NUMERICAL ANALYSIS OF THE BEHAVIOR OF CONCENTRICALLY BRACED FRAMES UNDER CYCLIC LOADING CONSIDERING COLUMN ORIENTATION

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ABSTRACT

Concentrically braced frames (CBFs) are widely used as lateral-load resisting system in steel structures. They are economical, and their strength and stiffness may satisfy seismic demand of structures in seismic regions. During severe, infrequent earthquakes, brace yielding and buckling occurs, and this behavior provides the ductility and the energy dissipation capacities for structures. In recent years, a number of studies have examined different factors, which influence the performance of SCBFs; in particular, the gusset plate and the brace member details.
This study is carried out to investigate the effect of column orientation and the resulting flexibility of the connection region on the overall behavior and performance of the system. Nonlinear analyses using a detailed inelastic finite element model (FEM) and the concept of plastic equivalent strainare carried out to study the behavior of frames subjected to cyclic loading.The results showed that the flexibility of the connection region caused by column orientation affects the overall response of the system including lateral load and ductility capacity.

INTRODUCTION

Steel braced frames are commonly used lateral-load resisting systems in design codes. Special concentrically braced frames (SCBFs) are increasingly used in seismic regions in recent years due to their satisfactory behavior and uncertainty as to the performance of special moment resisting frames after the 1994 Northridge earthquake. In SCBFs, the braces are connected to the beams and columns by gusset plate connections, and energy dissipation is provided by the tensile yielding and post buckling deformation of the brace. There are some requirements in design codes for braces to ascertain the nonlinear behavior of them. Out of plane buckling of the braces along with the expected tensile and compressive capacities of them has to be accounted in designing the gusset plate. The current design practice recommends a “$2t_p$” linear geometric offset to be used, where $t_p$is the plate thickness, in order to provide free end rotation resulting from out-of-plane brace buckling. However, relatively large and uneconomical gusset plates resulting from linear geometric offset encouraged researchers to develop substitution for that. In response to this demand, the elliptical geometric offset has been proposed by Lehman et al. (2008) at the University of Washington.
Many studies in the past examined the performance of the brace and the gusset plate connection. Nevertheless the beam and column as main members of the frame play important role in overall performance of the system. The common practice of the columns is I-shaped sections which provides a favorable seismic

performance. But the behavior of the frame is different when the gusset plate is connected to the column flange in comparison with web connection of the gusset plate. The latter connection providesa flexibility at the region of the connection of the gusset plate to the column. In the current design concept, columns are expected to show limited non-linear deformations, while it seems that the local flexibility makes the column susceptible to a considerable local plastic deformation. To investigate the effects of column orientation on the behavior of the system, non-linear analyses were performed using inelastic finite element (FE) model.
In the FE model a series of full-scale single bay, single story frame was used to evaluate the interaction of members. The initial model was adapted from the researches in the University of Washington which the gusset plate was connected to the flange of the column, then the I-shape column was rotated in the frame so that the gusset was connected to the column web. This approach was carried out in several models with different gusset plate geometry such as rectangular gusset plate with elliptical or linear clearance distance and also tapered gusset plate. Finally, a stiffening approach was proposed and evaluated by the FE model to compensate the adverse effect of local flexibility and improve the seismic behavior of the system.

