Performance assessment of gusset plate connections and their effects on the system level performance of steel Buckling-Restrained Braced Frames

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ABSTRACT

Previous studies indicate that the system-level response of special concentrically braced frames and buckling restrained braced frames can be critically affected by the performance of gusset plate connections. In this paper, relevant past studies on the performance of gusset plate connections are briefly reviewed. Gusset plate response as limit state in buckling-restrained braced frames is discussed, and potential failure modes are presented. The response of several gusset plate connection details, from previous experimental studies are verified through finite element analyses. An improved numerical model for the system-level seismic performance assessment of buckling-restrained braced frames is proposed. The proposed numerical model aims to capture the effects of gusset plate performance on selected seismic response attributes of buckling-restrained braced frames, including the global stiffness of the structure and the system post-yield response. The numerical model can be used in continuous or stub beam-column-gusset connection designs. The proposed model is adopted in the seismic performance assessment of an example five-story buckling-restrained braced frame structure and response-history analyses are carried out under a suite of 16 ground motions, at both design basis and maximum credible earthquakes. The accuracy improvements resulting from the proposed numerical model, compared to the widely-used simplified models, are discussed.

Keywords: *Steel Structures, Gusset Plate Connections, BRBF, Seismic Performance, Nonlinear Modelling*

INTRODUCTION

Background

Buckling restrained braced frames (BRBFs) and special concentrically braced frames (SCBFs) are effective lateral load resisting systems (LLRS) in steel buildings with desirable performance attributes such as a ductile post-yield response and high elastic stiffness. In addition, BRBFs have the advantage where the brace elements yield in compression as well as tension. This effectively reduces the brace susceptibility to early fracture caused by low-cycle fatigue and results in a more ductile structural performance. These have led to mass implementation of SCBFs and BRBFs in steel structures within the last decades.

Early studies on BRBFs and SCBFs were focused on component level testing of buckling restrained braces (BRB) and special concentrically braces (SCBs) as structural elements [1-4]. Such studies established BRBs and SCBs as structural components with attractive characteristics such as high ductility and high stiffness. However, as studies shifted towards a system level seismic performance assessment of BRBFs and SCBFs [5-9], special attention was given to the performance of the Beam-Brace-Column (BBC) gusset connections. One of the conclusions drawn from these studies was that the detailing of the BBC gusset connection critically affects the overall seismic performance of BRBFs and SCBFs, specifically at high drift ratios associated with the maximum considered earthquake (MCE). These studies showed that the earlier component level tests did not accurately capture the performance of BRBFs and SCBFs at a system level, due to the complex interactions between the brace and the gusset plate connection during inelastic deformations. Such findings inspired numerous experimental and analytical studies specifically devoted to studying the performance of BBC gusset plate connections. Specifically, their effect on the overall seismic performance of the system was studied, and new design recommendations for gusset plate connections were provided [10-12]. Many studies have noted that commonly adopted methods for the design of gusset plate connections such as Whitmore [13] and Thornton [14] often result in connection details that could limit the response of BRBFs and SCBFs by undesired connection failures [15-17]. Additional studies on the performance of steel concentrically braced frames include improved numerical models for the performance assessment of such systems, to capture the effects of connection detailing on the response of the system. For instance, Hsaio et al. [18-19] proposed improved numerical models, using line elements, for the performance assessment of SCBFs, which included the effects of the connection as well.

Objectives

The study reported in this paper was carried out as part of a large experimental program at the University of Toronto using the *UT10 Hybrid Simulation Platform* (UT10) [20,21]. The UT10 has been developed for multi-element hybrid simulations on uniaxial rate-independent elements, using the UT-SIM Frame work [22-24]. One of the projects, which used the UT10, was a performance assessment of a five-story BRBF through multi-element hybrid simulations. In this project, the response of the BRB elements were physically evaluated using the adjustable yielding brace (AYB) [20,21], while the rest of the structure was modelled numerically. The objective of the present study is to ensure that the effects of connection detailing, on the system-level response of the structure, is captured in the hybrid simulation. Therefore, the study aims to propose an improved numerical model to be used in the performance assessment of BRBFs with concentrically diagonal BRBs, which captures the effects of connection detailing. The study provides a summary of a numerical study on the response of gusset plate connections in BRBFs, and its effect on the global performance assessments of such systems. Benchmark numerical models are developed to match the response of previously tested gusset plate subassemblies. Detailed designs are carried out for a five-story BRBF reference structure, as per ASCE 7-10 [25] and AISC 2010 [26, 27]. Next, FE models of the BBC joint subassemblies of the example structure are developed and a component level response assessment is carried out. An improved numerical model is then proposed to consider the effects of the gusset connections in system level performance assessment of BRBFs.

