Journal of Civil Engineering and Architecture 18 (2024) 530-539
doi: 10.17265/1934-7359/2024.11.003

Effect of Variable Concrete Strength on the Behavior of Buckling Restrained Braced Frames under Blast Load

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Abstract: BRBF (buckling restrained braced frame) is a relatively new lateral force resisting system for building structures. BRBFs are used mostly to resist seismic force due to their high ductility after yielding and the ability to absorb higher strain energy. ASCE (American Society of Civil Engineers) first permits the use of BRBFs as a single seismic force-resisting system by quantifying the seismic parameters such as response modification coefficient (R), over-strength factor (Ω_o) and deflection amplification factor (C_d) for a structure built with BRBF, in their 2010 code (ASCE 7-10). But it has not been investigated how a structure built with BRBF, which is primarily designed to resist seismic force, and behaves under sudden occurrence of a blast load. This research investigates the performance of a BRBF subjected to blast loading. In other words, this paper focuses on the effect of blast loading on BRBF. The architype for this investigation is a chevron type braced frame. The frame is subjected to a short duration blast load that lasts only 21.7 mili-second (ms). Blast loading effects on the braced frame are assessed by identifying the weakest plane of failure, deformation characteristics and out of plane bending. The research investigates how the properties of the surrounding concrete, especially compressive strength, affect the overall strength of the BRBF on resisting blast loading. It is observed that the compressive strength of the surrounding concrete plays a significant role in reducing the deformation characteristics, both in-plane and out-of-plane.

Key words: BRBF, blast loading, response modification coefficient (R), chevron brace.

1. Introduction

BRBFs (buckling restrained braced frames) are lateral force-resistant systems commonly used to provide lateral bracing for structures in seismic events. BRBs (buckling restrained braces) are used in various applications such as metal buildings, single brace retrofitting projects, high-rise outrigger systems [1]. So far, the use of BRBFs have been restrained for seismic design. Apart from Chavarian et al. [2], no significant research has been done to analyze and assess the behavior of the BRBF under blast loading. Because of the high use of the BRBFs in the industry, an explosive attack on the system component could prove to be devastating. The goal of this research is to understand the performance of BRBFs under blast loading when they are used in a building structure primarily designed for seismic forces. The research aims to aid in the development of blast resistant design of BRBs focusing on relevant implications such as failure modes and collapse mechanism.

1.1 BRBF

CBFs (concentric braced frames) and MRFs (moment resisting frames) are efficient lateral force-resisting systems in building structures. After 1994 Northridge earthquake, a significant number of seismic force-resisting systems comprising mainly moment frames and concentric brace frames, experienced severe damage, requiring extensive repair. During a major ground motion (earthquake event), the bracing members undergo large deformations in cyclic tension and compression into the post-buckling range, causing large reversed cyclic rotation at the plastic hinges

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formed in the brace members and in the connections.

Buckling of conventional braces under compression, leads to a sudden loss of stiffness and progressive degrading behavior which limits the amount of energy dissipation. To overcome these deficiencies researches have focused on relatively new type of bracing system named BRBFs. BRBFs were first utilized in North America recently [3]. BRBFs are a special class of concentrically braced frames that are composed of columns, beams and braces which are mainly under axial forces. Neither X-bracing nor K-bracing configuration is used for BRBFs and is mostly utilized in SDCs (Seismic Design Categories) D, E, or F. BRBs consist of a steel core, a buckling restraining system (concrete or grout) which effectively reduces the un-braced length of the compression member and eliminates the buckling failure mode and prevents composite action between the steel core and the surrounding concrete [4]. Figs. 1a and 1b show steel core and brace cross section respectively that have been used for this study. Bonding of the steel core to the concrete is precluded to ensure that each element, specifically steel core, behaves separately and to prevent composite action that would change the brace behavior to composite brace manner.

1.2 Quantifying Blast Load on Structures

Blast is a rapid release of energy resulting in a pressure wave that propagates outward from the source [5-7]. The resultant load is produced by the rapid expansion of the energetic material which creates a pressure disturbance or blast wave radiating away from the explosion source which includes high explosives, propellants, and other gases propagating as a shock wave with supersonic speed that are released suddenly. The impact of blast load on a building structure is quantified using a method suggested by Dusenberry [6] and adopted by ASCE [8].

To understand blast load effect on a BRBF, a one-story single bay rectangular building of equal wall lengths is considered in this research. The elevation and the plan views of the whole frame system are shown in Fig. 2. For simplification, it was assumed that the BRBF in the middle would take one-half of the equivalent blast load applied on a 20 ft tributary area (10 ft from the left and right walls each). The equivalent blast impulse load is calculated based on a tributary area of 200 sq. ft. (20 ft width × 10 ft height) and applied on the flange of the left column of the BRBF.