DESCRIBTION OF THE NUMERICAL MODEL AND VERIFICATION

The experimental research program at the University of Washington provided a basis for analytical research and design recommendations. The basic model selected for the current study was sub assemblage of a CBF system having a single bay and single story with a diagonal brace. The centerline measurements of the specimen were 3.66 m by 3.66 m (12 ft by 12 ft). The sub assemblage consisted of a brace (HSS 5x5x 3/8), two beams (typically W16x45), two columns (W12x72), and two gusset-plate connections as illustrated in Figure 1. The shear tab connection was used for beam-to-column connection. For all connections of the frame, members were tied together. As it was mentioned before, in this study three types of gusset plate were also used to evaluate the interaction of column orientation with common types of gusset plate. The models were subjected to a cyclic inelastic deformation history based upon ATC-24 testing protocol as illustrated in Figure 2. Table 1 shows the material properties used to define the stress-strain relationships for members. All specimens used for parametric study are listed in Table 2. Each specimen was used to study a specific parameter. The first specimen was designed according to the balance design procedure proposed by Roeder et al, and served asthe reference model. The column section of this specimen is I-shaped and the gusset plate used $6t_p$ elliptical clearance as shown in Figure 3(a). The column orientation and the gusset plate geometry were different in other specimens. For instance, the specimen WL2t had 2tp linear offsetto reflect current seismic design specifications, and specimen W90T15 had rotated column and the tapered gusset plate was connected to the web of the column, as shown in Figure 3(b).

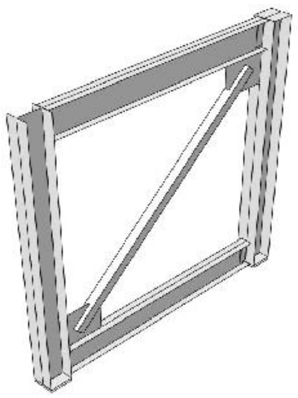


Figure 1. FE model of the reference specimen

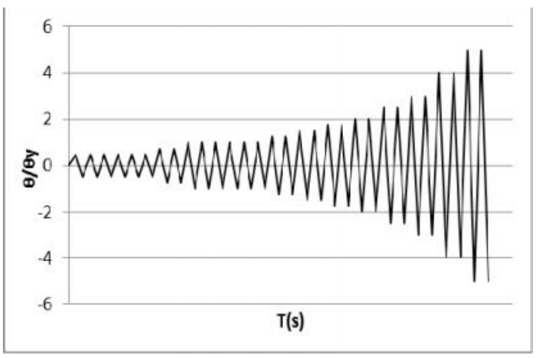


Figure 2. Normalized displacement history

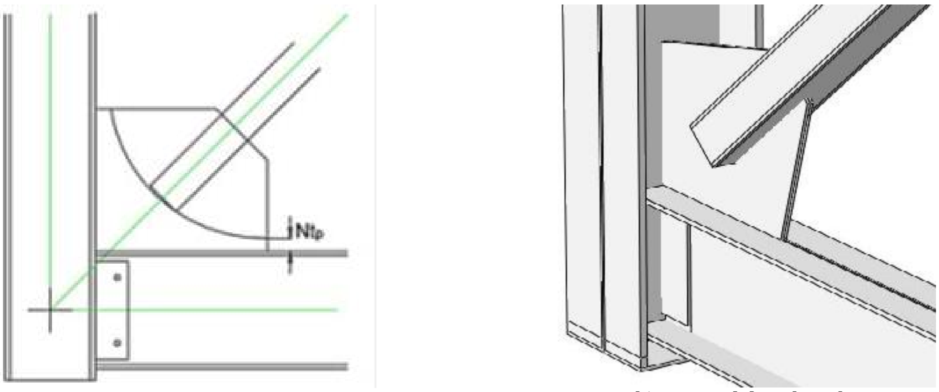
Table 1. Material properties of members

Members	Module of Elasticity (GPa)	Yield Stress (GPa)	Ultimate Stress (GPa)	Ultimate Strain (%)
Gusset plate, beam, and column	200	0.27	0.51	0.28
Bracing member	180	0.3	0.41	0.26

Table 2. Characteristics of numerical models

Specimen designation	Clearance distance	Brace length	Gusset plate geometry	Gusset plate size (mm)	Stiffener