BENCHMARK FINITE ELEMENT MODELS

Gusset Plate Assembly Database

The BBC subassemblies used for validation of benchmark models are those tested and reported by Yam and Cheng [16]. The layouts of the test setups are shown in Figure 1. In one of the setups (left), the strong beam could move out of plane using rollers. In the other (right), tension rods were used to induce bending moment in the frame members. Additional information about the experiments is provided by Yam and Cheng [16]. Columns 1 to 5 of Table 1 summarize the properties and results of selected experiments that are used for validation of benchmark numerical models in the present study.

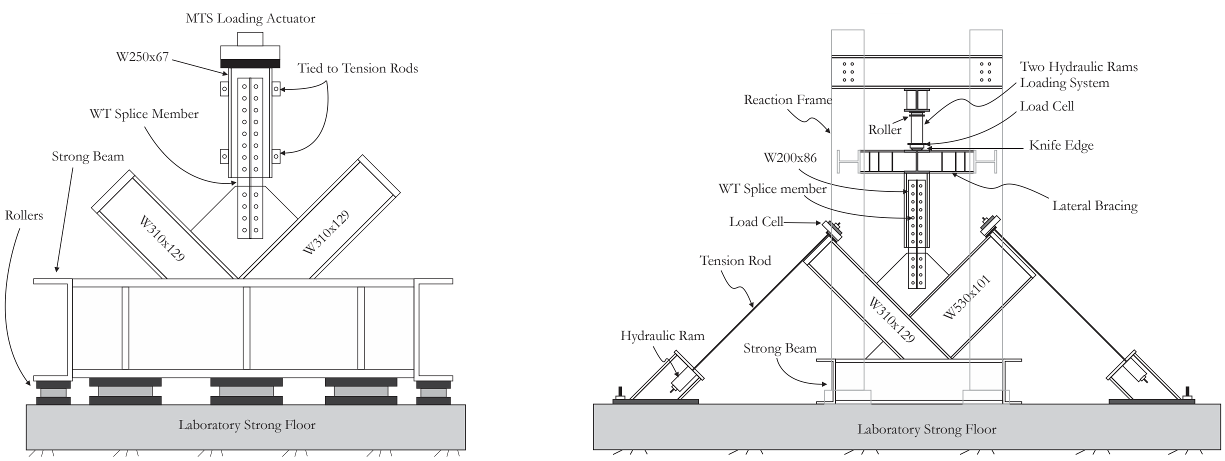


Figure 1. Experimental Setups for Tests Reported by Yam and Cheng [16]

Table 1. Summary of the Experimental Results by Yam and Cheng [16] Used in the Present Study

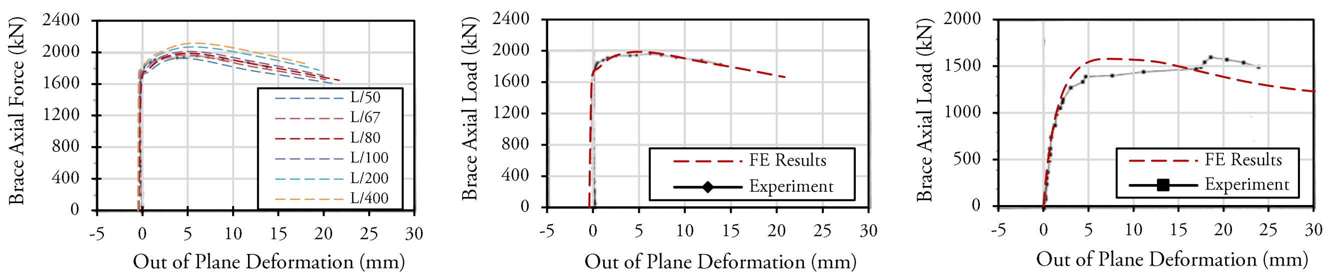
Specimen	Thornton Length (mm)	Gusset Thickness (mm)	Gusset Size (mm)	$F_{Exp.}$ (kN)	$F_{FE\ Model}$ (kN)	Error (%)	Imperfection
GP1	200	13.3	500 x 400	1956	1987	1.6	$L_{Th}/80$
GP2	200	9.8	500 x 400	1356	1374	1.3	$L_{Th}/67$
GP3	200	6.5	500 x 400	742	693	6.6	$L_{Th}/100$
SP1	470	13.3	850 x 700	1606	1579	1.7	$L_{Th}/100$
SP2	470	9.8	850 x 700	1010	1003	0.7	$L_{Th}/200$
MP1	200	13.3	500 x 400	1933	1929	0.2	$L_{Th}/80$
MP2	200	9.8	500 x 400	1316	1363	3.6	$L_{Th}/67$
MP3	200	6.5	500 x 400	721	740	2.6	$L_{Th}/100$