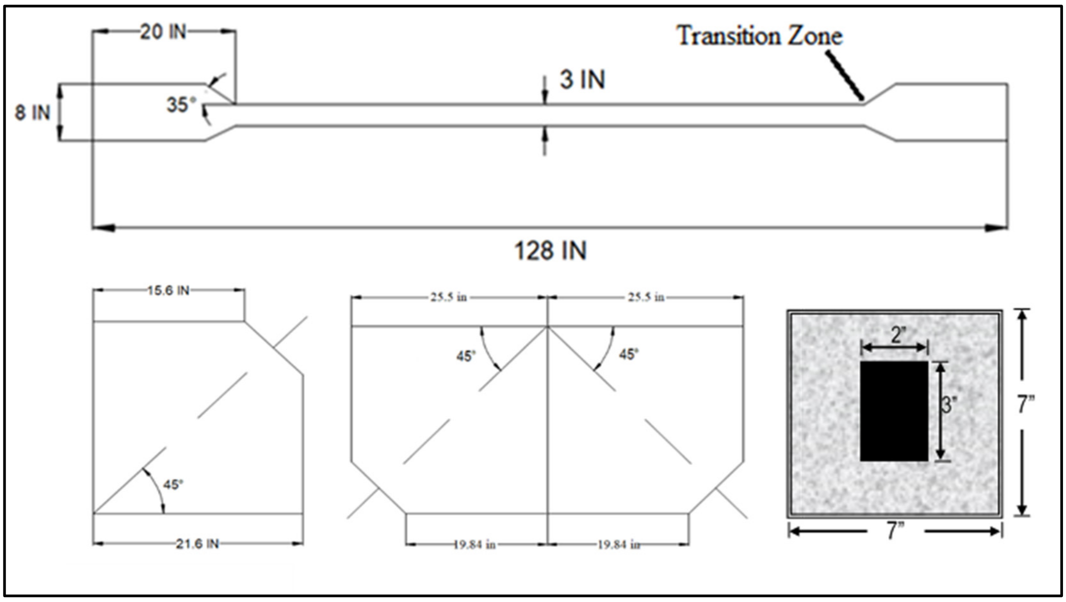

Fig. 1 The dimensions of the members of the BRBs, (a) steel core plate of the BRBs, (b) gusset plate, (c) steel core, concrete and steel casing of the BRBs.

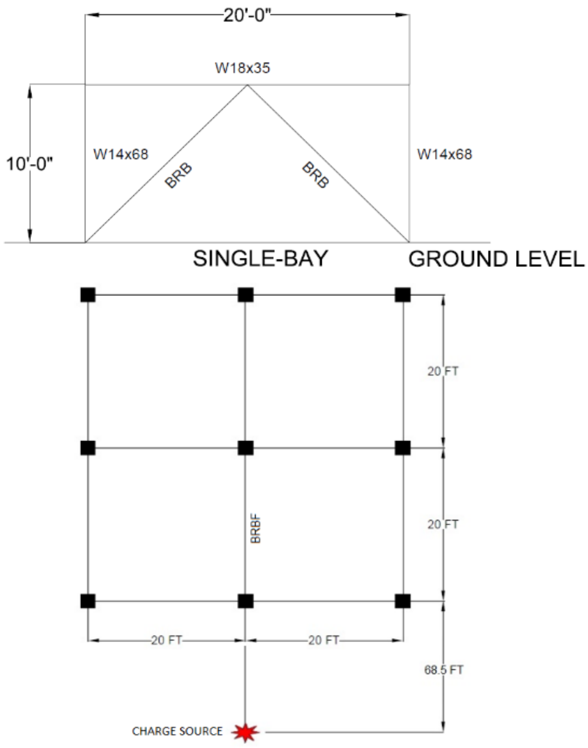

Fig. 2 Elevation and plan views of the schematic building with BRBF.

This equivalent impulse load with the tributary width factored can be seen in Fig. 3. In Fig. 3, the equivalent time dependent impulse load (stress) is calculated and applied at the left side flange of the left column. The total time for the application of the load is 21.7 mili-second (ms). Fig. 4 shows the application of the impulse load on the left column of the braced frame.

2. Modeling and Analyses

The base model consists of a single-bay chevron frame with rectangular BRBs encased by concrete around the steel core. Equivalent blast impulse load (stress) is applied at the left column and explicit dynamic analysis is performed to evaluate the behavior of the BRBF. During the process, the strength of the concrete encasing the central core brace is varied to observe the influence of concrete strength on the performance of the BRBF.