		(cm)			
WE6t(Ref)	6t-E	400.52	Rectangular	635x533x9.5	-
WL2t	2t-L	341.52	Rectangular	864x762x12.7	-
WT15	6t-E	400.52	Tapered of 15	635x533x9.5	-
W90E6t	6t-E	400.52	Rectangular	635x533x9.5	-
W90L2t	6t-E	341.52	Rectangular	864x762x12.7	-
W90T15	6t-E	400.52	Tapered of 15	635x533x9.5	-
W90E6t-St	6t-E	400.52	Rectangular	635x533x9.5	312x152x10
W90L2t-St	2t-L	341.52	Rectangular	864x762x12.7	312x152x12
W90T15-St	6t-E	400.52	Tapered of 15	635x533x9.5	312x152x10



a) Elliptical clearance b) FE model with web connection

Figure 3.Elliptical clearance and web connection in models

In the FE model all members were modeled using four-node quadrilateral shell elements, which has 6 degrees of freedom at each node. Combined hardening was adopted to predict inelastic behavior of steel under cyclic loading. The brace was given initial imperfection magnitude of L/1000 according to the elastic buckled shape of the brace, where L is the length of the brace.A mesh refinement was carried out to determine the mesh size required to reach convergence and desirable accuracy while optimizing execution time. A mesh size of 10 mm by 10 mm was used at the middle of the brace and at the gusset plate connections to the beam and column where the plastic behavior is significant. A coarser mesh was used in other regions, where only limited inelastic deformation occurred.

The FE model was verified with the experiments carried out in TarbiatModares University (Fooladvand & Aghakouchak, 2012). Test setup of the experiments is shown in Figure 54. The centerline measurements of the specimen were 2 m by 2 m. The sub assemblage consisted of a brace (Box 60x40x6 mm), two beams (typically IPE270), two columns (IPB120) and two gusset-plate connections. Shear tab connection was used for beam-to-column connection in order to provide the hinge connection. All connections of the frame were welded connections and in FE model all members were tied together. Boundary conditions were applied to the frame to simulate the test conditions (Figure 4 and Figure5). Columns are subjected to the out of plane restraints at the locations. Pin and rollers supports were used at the base of each column to simulate the supports of the frame. Two specimens (Test-01 and Test-02), which were different in gusset plate design, were modeled. Detailed comparison of the FE model and the experimental results showed that they were in fair agreement (Figure 6). The inelastic FE analysis also could appropriately simulate the local buckling, stress distribution, and yielding.

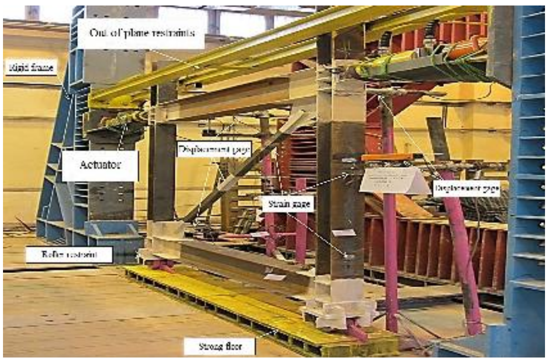
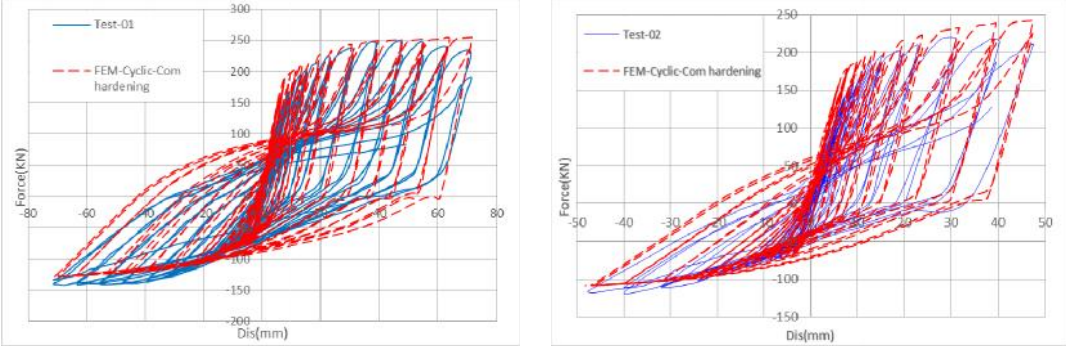


