenarios with their pros and drawbacks

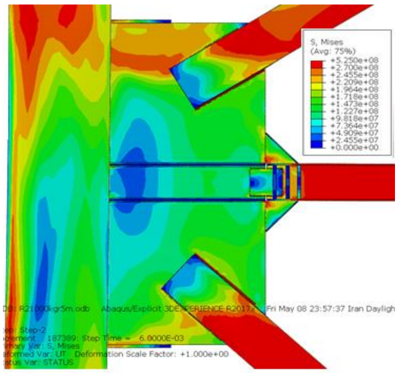
NO	Scenario description	Pros	Drawbacks
Scenario1	Due to the large plastic strains in the web and the lower and upper regions of the gusset plate in the first two stories, three web stiffeners and two horizontal gusset stiffener at the top and bottom of the gusset plate are used	The two horizontal stiffeners at the top and bottom of the gusset plate effectively reduce the plastic strain in the adjacent areas and the middle of the gusset plate on the column side compared with the initial connection	Despite the slight reduction of stress concentration and plastic strains in the vicinity of web stiffeners, there isn't any noticeable improvement between web stiffeners. Using of web stiffeners also extends the yielding toward the middle part of the web
Scenario2	Two triangular stiffeners are also used in addition to three web stiffeners and two horizontal gusset stiffeners to increase the shear capacity	The triangular stiffeners notably reduce stresses and plastic strain in the reinforced region. The stresses in the part of the beam reinforced simultaneously by the vertical web stiffeners, and triangular plates are in acceptable range	They make critical region move in beam right after the stiffener plates
Scenario3	According to the transition of the critical stress region to the middle of the beam in Scenario2, vertical stiffeners are extended along the entire length of the web of the first two floors beams.	Vertical web-stiffeners control the plastic strain in limited vicinity around them, they are not useful for the regions between two stiffeners	Despite existing web stiffeners throughout the entire length of the beam, there is no significant improvement compared with scenario 2. there are relatively large stresses and plastic strains in the first third of the beam webs
Scenario4	The beams are reinforced with continuous flange plates to improve the flexural capacity of the connected beam due to the probable increase in moment caused by the triangular and web stiffener plates	Stresses in the middle areas of the beam outside the range of stiffeners reasonably reduces. forces and stresses in braces and their connecting plates are reduced	The beginning and end of the beam at the connections are under severe stress and the damage also extends to the triangular stiffener plates and the flange of the beam The stress on the side of the column that is connected to the gusset plate significantly increases
Scenario5	Due to the need for the shear strengthening of the beam, the web is reinforced by using two continuous plates on both sides	The stresses and plastic strains of the beam web significantly decreases	The stress increase in the web-to-gusset connecting plate



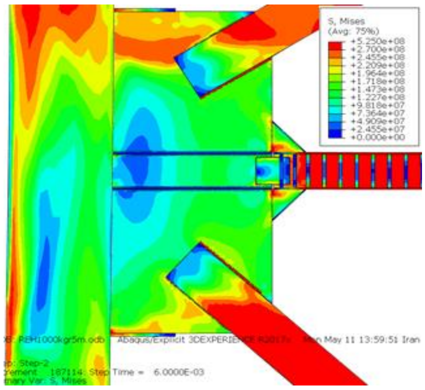
Without Stiffeners



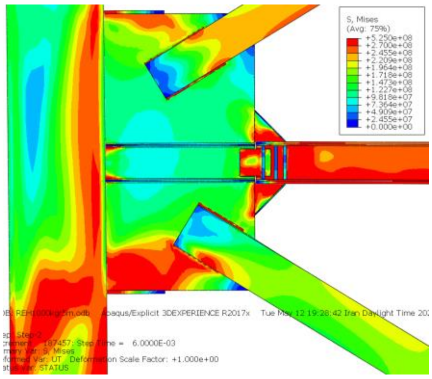
(Scenario 1)