2.1 Modelling of the Frame Components

The chevron BRBF model is composed of two columns of the same size (W14×68), a beam (W18×35), two BRBs, stiffeners on the column, and gusset plates to attach the members. The beam-to-column connections are kept rigid. A working span of 20 feet was used for the beam and a working height of 10 feet for the columns. The properties and dimensions of these shapes are obtained from AISC Steel Construction Manual (16th Edition) [9]. The steel brace core dimensions are 3″ (width) × 2″ (thickness) making the total cross-sectional area to be 6 square inches. The total length of the brace from connection end to connection is 128 inches.

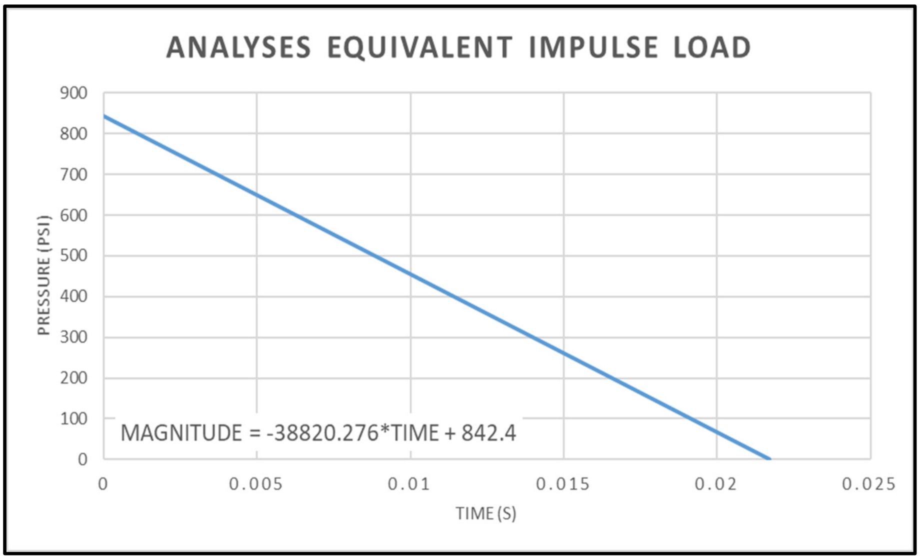

Fig. 3 Equivalent impulse load applied on the BRBF.

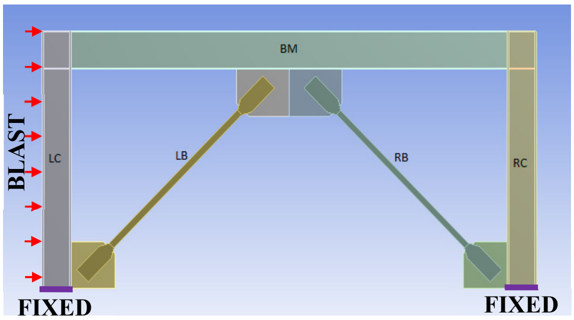

Fig. 4 Undeformed chevron BRBF with distributed blast load on left column.

The length of the steel core is 88 inches. For the concrete encasing, a rectangular HSS (Hollow Structural Section) section was used. The steel casing had a ¼″ thick wall. The gusset plates were used to attach the braces onto the columns and beam. The gusset plate dimensions are shown in Fig 1. The thickness of the gusset plate is two inches consistent with the depth of the steel brace.

2.2 Material Model

The materials are modeled as non-linear elastic due to high energy in a blast load. A36 (f_y = 36 ksi) and A992 (f_y = 50 ksi) steels are used for the braces and the W-flange sections for beam and columns respectively. The steels were modeled as multi-linear with isotropic strain hardening. True stress and strain behavior of steel material is used for realistic results [2]. Concrete is a very difficult material to model due to the fact that its strengths in compression and tension are different. The concrete model used in the analyses is based on the RHT (Riedel-Hiermaier-Thoam) material model. The RHT default model for concrete in Ansys has been used in this research. Hu et al. [10] first introduced RHT model for concrete.

3. Analyses

Fig. 4 shows an undeformed chevron type BRBF frame that is modeled in SolidWorks and analyzed in Ansys Workbench with the blast load applied on the left column calculated from the building configuration. Ansys explicit dynamic analysis is used to determine the response of the structure due to a rapidly time dependent loading along with non-linear material. Explicit dynamic analysis involves simulating an explosive load for a duration of 21 milliseconds. The explicit dynamic (LS-DYNA Export) system uses an explicit integration method as stated by Lee [11].