Figure 4. Test configuration of the experiment used for verification

Figure 5. Boundary conditions of numerical models



a) Test-01 b) Test-02

Figure 6. FE response in comparison with experimental results

FAILURE ANALYSIS

Although the numerical model can simulate the overall and local behavior, it is incapable of predicting the initiation of cracking, ductile tearing or fracture of the steel directly. The concept of critical plastic equivalent strain was used in order to get insight into the failure modesand to estimate ductility and energy dissipating capacity. This concept is based on the accumulation of the inelastic strain in critical region. The ε_{eqv}^{pl} (PEEQ) was used as a primary indicator of fracture and tearing potential for experimental test specimens at the center of the brace and the reentrant corners of the interface gusset-plate welds. Detailed comparison between experimental and numerical results has shown that, inelastic strain predicted by analytical model is a reliable index to be used as an indicator of initial cracking and fracture of the steel and weld.

Comparison of analytical results with experimental observations(Fooladvand,2012) showed that the minimum value of ε_{eqv}^{pl} in which fracture occurred at the middle of brace was 6.7, and the minimum value of ε_{eqv}^{pl} at the reentrant corners of interface gusset plate weld for crack initiation was 2.2. The threshold of ε_{eqv}^{pl} strongly varies for different material properties, location and coordination of cracks, and mesh size. Due to dependency of ε_{eqv}^{pl} and corresponding threshold limits on the mesh size, a constant mesh size was used in critical locations in numerical models to make the comparison of ε_{eqv}^{pl} meaningful. Although direct experimental result is not available for column local failure in the system, the mechanism is similar to the reentrant corners of interface gusset plate welds considering the heat affected zone and resulting brittle behavior. Thus, a value of 2.2 was considered for the limit of ε_{eqv}^{pl} for initiation of cracks in column at the location of connection to the gusset plate.

OVERVIEW OF NUMERICAL PARAMETRIC STUDY

In this study, the FE analysis was used to examine the column local flexibility. The studied parameters included the column orientation, the gusset plate geometry, and their interaction. The evaluated behavior included overall load-resisting capacity of the frame and local yielding in the brace, gusset plate, and columns. Based on equivalent plastic strain, the potential for crack initiation and fracture at the mid-span of the brace, corner of the gusset plate, and column at the location that was connected to the gusset plate were evaluated. They were used for calculation of the ductility capacity of the system.

Detailed description of specimens is presented in Table 2. The designation of each model demonstrates the properties of the specimen, the studied parameter and its value. As listed in Table 2 the simulated models included:

- Models WE6t and W90E6t, which studied Elliptical clearance of 6tp with flange and web connection, respectively;
- Models WL2t and W90L2t, which studied Linear clearance of 2tp with W shaped column;
- Models WT15 and W90T15, which studied Tapered plates with 15 taper angle with W shaped column;
- Models W90E6t-St, W90L2t-St, and W90T15-St which studied the previous models with Stiffener;

RESULTS

A summary of the results of parametric study, which include resistance capacity, drifts, and ductility capacity ratio corresponding to the assumed limiting equivalent plastic strain values, are provided in Table 3.Peak resistance values were normalized to the reference model to facilitate the comparison. The ductility capacity ratio (μ_c) is defined as follow:

$$\mu_c = \frac{\Delta_c}{\Delta_y} \tag{1}$$

whereΔ_c is the lateral displacement of frame before failure(fracture) and Δ_y is the yield displacement of the frame. Figure 7 shows the comparison of hysteretic response of models with flange connection with that of models with web connection. (a, b and c)illustrates variation of ε_{eqv}^{pl} values in the middle of the brace, corner of the gusset plate, and critical zone of column in connection region in somemodels. These figures will be discussed in detail in the following sections.