Modelling Methodology

The methodology for FE nonlinear buckling analysis of gusset plate subassemblies is similar to that done by Uriz et al. [28], where it was shown that the nonlinear buckling response of braces could be imitated by an initial imperfection in the form of the first buckling mode in the member. This method was later adopted for gusset plate connections by Fang et al. [29], where the authors applied the initial imperfection, as the first buckling mode of the gusset plate subassembly, as a percentage of gusset plate thickness. In the present study, the same approach is adopted, where the first mode of buckling is chosen as the initial imperfection. However, a different approach is adopted to estimate the extent of initial imperfection. As the compressive capacity of gusset connections are indirectly proportional to their respective Thornton [14] length, a fraction of the Thornton length is used as the amount of imperfection (i.e. $L_{Th}/200$, where L_{Th} is the gusset connection respective Thornton length). 4-node shell elements with reduced integration (S4R) were used for all members. Then, a linear buckling analysis is performed on the specimen in ABAQUS. Next, a nonlinear buckling analysis is carried out in ABAQUS by introducing an initial deformation in the model, in the form of the first buckling mode. The deformed shape is normalized to the desired imperfection values. Several imperfection values are attempted for each specimen. Figure 2 (a) shows the results obtained from different nonlinear analyses with different initial imperfection amplitudes for specimen GP1, where the values of imperfection are identified as a fraction of the Thornton length corresponding to the system. The best match is provided by an imperfection equal to $L_{Th}/80$. Figure 2 (b) shows the comparison only for the case with an imperfection of $L_{Th}/80$.

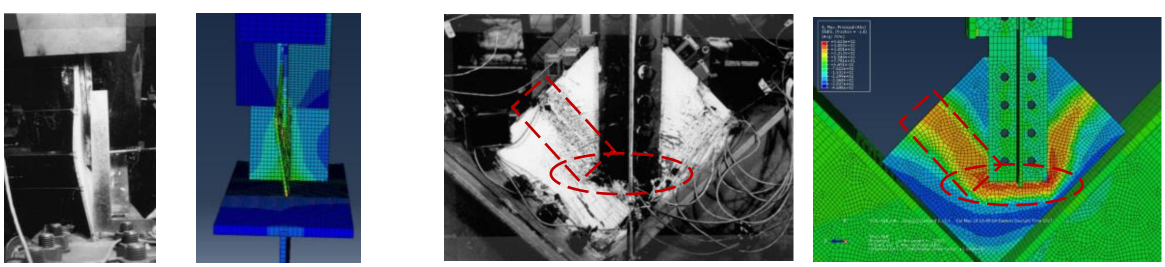
Results

In a similar manner, nonlinear buckling analyses are carried out on specimens listed in Table 1. The results of the FE models are presented and compared with their corresponding experimental results for specimens GP1 and SP1, in Figures 2 (b) and (c), respectively. The summary of the results is also presented in the last four columns of Table 1 in terms of maximum loads observed in the experiment, the maximum load obtained from the FE model, achieved accuracy, and the initial imperfection that best matched the experimental results. The initial imperfections are measured as fractions of the Thornton length for the gusset plate. The initial imperfections that lead to the best prediction of the maximum load, in the FE model, are also provided.



(a) *(b)* *(c)*

Figure 2. Finite Element Results vs. Experimental Results: (a) Specimen GP1 Imperfection Analysis, (b) Specimen GP1, (c) Specimen SP1



(a) *(b)*

Figure 3. Experimental Results vs. FE Models (a) Specimen MP3A, and (b) Specimen MP1 (experiments from Yam and Cheng [16])