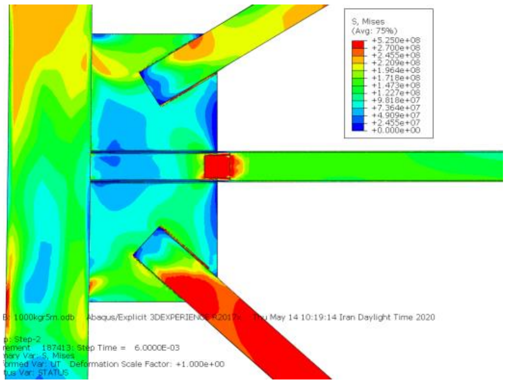
(Scenario 2)



(Scenario 3)

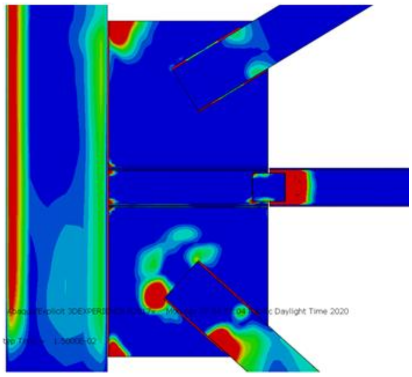


(Scenario 4)

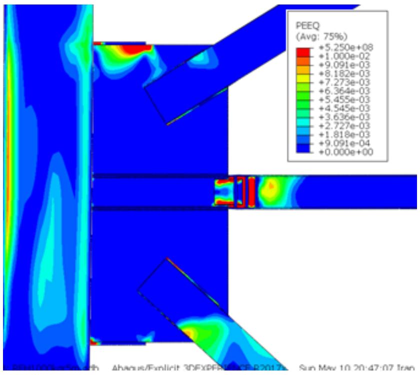


(Scenario 5)

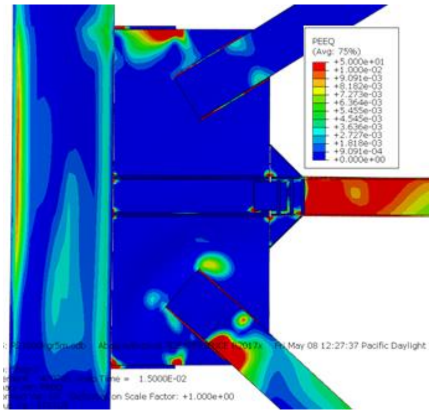
Fig 6: Equivalent von-Mises stress contour for five studied scenarios



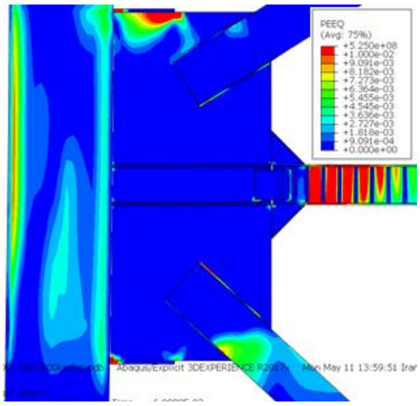
Without Stiffeners



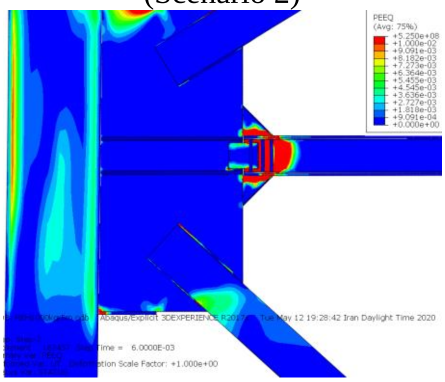
(Scenario 1)



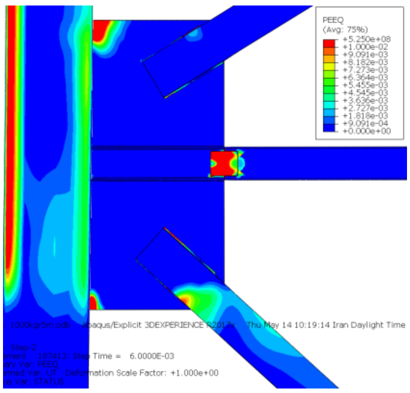
(Scenario 2)



(Scenario 3)



(Scenario 4)



(Scenario 5)