Table 3. Summary of parametric study results

Specimen designation	Stiffness (kN/mm)	Tension resistance (kN)	Compression resistance (kN)	Drift ratio (%)	Ductility	Predicted failure
WE6t(Ref)	66.4	1126	440	2.67	10.4	Midspan of brace and corner of the gusset plate
WL2t	115%	115%	125%	1.91	7.8	Midspan of brace
WT15	93%	95%	86%	1.91	7.3	corner of the gusset plate
W90E6t	78%	88%	61%	2.67	10.9	Column web
W90L2t	82%	101%	75%	1.91	7.9	Column web
W90T15	85%	86%	52%	2.67	10.9	Column web
W90E6t-St	97%	102%	93%	2.67	10.9	Midspan of brace
W90L2t-St	86%	104%	80%	2.29	9.4	Midspan of brace
W90T15-St	79%	90%	50%	2.67	10.9	corner of the gusset plate

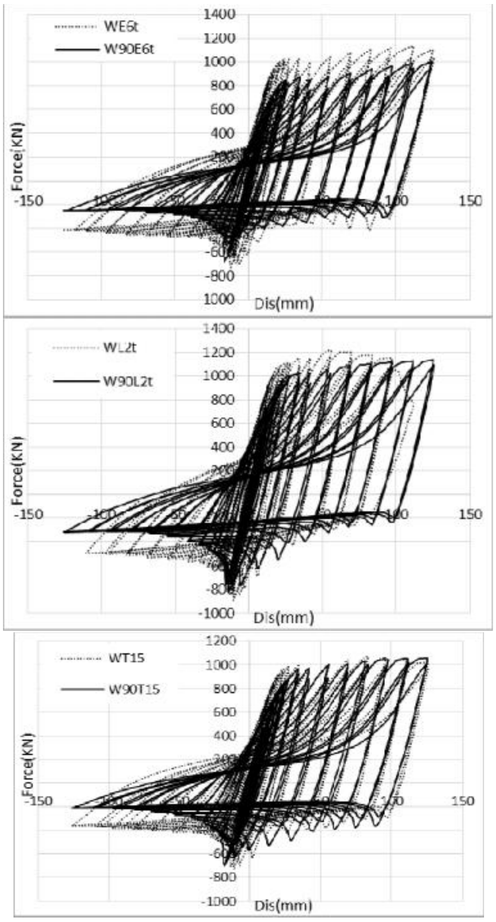


Figure 7: hysteretic response of models with flange and web connection

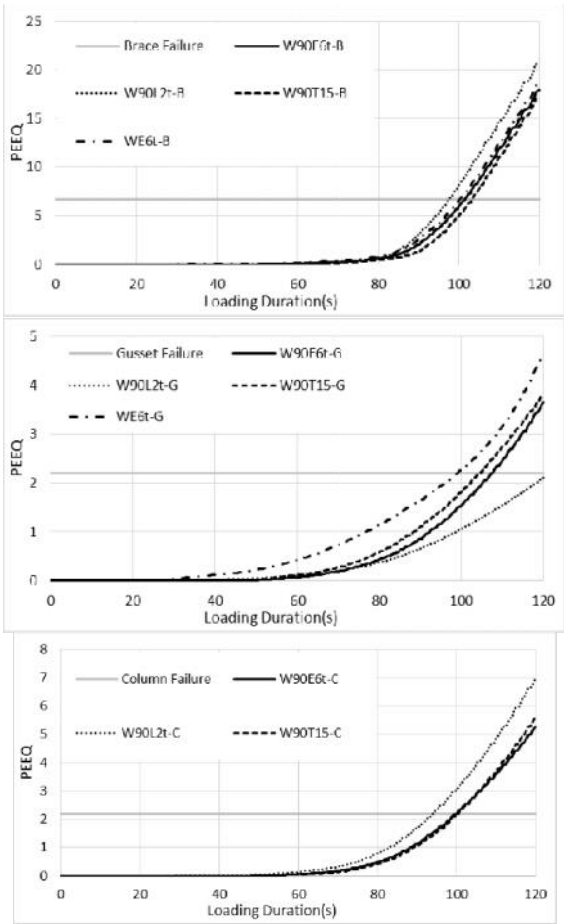


Figure 8: Growth of ε_{eqv}^{pl} in critical regions in models with flange and web connection

