Gusset Connection Demand in Brace Frames with Moment-Connected Beams Under Cyclic Load

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Abstract

Gusset connections in buckling-restrained brace (BRB) frames are likely to experience large force demands during earthquakes, and their ability to maintain resistance is critical to the structure’s seismic performance. Primary gusset forces arise from axial forces developed in the BRB. For frames with fixed-end beams, forces also arise from flexural deformations of the frame. When the brace frame is loaded laterally, brace-induced gusset-connection forces are partially offset by flexure-induced gusset-connection forces. However, during the unloading phase of cyclic loading, force reversal in the BRB takes place over a shorter drift range compared to the frame, causing demands from the brace and the frame act in the same direction and compound at the gusset connection. While the AISC Seismic Provisions require that brace connection forces be combined with moments corresponding to the beam’s expected flexural strength, there is no guidance on how these forces should be combined.

The aim of this study is to define hysteretic conditions where the additive effect occurs and establish methods of quantifying the effect. To model the lateral resistance of the brace frame, a parallel spring model is considered consisting of a superposition of contributions from the brace and the frame. Results from force-deformation models of the brace and the frame are compared to examine whether simple models are sufficient to capture the phenomenon with sufficient accuracy. Finally, mechanics-based combination rules are established for force demands from the brace and the beam so that gusset connections can be designed more reliably and economically.
1. Introduction

Designing the gusset connection region of brace frames with moment-connected beams requires consideration of local forces at the column face arising from the brace and the beam. A recent study by Sabelli et al. (2021) presented design demands at the gusset connection from brace axial force and beam moment, showing that demands from beam moment typically acts in a direction that offsets the demand from the brace’s axial force, thereby reducing the overall demands at the connection. However, this is not necessarily the case during seismic loading, where load reversal under inelastic deformations causes demands from these two sources to compound during the unloading phase.

While the AISC Seismic Provisions require that brace connection forces be combined with moment corresponding to the beam’s expected flexural strength at the gusset connection, there is no guidance on how these forces should be combined. Simple superposition of moment demand from the maximum brace axial forces with maximum flexural forces acting in the same direction will overpredict the potential gusset-connection forces. This study will investigate force demands on gusset connections for brace frames with moment-connected beams to quantify the combination of forces from the brace and the beam so that connections can be designed to provide desired performance while achieving reasonable economy.

As will be discussed in more detail in subsequent sections, the additive effect is most prominent during the unloading phase if the brace unloads over a short range of deformation to reach its maximum strength in the opposite direction. In the unloading phase, a brace in a special concentrically braced frames (SCBF) moving from tension to compression does not reach a considerable compressive axial force due to buckling. The same brace traveling from compression to tension has relatively small stiffness, causing it to reach its maximum tensile strength over large
deformations. On the other hand, buckling-restrained-braces (BRB) have relatively balanced capacity and large stiffness when loaded in both directions, so they are able to reverse their internal force over a short range of axial deformation. As a result, the issue presented is more prominent for BRB frames and less of a concern for SCBF, thus BRB frames will be the main topic of discussion in this study.

![Figure 1: Hysteresis of HSS brace and BRB (Black et. al. 1980; Merritt et. al., 2003)](image)

**2. Analysis Methodology**

To create a model of force demands at the gusset connection, a typical beam-column connection with two braces framing into the joint is considered (Figure 2). For this connection, forces and moment at the mid-height of the beam where it intersects the face of the column can be determined using statics as the following:

\[ F_V = (F_{11} + F_{12}) \sin(\theta) \]  \hspace{1cm} (1)

\[ F_N = (F_{12} - F_{11}) \cos(\theta) \]  \hspace{1cm} (2)

\[ M_f = \frac{(F_{11} + F_{12}) \sin(\theta) d_c}{2} + M_b \]  \hspace{1cm} (3)

Where \( F_{11} \) and \( F_{12} \) are axial forces from each brace, \( \theta \) is the angle of the braces with respect to the horizontal axis, and \( M_b \) is the moment from the beam. \( M_f \) is determined by taking moment about the centroid of the joint assuming brace centerlines intersect at a common working point. These
equations show that the moment at the face of the column has contributions from the beam and the brace, while the two perpendicular forces are only a function of the axial force in the brace.