Fig 7: Equivalent plastic strain contour for five studied scenarios

4. Conclusion

In this study a five-story steel braced frame was studied under near-field blast load. Based on the observed damages, the studied frame was reinforced with stiffener plates in five different scenarios. These reinforcing plates were applied in first and second stories connections because of being critical regions. In each case, the stresses and the plastic strains were investigated. The following results were obtained based on observations:

1- Frame members in the side toward the explosion experience more severe local damage, and asymmetrical deformations and damage are notable.
2- The damage in the structure decrease rapidly as the height increases.
3- The stress and plastic strain contour imply that although vertical web stiffeners and triangular plates on the top and bottom flanges eliminate the severe plastic strain and stress concentration in the connection zone, they make the critical regions move toward the middle of the beam.
4- The use of web stiffeners along the entire web along with triangular plates just controls the plastic strains in very limited regions around the stiffeners.
5- Flexural reinforcement of beams connected to joints reinforced with triangular plates has no effect on improving its performance and may lead to further damage to the stiffener plates and the gusset plate.
6- The use of triangular stiffeners at the top and bottom of the beam flange at the joint increases the joint stiffness and thus increases the stress on the gusset plate at the column side.
7- Continuous reinforcement of the beam webs with web sheets significantly reduces the stresses on the web and is a more effective way than the web stiffeners. Also, such an approach creates fewer stress changes in other members due to minor changes in the stiffness of the connected members.

Conflict of Interest

On behalf of all authors, the corresponding author states that there is no conflict of interest.

References

[1] Khandelwal, K., S. El-Tawil, and F. Sadek, Progressive collapse analysis of seismically designed steel braced frames. Journal of Constructional Steel Research, 2009. 65(3): p. 699-708.

[2] Astaneh-Asl, A., et al., Use of catenary cables to prevent progressive collapse of buildings. Report No.: UCB/CEE-STEEL-2001/02, 2001.

[3] Marjanishvili, S., Progressive analysis procedure for progressive collapse. Journal of Performance of Constructed Facilities, 2004. 18(2): p. 79-85.

[4] Liu, J., Preventing progressive collapse through strengthening beam-to-column connection, Part 1: Theoretical analysis. Journal of Constructional Steel Research, 2010. 66(2): p. 229-237.

[5] Ellingwood, B.R., Mitigating risk from abnormal loads and progressive collapse. Journal of Performance of Constructed Facilities, 2006. 20(4): p. 315-323.

[6] Ellingwood, B.R. and E. Leyendecker, Approaches for design against progressive collapse. Journal of the Structural Division, 1978. 104(3): p. 413-423.

[7] Smith, S.J., D.M. McCann, and M.E. Kamara, Blast resistant design guide for reinforced concrete structures. 2009.

[8] Cranz, C., Lehrbuch der ballistik. Vol. 1. 1925: Рипол Классик.

[9] Hopkinson, B., British ordnance board minutes 13565. The National Archives, Kew, UK, 1915. 11.

[10] Chun, S., Nonlinear fluid-structure interaction in a flexible shelter under blast loading. 2004, Virginia Tech.

[11] Brode, H.L., Numerical solutions of spherical blast waves. Journal of Applied physics, 1955. 26(6): p. 766-775.

[12] Henrych, J. and R. Major, The dynamics of explosion and its use. Vol. 569. 1979: Elsevier Amsterdam.

[13] Kingery, C.N. and G. Bulmash, Airblast parameters from TNT spherical air burst and hemispherical surface burst. 1984: US Army Armament and Development Center, Ballistic Research Laboratory.

[14] Smith, P. and J. Hetherington, Blast and ballistic loading of structures. Laxtons. 1994, Oxford.

[15] Kazerani, S., N. Fanaie, and S. Soroushnia, Seismic behavior of drilled beam section in moment connections. Numerical Methods in Civil Engineering, 2017. 1(4): p. 1-6.

[16] Fanaie, N., S. Kazerani, and S. Soroushnia, Numerical study of slotted web drilled flange moment frame connection. Numerical Methods in Civil Engineering, 2017. 1(3): p. 16-23.