EVALUATION OF COLUMN ORIENTATION

To investigate the effect of column orientation and its local flexibility at the connection region, lateral load capacity was evaluated and local strain concentrations were observed at critical regions. As it is indicated in Table 3, the elastic stiffness values and lateral load capacity of the models with web connection reduced both in tension and compression. This could be because of local flexibility or moment of inertia of the column. Local behavior including yielding, cracking, and fracture in the framing members and the connections has an important role in the seismic performance of the system.The results showed that column orientation affects the ε_{eqv}^{pl} in the center of the brace, corner of the gusset plate, and column at connection zone. The curves of plastic equivalent strain at critical locations for some models are depicted in The inelastic strain concentration at connection region is depicted for specimen W90E6t in There was a slight decrease in ε_{eqv}^{pl} at the middle of brace and corner of the gusset alongside with a significant increase in that in column in models with web connection. This may cause undesirable fracture in column web and change the mode of failure in the system. This undesirable failure results from a local flexibility caused by the absence of column web in the line with gusset plate. This flexibility allows the web of the column to deform repeatedly in cyclic loading which leads to accumulation of inelastic strain and makes the column susceptible to fracture. Regarding the interaction of gusset plate geometry and column orientation, it can be seen that gusset plate with 2tp linear clearance increases the concentration of the ε_{eqv}^{pl} at the column web more than the two other gusset plates. It originated from linear clearance distance which leads to larger and stiffer gusset plates.

These are important observations regarding the effects of column orientation on the behavior of SCBFs. First becausereduction in lateral load capacity leads to lower energy dissipation of the system during earthquake. Secondly, the columns are expected to show limited inelastic deformations and preferred, ultimate failure of the SCBFs is fracture and tearing at the brace and corner of the gusset plate, respectively; however, analytical results suggest that local flexibility of the studied columns at the connection region imposes large inelastic deformation demand on the column and could changes the failure mode of the system.

EVALUATION OF STIFFENED COLUMNS

The result of this study shows that although frames with web connection possess ductile performance, their displacement capacity is restricted by the column yielding; therefore, they do not exhibit a preferred seismic performance and failure mode. Hence, in this study, stiffening approach was proposed to compensate the deficiency of these frames and facilitate the preferred seismic performance.

The effects of relative flexibility at connection region on the overall and local response of the system is not negligible. Columns play a crucial role in resisting gravity loads and stability of the structure; thus, the plastic behavior induced by mentioned flexibility could impose some risks on their behavior. In addition, it can raise the costs of rehabilitation after earthquakes. Thus, the proposed approach was considered to restrict that flexibility. To reach this aim, a horizontal stiffener was added to the columns, as shown inFigure 10. The thickness and width of the stiffener was equal to thickness of the gusset plate and width of the column, respectively. The best location of the stiffeners was h/8 from the top of the gusset plate, where h is gusset plate height. The stiffener mobilizes the flange of the column in frames which column is oriented in them.