Figure 2: Free body diagram of typical beam-column connection

The moment and forces calculated with the three equations above can be used to determine the distribution of stresses on the column, which are then used to design the connection. Since determining the stress distribution is a statically indeterminate problem, designers typically assume a distribution (Figure 3), then rely on the lower-bound theorem and sufficient ductility in the connection to ensure the chosen assumption produces conservative results. A common stress distribution considered in practice is the uniform force method (Thornton, 1991). In this study, instead of focusing on the stress distribution, the main parameter of interest is $M_f$ from equation 3 as it is a more fundamental property that does not depend on the assumption of force distribution. The additive effect of moment at the face of the column holds true for any stress distribution method considered.
Figure 3: Potential force distributions from gusset to column

To observe how axial forces from the brace and moment from the beam in equation 3 interact, brace frames will be analyzed under a pseudo-static cyclic loading protocol. For simplicity, the lateral resistance of the brace frame is assumed to behave similar to a parallel spring, where the total lateral resistance at a specific story drift is the superposition of resistance from the frame and resistance from the brace. Making the parallel spring approximation allows for simple analysis of a variety of different frame and brace configurations without significant efforts in creating sophisticated models for each one. Examining how reliable the simplified model is when compared to a more complex model will be the subject of future study.

For this study, the brace frame will be subjected to displacement demands corresponding to 3% drift. While ASCE requires a maximum allowable story drift of 2% for typical buildings (2.5% under certain scenarios), a 3% story drift was considered to be more appropriate because lateral displacements for BRB frames are calculated using a C_d/R ratio of 0.625. The requirement implies that inelastic drifts are much smaller than elastic drifts, which is not accurate under most ground motions. As a result, it is more realistic to target a larger drift to partially correct for the pitfall.
3. Hysteretic Relationships

This section outlines the methodology used to transform force-deformation relationships for the BRB and the moment frame from testing data to lateral resistance versus drift relationships at the global scale. These relationships can be applied to equation 3 to calculate the total moment demand at the centroid of the joint in the parallel spring model.

3.1 BRB Force-Displacement

Sample cyclic test data of a BRB was obtained from the NEHRP Seismic Design Technical Brief No. 11 regarding BRB design (Figure 4). The test data is in terms of average axial strain and needs to be converted to story drift via the equation below:

\[ \varepsilon = IDR \cos(\theta) \frac{h}{L} \]  

Where \( \theta \) is the angle of the brace with respect to the horizontal axis, IDR is the story drift, \( h \) is the story height, and \( L \) is the width of a bay.

To determine the appropriate excursions to use in the model, a range of expected axial strains were calculated assuming that typical height to width ratios of a bay varies from 1 to 3. At a story drift of 3%, this translates to a strain range of 0.95% to 2.1%. The three outermost excursions in the test data lie in this range, thus is most representative of behavior during the unloading phase. Figure 4 shows digitized curves from tension to compression excursions; excursions from compression to tension were also digitized. Note that the additive effect of moments occurs in the unloading curve, so the shape of the initial loading curve is not critical to the analysis. Comparing the digitized excursions, it was found that the shape of each excursion is relatively similar to each other. As a result, the average of the three curves was deemed sufficiently representative of typical behavior and was fitted to a polynomial function (Figure 5). A power
function by Cofie and Krawinkler (1985) to describe the hysteresis loops, \( \sigma = Ke^n \), was explored but was found to give unsatisfactory fits.