[17] Ghalamzan Esfahani, F. and N. Fanaie, Finite element analysis of a rigid beam to column connection reinforced with channels. Numerical Methods in Civil Engineering, 2017. 2(1): p. 37-48.

[18] Engelhardt, M. and T.A. Sabol, Testing of welded steel moment connections in response to the Northridge earthquake. Northridge steel update, 1994. 1.

[19] Tamboli, A.R., Handbook of structural steel connection design and details. 1999: McGraw-Hill New York.

[20] Popov, E.P., Seismic moment connections for moment-resisting steel frames. 1983: Earthquake Engineering Research Center, University of California.

[21] Engelhardt, M.D. and T.A. Sabol, Reinforcing of steel moment connections with cover plates: benefits and limitations. Engineering structures, 1998. 20(4-6): p. 510-520.

[22] Engelhardt, M.D. and T.A. Sabol, Seismic- resistant steel moment connections: developments since the 1994 Northridge earthquake. Progress in structural engineering and materials, 1997. 1(1): p. 68-77.

[23] Engelhardt, M. and A. Husain, Cyclic-loading performance of welded flange-bolted web connections. Journal of Structural Engineering, 1993. 119(12): p. 3537-3550.

[24] Civjan, S.A., M.D. Engelhardt, and J.L. Gross, Retrofit of pre-Northridge moment-resisting connections. Journal of Structural Engineering, 2000. 126(4): p. 445-452.

[25] Jones, S.L., G.T. Fry, and M.D. Engelhardt, Experimental evaluation of cyclically loaded reduced beam section moment connections. Journal of Structural Engineering, 2002. 128(4): p. 441-451.

[26] King, K.W., J.H. Wawclawczyk, and C. Ozbey, Retrofit strategies to protect structures from blast loading. Canadian Journal of Civil Engineering, 2009. 36(8): p. 1345-1355.

[27] Caldwell, T., Bomb blast damage to a concrete-framed office building Ceylinco House — Columbo, Sri Lanka, in Proceedings of Structures Congress. 1999: New Orleans, LA.

[28] Corley, W.G., Lessons learned on improving resistance of buildings to terrorist attacks. Journal of Performance of Constructed Facilities, 2004. 18(2): p. 68-78.

[29] Sabuwala, T., D. Linzell, and T. Krauthammer, Finite element analysis of steel beam to column connections subjected to blast loads. International Journal of Impact Engineering, 2005. 31(7): p. 861-876.

[30] Urgessa, G.S. and T. Arciszewski, Blast response comparison of multiple steel frame connections. Finite Elements in Analysis and Design, 2011. 47(7): p. 668-675.

[31] Caldwell, T. BOMB BLAST DAMAGE TO A CONCRETE-FRAMED OFFICE BUILDING-CEYLINCO HOUSE-COLOMBO, SRI LANKA. in 1999 New Orleans Structures CongressStructural Engineering Institute of American Society of Civil Engineers, Structural Association of Alabama, National Council of Structural Engineers Associations, Florida Structural Engineers Association, Louisiana Section of ASCE, Baton Rouge Branch of AStructural Engineering Association of American Society of Civil Engineers, Structural Engineers Association of Alabama, National Council of Structural Engineers Associations, Florida Structural Engineers Association, Louisiana Section of ASCE, Baton Rouge. 1999.

[32] Krauthammer, T., Blast-resistant structural concrete and steel connections. International Journal of Impact Engineering, 1999. 22(9-10): p. 887-910.

[33] Krauthammer, T., Structural concrete and steel connections for blast resistant design. Int J Impact Eng, 1999. 22(9-10): p. 887-910.

[34] Explosions, A., TM 5-1300. The Design of Structures to Resist the Effects of US Department of the Army, Navy, and Air Force, 1990.

[35] Krauthammer, T., J. Lim, and G. Oh. Lessons from using precision impact test data for advanced computer code validations. In 10th International Symposium on Interaction of the Effects of Munitions with Structures. 2001.

[36] Krauthammer, T., et al. Three-dimensional structural steel frame connections under blast loads. In Proc. 73 Shock and Vibration Symposium, Shock rd and Vibration Information Analysis Center, Newport, RI. 2002.