4. Results and Discussions

The research was aimed at studying the effect of strength of concrete, which encases the steel core brace, on the performance of BRBFs under blast loading. Parametric studies were conducted to evaluate the performance of the BRBF and its component elements. Analyses were performed to investigate the effect of concrete strength on possible failure modes of the BRBF as a whole and progressive collapse of the component elements such as the concrete encasing and the steel brace core. For this investigation, we analyzed the BRBF varying the concrete strengths to be 31.2 MPa (4,500 psi), 40 MPa (5,800 psi), 48 MPa (7,000 psi), 69 MPa (10,000 psi) and 140 MPa (20,000 psi). The objective of this study is to understand how the concrete strength of the braces influences the formation of the plastic hinge and also the progressive collapse of the frame. In the following sections, we will discuss the behavior of the entire BRB frame, the left column with the formation of plastic hinge and subsequent crack formation around the column base and core brace, under blast loading varying the strength of the brace concrete. The reason for selecting these elements is that the analyses show that the failure and collapse starts within these elements first.

4.1 Deformation of the BRB Frame

For BRBF under loading, it is expected that encasing concrete strength should not have an effect on the formation of the plastic hinge on the frame columns (especially the left column where the load has been applied). The main purpose of the concrete is to prevent buckling of the core brace until they fail due to compression and not to influence the performance of other components. The deformation of the BRB frame at the time of collapse is shown in Fig. 5, where the concrete strength is kept at 31.2 MPa (4,500 psi).

The general trend of the total deformation of the BRBF tends to decrease as the concrete strength increases. The total deformation was 16.29 inches for 31.2 MPa concrete and 12.35 inches for 140 MPa concrete. As the concrete strength is increased, the BRBs were able to absorb more energy and so the deformation in members of the frame was decreased.

A reason for the large values in the total deformation results is because we traced the displacements of the eroded nodes. In the analyses, when the eroded nodes refrain the mass and momentum of the free node, and it is possible for the node to be involved in subsequent impact events. The eroded nodes were still recorded as they were still deforming away from the original undeformed model. During the analyses, the deformation results recorded in the left column first record values under plastic deformation. Once the strain in the column reaches the plastic limit of the material, the values shift from recording the nodes of the intact column to recording the values of the eroded nodes of the ruptured column. Though the results include the data of the eroded nodes, the typical trends found in the behavior of the results remain true which is: deformation decreases with the increase in concrete strength. The displacement of eroded nodes described can be seen in Fig. 5 where the rupture occurred at the bottom of the left column and the left brace.

4.2 Behavior of the Left Column

The left column underwent plastic deformation almost immediately. Fig. 5 shows the deformation of the left column. Since the load is applied directly on the web of the column, the column experiences the most energy, thus plastic behavior is instant. At first, the column began to bend about its strong axis as the load is applied in-plane of the frame and the deformation is concentrated near the center of the column. The deformation starts to the shift lower end and a plastic hinge is formed near the top end of the gusset plate and column connection near the support. Progressively, after the plastic hinges are formed, rupture in the column occurs (Fig. 6).

Two plastic hinges form on the lower end of the column (Fig. 6). The first one occurs at the support of the column near the gusset plate and column connection. This is the region where rupture occurs first. The other plastic hinge forms at the top end of the gusset plate and column connection. Fig. 6 shows the locations and formation of the plastic hinges. The failure in the column first starts at the bottom plastic hinge because the ridge fibers between web and flange begin to pull each other as the web follows the bending deformation of the column. The bending action causes a rotation

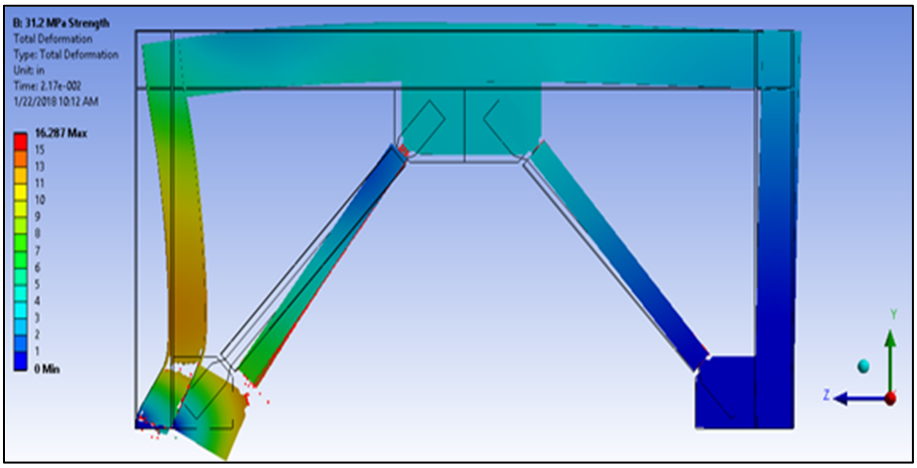

Fig. 5 The typical total deformation of the model.