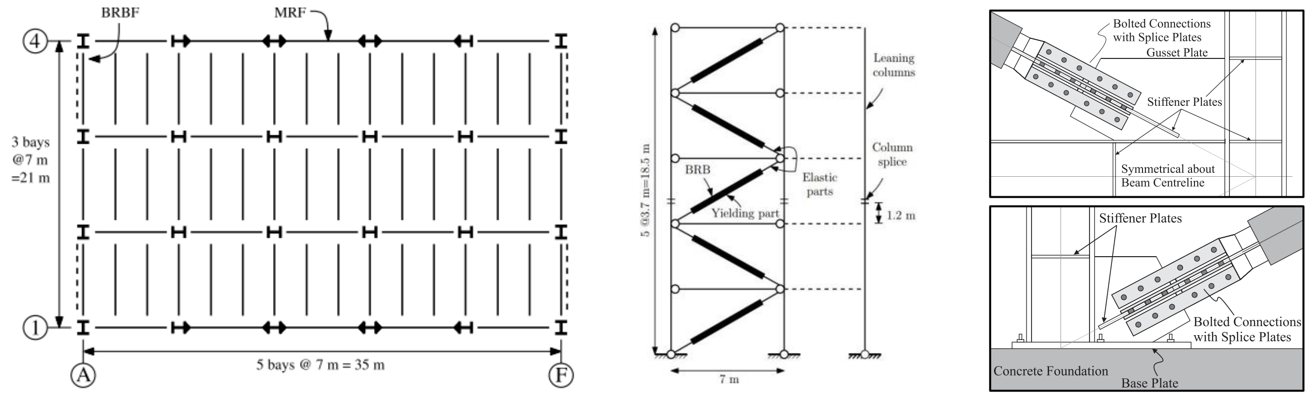
Shown in Figure 3 is a visual comparison between typical results from the FE models and typical experimental results [16]. Figure 3 (a) shows a comparison of the deformed shapes for specimen MP3A. Figure 3 (b) shows the front view of specimen MP1 during the experiment and after the FE analysis. The stress distribution in the FE results shows good agreement with the yield lines observed in the experiment.

NONLINEAR BUCKLING ANALYSIS OF GUSSET PLATE SUBASSEMBLIES

Reference Structure

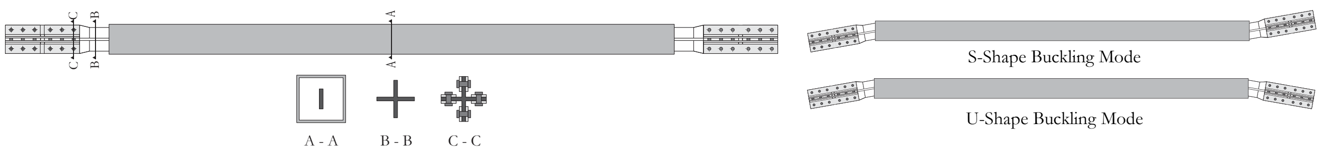
The reference structure is a 5-story steel structure located in Los Angeles, California. The LLRS of the structure is formed by BRBFs, and moment resisting frames (MRFs), in two orthogonal directions. The structure is designed as per ASCE 7-10, AISC 360-10, and AISC 341-10 [25-27]. The building plan and one of the BRBFs are shown in Figures 4 (a) and (b), respectively. The gusset plate connections are designed to be capacity protected using the Whitmore and Thornton methods. The shear tabs are designed according to the AISC 360-10 [26] provisions, using double angles, welded to the columns and bolted to the beams. The slabs are designed using the CANAM design catalogues [30]. The gusset plate geometry is affected not only by the design loads, but also by the geometry and layout of the surrounding structural components. On this basis, the BRBF shown in

Figure 4 has 4 types of gusset plates as follows: (Type 1) first floor, (Type 2) intermediate floors below the splice, (Type 3) intermediate floors above the splice, and (Type 4) top floor gusset plate. Typical details used in the design of Type 1 and Type 2 gusset plates are shown in Figure 4 (c). Additional details on the design of the reference structure can be found in [21].



(a) *(b)* *(c)*

Figure 4. Reference Structure, (a) Plan, (b) BRBF, and (c) Gusset Plate Connection Details