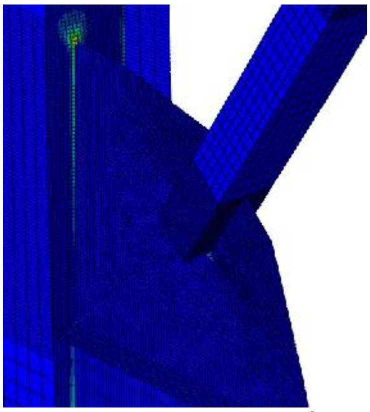


Figure 9. Concentration of ε_{eqv}^{pl} at the connection area in model W90E6t

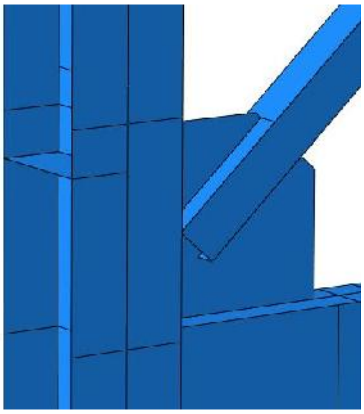


Figure 10. Location of stiffener in model W90E6t

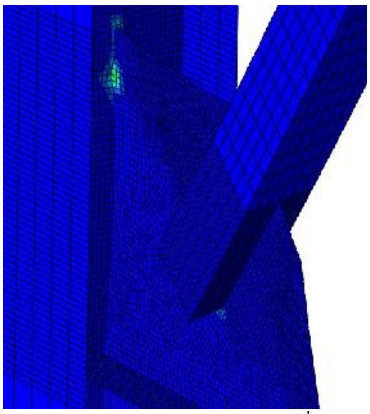


Figure 11. Concentration of ε_{eqv}^{pl} in connection area in model W90E6t-St

Using stiffener in columns did not significantly change the load resistance capacity of the models; however, it affected the concentration of the ε_{eqv}^{pl} in critical region, especially column web and corner of the gusset plate shows plots ofε_{eqv}^{pl} concentration for model W90E6t-St in connection region.There was a remarkable decrease in ε_{eqv}^{pl} in the column alongside with an increase in that in the corner of the gusset plate in stiffened model. Comparison of ε_{eqv}^{pl} growth in models W90E6t, W90L2t, W90E6t-St and W90L2t-St, obtained from the FE analysis is depicted in Figure 12. The summary of the results of these models is represented in Table 3. Based on limiting ε_{eqv}^{pl} in critical zones, the proposed approach resulted in larger drifts and shifted the possible failure mode from the column to the middle of the brace. As a result, the ductile performance of the system improved and balance behavior was achieved.

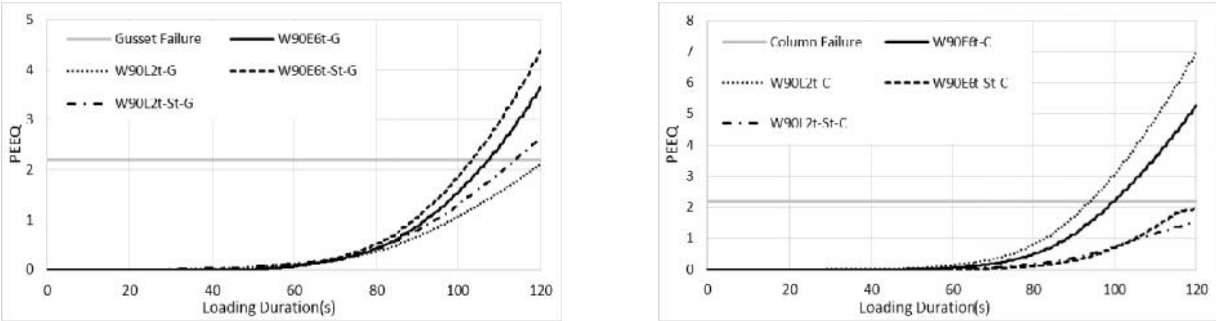


Figure 12. Growth of ε_{eqv}^{pl} in critical region in W90E6t, W90L2t, W90E6t-St, and W90L2t-St

SUMMARY ANDCONCLUSION

Three series of numerical models were constructed in order to investigate the effects of local flexibility at the region of the gusset plate connection to the column in SCBFs; using validated inelastic FE model and the equivalent plastic strain concept. First serieswith connection to the flange of the column served as reference models. In the secondseries, the connection was provided to the web of thecolumns.The global response andlocal behavior of these frames changed in comparison with reference models. The initial stiffness, tension and post buckling capacity of these models were reduced. The ε_{eqv}^{pl} concentration decreased in corner of the gusset plate and middle of the brace, but significantly increased in the column web at the location of connection to the gusset plate. Thus, the failure mode of these models was changed from middle of the brace and corner of the gusset plate to the column.

In order to eliminatethe detrimental effects of local flexibility on the performance of SCBFs, using stiffener at the connection region was investigated. The stiffener in third series of models with web connection did not affect the global response of the system; but decreased the concentration of plastic deformations in column and changed the failure mode in models. This method also increased the ultimate displacement capacity of the frame in some models, which led to improvement in ductility capacity of the system.

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