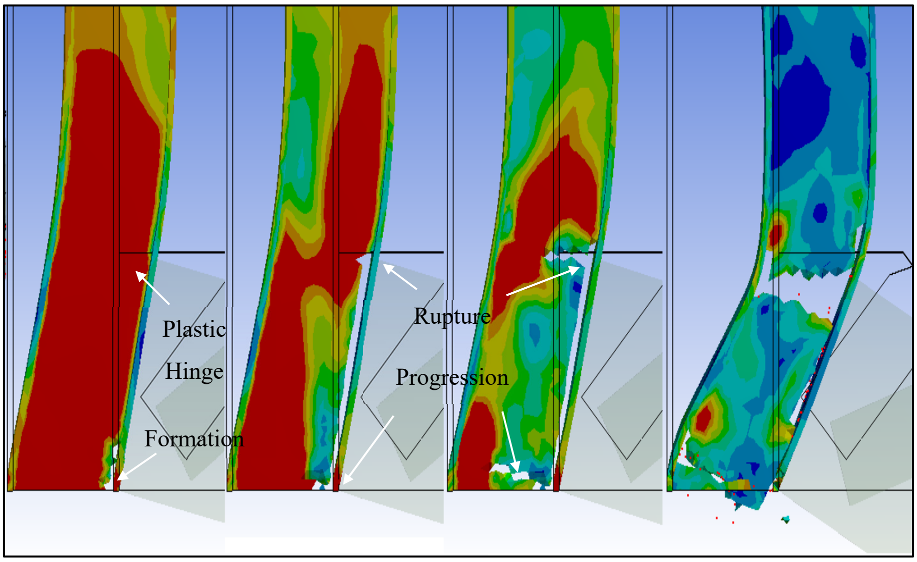

Fig. 6 Progression of rupture in the left column due to formation of the plastic hinges.

about the support and so the gusset plate follows this behavior. The column flange is held relatively straight as it is attached to the gusset plate which resists this rotation as it is attached to the braces. Thus, the fibers on the web and flange ridge are pulled. As time elapses, the rupture propagates along the edge between the web and flange.

Results show that when the strength of the brace concrete is kept at 31.2 MPa, 40 MPa and 48 MPa, the rupture first occur in the left column. But when the concrete strengths are increased to 69 MPa and 140 MPa, the steel core braces rupture first.

This happens as the concrete strength improves the performance of the brace and so the brace is able to absorb more energy within the same time.

Following the trend where the left column ruptures first, the left brace is able to resist the rotation caused by the lateral deformation of the left column. This increase in resistance stiffens the flange of the column which explains the propagation of the rupture going upwards until the top plastic hinge. This rupture occurs due to the fibers being pulled apart similar to that of the initial rupture in the left column. Once the rupture in the brace is large enough such that the resistance to the rotation is low, then the rupture in the column starts to shear into the web. The lower shear action into the web occurs as a result of the continuing rotation of the column near the support.

4.2.1 Lateral Deformation in Left Column

Fig. 7 shows the lateral deformation in the left column. Before the rupture in the column occurs, the lateral deformation of the left column is decreased as the concrete strength is increased. Comparing the time when rupture just occurs for the first time, the values of the lateral deformation in each case with varying concrete strength can be seen to be similar. There is a slight decrease as the concrete strength was increased but not that significant before the rupture. This is attributed to the concrete strength increase, allowing the braces to better dissipate the energy. However, once rupture occurred, the deformation for each concrete strength differs. The pattern of the deformation decreases with the increase in concrete strength is still true though. Once rupture occurs, the increase in concrete strength improves the resistance to this lateral deformation. This can be seen in the results near the end (Fig. 7). The lower strength of the concrete, encasing the braces, causes higher deformation in a short duration at the left column as opposed to the higher strength concrete produces less deformation in the same time period. This indicates that the strength of the encasing concrete has significant influence on the deformation of left column as well as the collapse of the frame.