(a) *(b)*

Figure 5. (a) Typical BRB Detail, and (b) BRB Buckling Failure Modes

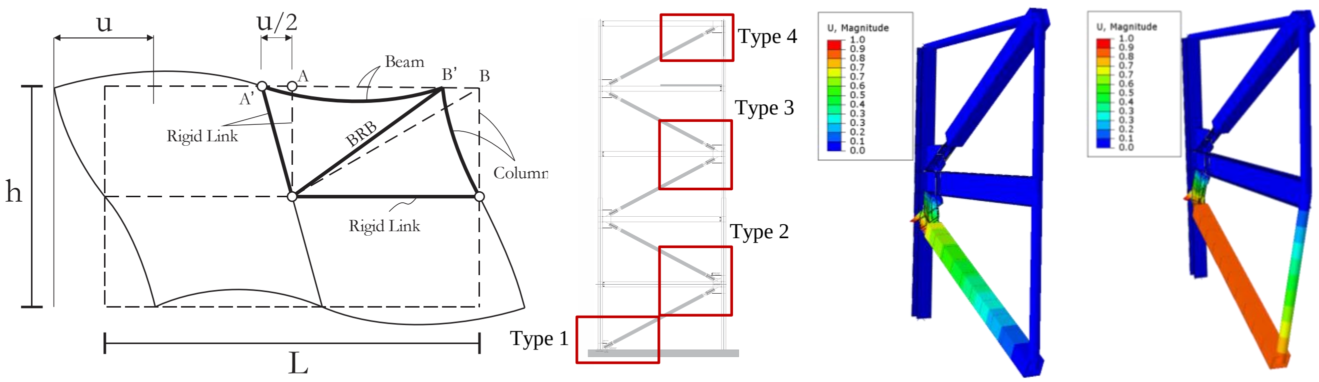
Modelling Methodology

Figure 5 (a) shows the typical detail of the BRBs used in the design of the reference structure. Figure 5 (b) illustrates the common buckling modes of BRBs. In order to capture the effect of beam and column on the compressive response of the gusset plate connections, the brace beam column joint subassembly must be modelled with inclusion of the beam and the column. The approach that is used in the present study for modelling the BBC joint subassemblies of the reference structure is inspired by that provided by Kaneko et al. [31]. In such approach, half the beam and the column, adjacent to the joint, are included in the FE model such that the FE model is representative of a quarter of the frame. Afterwards, far ends of the beam and column, included in the FE model, are connected to the center-point of the frame using truss elements. This is a reasonable assumption as the mid-points represent the inflection points. This approach is schematically shown in Figure 6 (a). Following this approach and considering the example BRBF, four unique FE models must be developed to capture the response of all gusset plate joint types. The unique joint types are shown in Figure 6 (b). The difference between types 2 and 3 is due to the change in column section. In order to have the subassembly respond in U-Shape and S-Shape buckling modes, as indicated in Figures 6 (c) and (d), appropriate boundary conditions must be considered. The boundary conditions are briefly discussed for Types 2 and 3 joint subassemblies. For both modes of buckling, the beam mid-point is free to move in the frame plane and the mid-point of the bottom column is assigned a pin support. For the U-Shape buckling mode, the mid-point of the bottom brace is assigned a pin support in the plane, while allowing movement out of the frame plane. In addition, the encasing of the BRB is assigned a rotational constraint to stay parallel with the frame plane. For the S-Shape buckling mode, the mid-point of the bottom brace is assigned a pin condition, while not allowing movement out of the frame plane. The rest of the procedure in the FE analyses are similar to that, described for the benchmark models adapted to the design of the gusset connections. The beam, column, the gusset plate, and the BRB cruciform section that extends out of the encasing, are modelled using 4-node shell elements with reduced integration (S4R). A truss element was used to model the BRB core and a general beam section, with negligible axial rigidity (EA) was used to model the encasing. The moment of inertia of the beam element, representing the encasing, was determined based on the design and the detail shown in Figure 5 (a). The purpose of these analyses is to determine the compressive capacity of the gusset plate connection. Therefore, a linear response is assumed for the BRB element while a nonlinear behavior was assumed for the other members. This is done to ensure that the loads are not capped by BRB yielding. As previously shown, the amount of initial imperfection that is applied to the model does not have a significant effect on the overall buckling response. Therefore, in nonlinear FE analyses, an initial imperfection in the form of the first buckling mode, normalized to the maximum values observed in benchmark models (last column of Table 1), is used.

Results

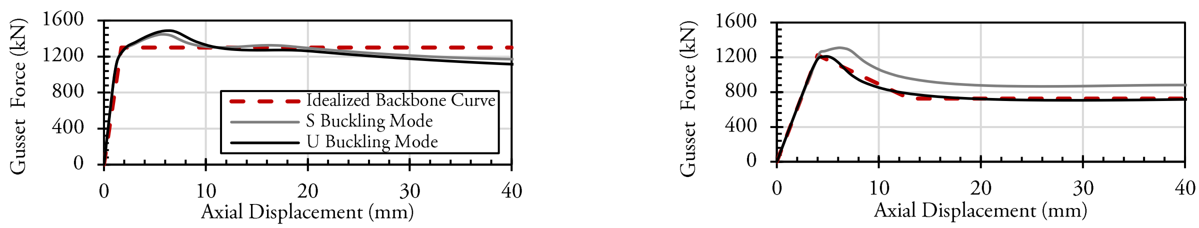
Analyses are performed on each of the four unique gusset BBC subassemblies, as shown in Figure 6 (b). Displacements were applied to the mid-points of the floor beam and their magnitudes were increased incrementally. Each case was analyzed twice;

once with boundary conditions producing the U-Shape buckling mode, and once with boundary conditions producing the S-Shape buckling mode. Force-deformation responses are obtained for each case. Figures 7 (a) and (b) show the force-deformation plots for Type 1 and Type 2 joint subassemblies, respectively. The force-deformation plots for Types 3 and 4 subassemblies resemble that of Type 2 with slightly smaller force values. As can be observed, the post-peak response of Type 1 joint subassembly is much more stable as the bottom of the gusset plate is connected to the base-plate and foundation, which further restrains the gusset plate. The negative post-peak stiffness of Types 2, 3 and 4 subassemblies, is amplified by the lateral torsional buckling of the floor beam. Idealized backbone curves are also shown in Figure 7.



(a) *(b)* *(c)* *(d)*

Figure 6. (a) Schematic Illustration of the Modelled Portion using the approach by Kaneko et al. [31], (b) BRBF Illustrating the Gusset Types, (c) S-Shape Buckling Deformed Shape for Type II, (d) U-Shape Buckling Deformed Shape for Type II



(a) *(b)*

Figure 7. Force-Deformation Response of Gusset BBC Subassemblies; (a) Type 1, and (b) Type 2

The maximum load observed in the BRB physical substructures, used in the performance assessment of the five-story building using the UT10, was observed to be 800 kN. Therefore, as all gusset plate connection joint subassemblies have a force-capacity higher than 800 kN, it can be concluded that gusset plate buckling is not a limit state, from a forced-based perspective. As the BRBs reach their yielding force and as the structure deforms into the nonlinear range, the increased drifts could affect the gusset plate capacity. This can be addressed by considering the two possible scenarios. In the case where the brace and the gusset plate are in compression, as shown in Figure 8, the frame members will rotate such that the angle between the beam and column will approach an obtuse angle ($\alpha > 90^o$). This action will put the gusset plate in tension, and therefore, will stiffen the gusset plate. As a result, the buckling capacity of the gusset plate will not decrease. Therefore, gusset plate buckling, in compression, or yielding, in tension, are not limit states in the response of the structure. However, the effect of the response of the connections on the global response of the building structure must be captured in the system level response assessment.

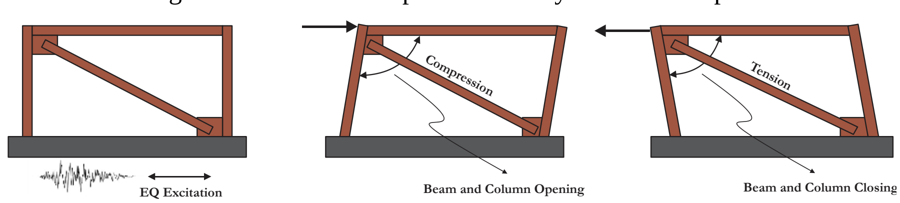


Figure 8: Different Cases for Lateral Frame Movement; Undeformed Shape, Brace in Compression, and Brace in Tension

PROPOSED NUMERICAL MODEL

Depending on the type of analysis, the structure under consideration, and the response parameter of interest, a more accurate model may be beneficial. For instance, the building additional post yield stiffness that is achieved through the connection action can affect the residual drifts of the structure. In this section, an improved frame model is proposed for BRBFs, which considers the effects of connection detailing, on the response of the system. The model is primarily intended to account for the increase in the lateral stiffness and the additional indeterminacies in the structure, caused by gusset plate connections. In addition, it can be used to capture the failure of the gusset plates as a limit state, using the backbone curves given in Figure 7. Even if gusset plate failure is not a limit state, the improved numerical model could lead to increased precision.