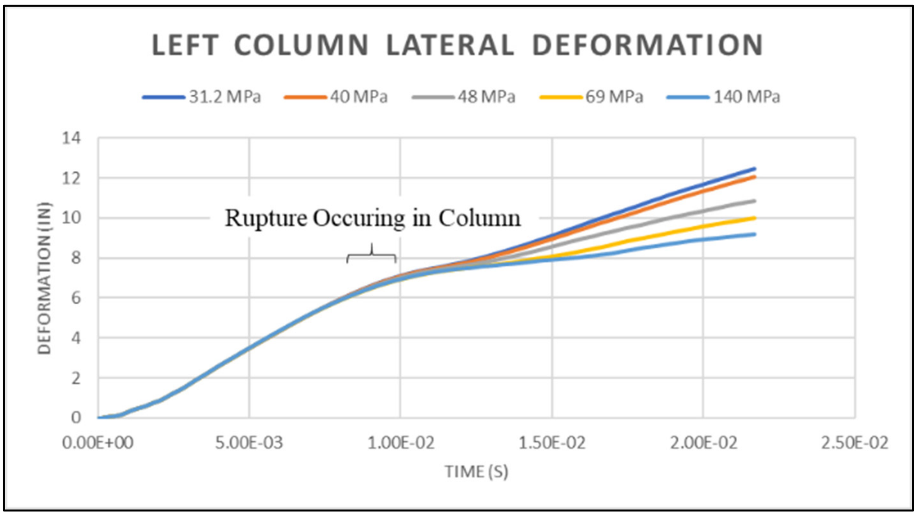

Fig. 7 The lateral deformation in the left column over time.

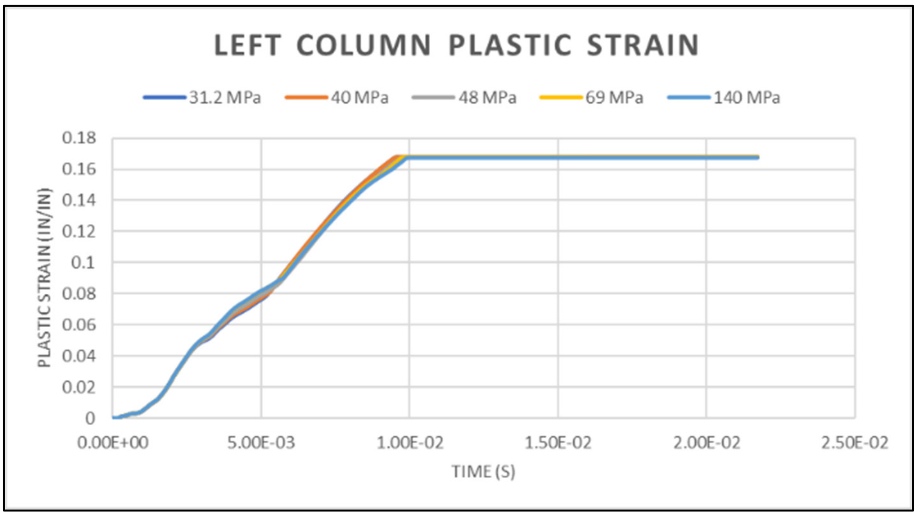

Fig. 8 Plastic strain developed in the left column over time.

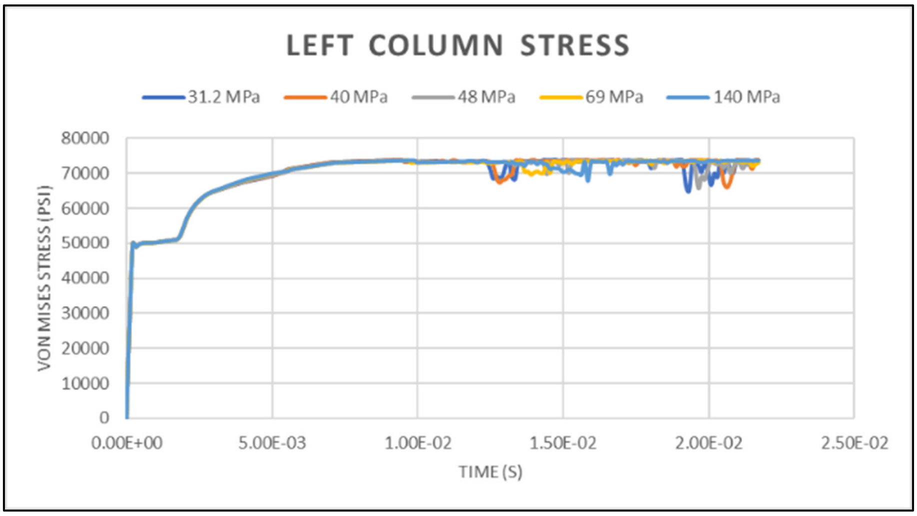

Fig. 9 Stress developed in the left column over time.

4.2.2 Plastic Strain and Stress in Left Column

The results of the plastic strain and equivalent stress developed in the column are shown in Figs. 8 and 9 respectively. The magnitude of the load causes the left column to enter plastic strain within 2.17×10^{-4} s of the analyses. The plastic strain increases until reaching a constant value. The maximum stress developed in the model reached the maximum value defined in the material properties. The corresponding strain value to the stress value is the constant strain value which was also the plastic strain limit in the analyses and so rupture occurs.

4.3 Left Brace

During the early stages of the analyses the left brace experiences tension caused by the lateral resistance to load. The right brace remains in compression. The left BRB fails first which in turn would affect the stability of the frame. The right BRB fares better but still entered plastic deformation as well as damage occurring in the concrete encasing. These two results tied with the damage in the left column lead to total failure of the lateral resisting structure.

Fig. 10 shows the deformation of the left brace and the concrete encasing with elapsed time. The steel casing is not shown for emphasizing the damage occurring on the concrete. The image progresses in time from left to right and top to bottom. The image shows the damage of the progressive collapse of the left brace from the beginning of the blast load till the rupture of the brace. The left column starts to bend which causes the attached bottom gusset plate to rotate clockwise. This rotation is resisted by the attached left brace on the gusset plate. As a result, the brace starts to bend about its strong axis. The load is large and the bending is excessive which causes a plastic hinge to form in the transition zone of the brace. Because the brace is attached to the top gusset plate as well, both braces experience similar deformation to that of the attached beam. The top gusset plate translates deformation of the connected beam onto the braces. Because the left end of the gusset plate deforms into the negative x-coordinate direction and the right end deforms into the opposite direction, this deformation is also experienced by the braces.

This is what causes the braces to bend about their weak axes on the top side. These two deformations cause bending which becomes the primary mode of failure for the brace. This weak axis bending causes the formation of a plastic hinge at the transition zone of the top end of the brace.

The typical brace deformation before rupture can be seen in Fig. 11. The plastic strain and stress developed in the left brace for different concrete strengths can be seen in Figs. 12 and 13 respectively. The general trend of the results show that the brace was able to absorb more energy within the same amount of time when

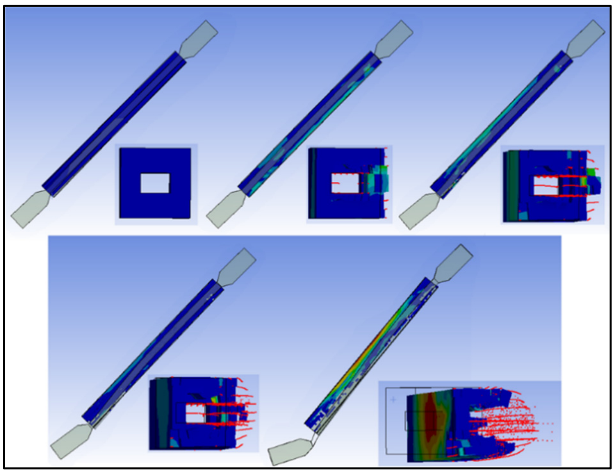

Fig. 10 The progressive damage on the left BRB.

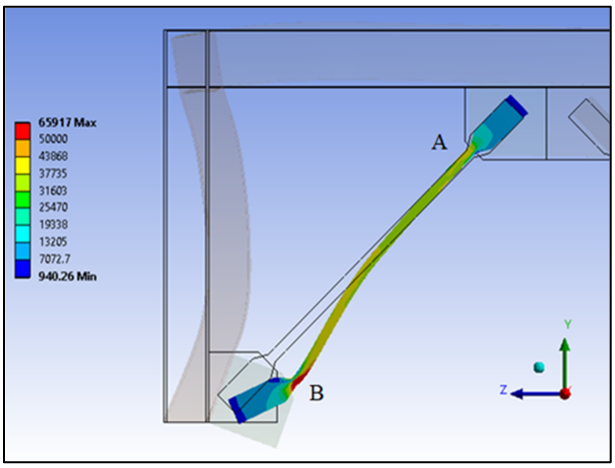

Fig. 11 Behavior of the left brace right before rupture.