The most common approach for modelling BRBFs is to use truss elements for modelling beams and braces and continuous elements for columns. In order to account for the added stiffness caused by the connections in the structure Choi et al. [32] suggested modelling the whole BRB with the same element, representing the core and increasing the stiffness of the brace by 1.45, in the elastic range. This approach is referred to as 'Model 0' in the present study. As the first improvement to Model 0, the stiffness of the elastic portions of the BRB element can be adjusted such that the overall stiffness of the BRB would be the same as that in Model 0. In other words, the same effective stiffness is used to account for the added stiffness caused by the connection. However, the increased stiffness is captured locally. In the present study, this model is referred to as 'Model 1'. The 1.45 factor that has been proposed by Choi et al. [32] is an approximation for capturing the added lateral stiffness caused by BRB connections. An alternative approach to this is to develop an FE model of the elastic portion of the BRB and the gusset plate and measure the axial stiffness of this portion physically. Such model would be similar to Model 1. However, instead of the 1.45 factor, the real stiffness of the gusset plate and elastic portion of BRB assembly is used in the model. Such approach requires more resources and development of FE models. This model is included in the present study to assess the validity of the 1.45 stiffness factor for the reference structure. In the present study, this model is referred to as 'Model 2'.

The limit states that are captured in the proposed model include: (1) Yielding of BRB in tension and compression, (2) Plastic hinge formation in the beam and column and their fracture under low-cycle fatigue, (3) Buckling of the gusset plate connection, and (4) Fracture of the cruciform section of the BRB, which extends out of the encasing, under flexural demands on the BRB. The proposed numerical model is illustrated in Figure 9 (a) and (b), depending on the design. The model adapted for the continuous beam detail shown in Figure 9 (a) is referred to as 'Model 3' and is discussed below in detail.

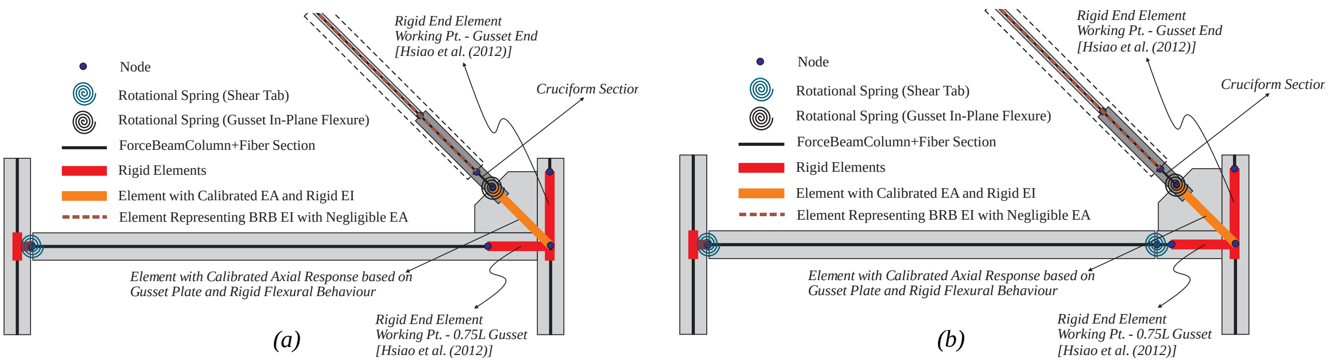


Figure 9: Improved Numerical Model Proposed for (a) Continuous Beam Detail, and (b) Stub Beam Detail

In Model 3, the moment rotation properties of shear tab connections are determined based on the study by Liu and Astaneh-Asl [33]. After determining the backbone curve for the moment-rotation response of the shear-tab connections, a hysteretic response such as the Hysteretic Material Pinch 4 Material in OpenSees [34] can be used for the cyclic response. All frame elements are modelled using fiber sections. The cruciform section that extends beyond the BRB encasing is modelled using fiber sections. Fracture under low-cycle fatigue is captured for the cruciform fiber section by using OpenSees Fatigue Material, which works based on the Coffin-Manson linear fracture model. Uriz and Mahin [9] describe the implementation of the Fatigue Material in OpenSees. Rigid elements are used to represent the portion of the frame members that are adjacent to the gusset plate. The lengths of the rigid elements are based on the study by Hsiao et al. [18, 19]. The core of the brace member is physically modelled at its corresponding location, using fiber sections. The elastic portion of the BRB that is within the BRB encasing is also modelled using fiber sections. The BRB encasing is defined using an element with its flexural rigidity, *EI*, and negligible axial rigidity, *EA*, to account for the presence of deboning material between the core and concrete fill in the BRB. Axial response of the gusset plate is captured by a flexurally rigid and axially calibrated frame element, to have the same axial stiffness as the gusset plate connections in the structure. As a more general model and when gusset buckling is expected, the idealized backbone curves shown in Figure 7 can be used to capture the response of the gusset plates in compression. At the point where the gusset plate meets the cruciform extension of the BRB, a localized rotational spring is defined, which is representative of the in-plane elastic rotational response of the gusset plate. The stiffness and ultimate capacities of gusset plates in tension, compression, and flexure, are determined from the FE models that were used to assess the response of the connection at a component level.