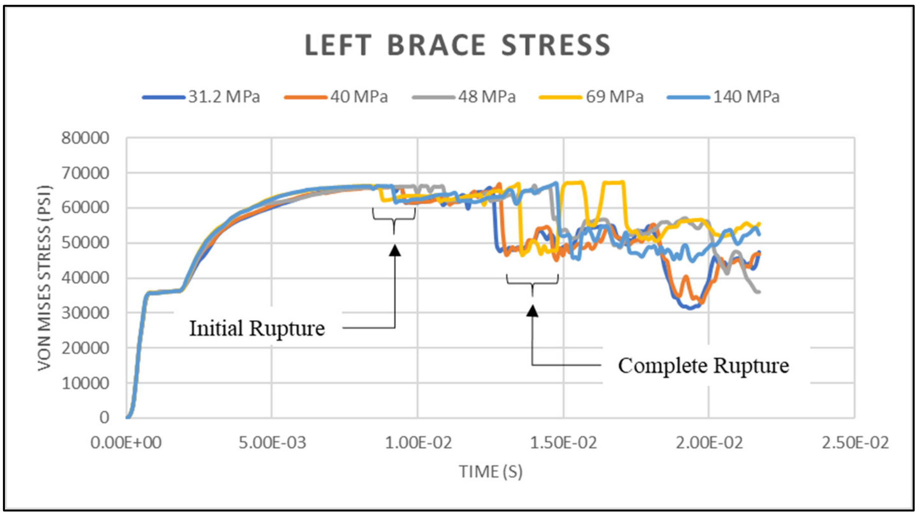

Fig. 12 The plastic strain developed in the left brace.

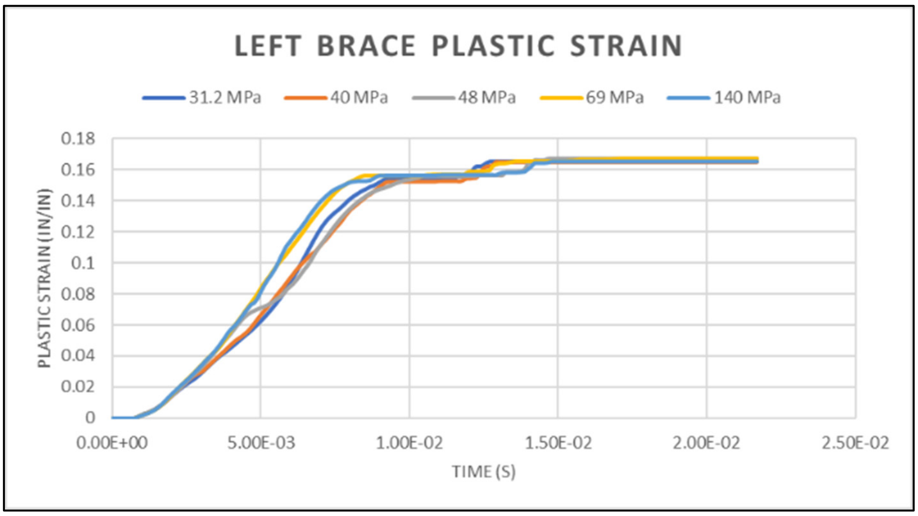

Fig. 13 The stress developed in the left brace over time.

compared to the other runs. The stress results show a disturbance in the stress as rupture is initiated in the transition zone as can be identified by the dips in the curves.

Both the plastic strain and stress results have data past rupture. This was due to the momentum concept. After rupture, the momentum bends the brace similar to before when the rupture occurred. After rupture, the analyses results show that the brace was able to take between 70%-75% of its capacity.

5. Conclusion

The research was aimed to understand the behavior of the BRBF under the blast loading and to identify the effects of varying strengths of the encased concrete on the potential failure of the buckling restrained braces as well as the entire frame. The results show that as the concrete strength is increased the overall performance of the frame is improved. The load magnitude was still too large for the frame and so the failure in the frame occurred that would lead to potential collapse. As concrete strength was increased, the overall deformation in frame decreased. The concrete encasing in the BRB also failed due to bending of the steel brace core at the same time as the concrete crushes. The duration and magnitude of the blast impulse load causes momentum to bend the left brace which in turn crushes the corresponding concrete encasing.

The objective of this research is to identify the element and location where the failure occurs in a BRBF under blast load. In conclusion, the performance of the frame can be improved by increasing the strength of the concrete surrounding the core brace. In that case, high strength concrete would be desirable to cover the core braces. When a conventional building is designed and constructed with BRBF to resist seismic loading, the building frames can activate its dampers while the blast load is so intense in very short period of time, and the structure fails before the dampers could even be activated. Blast loads are more intense than seismic loads. So, a building frame designed to survive high intensity seismic load may not be sufficient to withstand a significant blast load. This research gives an idea on how to proceed designing a BRBF structure to resist blast load even if they are primarily for seismic loading.

Acknowledgments

The authors are acknowledging the contributions and support from: (a) Department of Civil Engineering & Construction Management, California State University, Northridge, and (b) Star Seismic Inc.

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