TIME HISTORY ANALYSES

The reference structure is modeled with Models 0, 1, 2, and 3. The models are subjected to a suite of 16 near-fault ground motions. The ground motion selection and scaling were carried out as per the recommendations by the ASCE 7-16 [35]. Five ground motions are 'no-pulse' motion, while the rest are pulse motions. The ground motions are selected to be representative of the seismological characteristics of the region and scaled, once to match the MCE Uniform Hazard Spectrum (UHS), and once to match the Design Basis Earthquake (DBE) UHS. The results of the nonlinear time-history analyses are discussed, and selected parameters under MCE shaking are presented in the following section. Rayleigh damping of 3% is assumed for the first and the second modes of the system and direct time-step integration of the equations of motion is carried out. Additional details on ground motion selection and the time-history analyses can be found in [21].

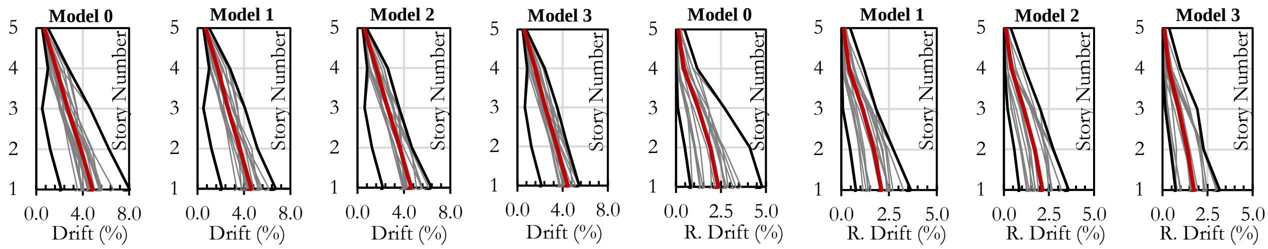


Figure 10: MCE-Level Response for all Models; Transient Drifts and Residual Drifts

Under the DBE level shaking, the response parameters from different models are not appreciably different from one another. This is because the structure has not reached lateral drifts at which the effects from connection modelling, start to affect the response significantly. The results of the analyses under the MCE level records are shown in Figure 10. In each figure, the red line shows the mean response, the black lines show the minimum and maximum values, and the grey lines show the response obtained from each individual record. When studying the transient drifts, it can be observed that more simplified models lead to very large drift ratios, under some records. Under one record, Model 0, predicts a drift ratio of 8%, which is past the structure's collapse point. In performance-based earthquake engineering, if a record leads to response parameters beyond the acceptable limits for the model to give reasonable results, the results under that ground motion should be excluded [35]. This is considered for results from Model 3, as the final model in this study. The results reported for Models 0-2 are only for comparison purposes, which requires the use of consistent ground motions. The accelerations were observed to be more or less the same for all models. This is expected, as the structure experiences the maximum accelerations in its elastic response, where the contribution of connection detailing to the overall stiffness of the system is not significant. Among the presented response parameters, the residual drifts are the most affected. Using Model 3, the maximum estimated residual drift drops from 2.35% (in Model 0) to 1.75%, which is a significant reduction. This is because of the higher effective post-yield stiffness in Model 3. The transient drifts from Model 3 are generally smaller than their corresponding values from other models. The base shear that is estimated using Model 3 is 855 kN, where Model 0 predicts a base shear of 743 kN. Model 1, which uses the 1.45 stiffness adjustment factor locally, predicts a base shear of 828 kN, which is reasonably close to the base shear obtained from Model 3, indicating the 1.45 stiffness factor to be a reasonable estimation from a force-based perspective.

CONCLUSIONS

The performance of gusset plate connections in concentrically braced frames is discussed. Previous studies on the component-level and system-level response of gusset plate connections are briefly reviewed. A numerical approach for the performance assessment of gusset plate connections is outlined. An improved numerical model is proposed to capture the effect of gusset plate detailing on the system-level response of BRBFs. Gusset plate buckling as a limit state, increased post-yield stiffness caused by additional indeterminacy in the system, and the flexural action caused by the presence of the gusset plate can be accounted for in the proposed model. The results of the response-history analyses indicate that under MCE shaking, accounting for the additional post-yield stiffness from gusset plate connections, leads to lower transient and residual drifts. Most importantly, as the model precision increases from Model 0 to Model 3, the dispersion in the results is reduced.

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