Figure 4: Sample cyclic test data for BRB (NIST, 2015)

Figure 5: Tension to compression excursions and fitted curve transformed to the origin
For the purpose of the parallel spring model, the parameters of interest are horizontal force versus drift. The following transformation will be applied each axis of the fitted curve to convert to the appropriate parameters:

$$\text{IDR} = \frac{\epsilon L}{\cos(\theta) h}$$

$$V_{\text{brace}} = \left(\frac{\sigma}{\sigma_y}\right) \sigma_y A_{sc} (KF) \cos (\theta)$$

Where equation 5 is equation 4 rearranged, $V_{\text{brace}}$ is the horizontal resistance of the BRB, $\left(\frac{\sigma}{\sigma_y}\right)$ is normalized axial stress, $\sigma_y$ is yield stress, $A_{sc}$ is area of the steel core, $KF$ is the axial stiffness adjustment factor, and $\theta$ is the angle of the brace with respect to the horizontal axis. The $KF$ factor is used to convert the core area to an effective area. Catalogues from Corebrace provide $KF$ values for braces spanning various bay widths and height. Figure 6 shows a plot of $KF$ versus length between working points for a typical 14ft story height ranging from approximately 1.29 to 1.56.

![Figure 6: Distribution of KF factor for 14ft](image-url)
3.2 Beam Moment-Displacement

Force-displacement relationship for a beam in a moment frame was obtained from test data by Krawinkler and Popov (1982) shown in Figure 8. The moment at the face of the column from the vertical force at the cantilever tip can be obtained from statics for the purpose of obtaining local forces at the joint. A cruciform model (Figure 7) can be used to relate deformation at the cantilever tip to the story drift of a frame that the cantilever is a part of. This model has several assumptions as follows: there are inflection points at the midspan of beams and columns, there is no displacement at the midspan of beams, localized deformations from connections at the joint are neglected, and the beam is the first component to yield ahead of the column and the joint assuming the frame is designed following the strong column weak beam principle. By making these assumptions, it is possible to calculate contributions to story drift from the beam, the column, and the joint using the aforementioned test data.

Figure 7: Schematic of the cruciform model (Krawinker, 1978)

Krawinkler (1978) derived expressions for lateral displacement in the cruciform model due to deformation in the beam and the column, assuming bay width and beam moment of inertia are
the same on both sides of the joint. Converting the lateral displacement into story drift, the following relationships for story drifts are obtained:

\[ \theta_c = \frac{(h - d_b)^3}{12E I_c h} V \]  
\[ \theta_b = \frac{h^2 (L - d_c)^2}{12E I_b h L} V \]

Where \( h \) is the story height, \( L \) is the bay width, \( d_c \) and \( d_b \) are the depth of the column and beam, \( I_c \) and \( I_b \) are the moment of inertia of the column and beam, \( E \) is the elastic modulus of steel, and \( V \) is the horizontal force at the tip of the column. Drift from the joint is calculated assuming the joint deforms elastically, and is subjected to forces equal to its shear capacity of the code:

\[ V_{pyy} = 0.6A_j f_{yc} \]  

Converting to shear strain, or drift, the following relationship can be obtained:

\[ \theta_j = \frac{0.6f_{yc}}{G} \]

Where \( f_{yc} \) is the yield stress of the column and \( G \) is the shear modulus of steel. The total story drift of the frame is the sum of contributions from the three sources described above:

\[ \theta_{total} = \theta_c + \theta_b + \theta_j \]

To convert from \( P \) in the test data to horizontal resistance of the moment frame, vertical force at the tip of the cantilever is related to the horizontal force at the top of the column as follows:

\[ P = \frac{Vh}{L} \]

Rearranging to solve for the horizontal force at yield and relating force at the tip of the cantilever to the test specimen beam’s yield moment, the following is obtained:

\[ V_{yield} = P \frac{L}{h} = \frac{M_p L}{L - \frac{d_c h}{2}} = \frac{2f_{yb} S_b L}{h(L - d_c)} \]
Finally, the yield drift of the frame can be obtained from equation 11 by substituting $V_{\text{yield}}$ for $V$. The yield drift can be approximately mapped onto the x-axis in Figure 8 at the location where the force-deformation curve begins to deviate from the linear range. The corresponding ordinate can be mapped to the beam’s yield moment. Even though the test specimen is a W24x76, the hysteresis curve will be normalized to suit other beam sizes assuming its behavior is representative of typical moment connections of beams. Representative excursions of the hysteresis are digitized and fitted to a polynomial function similar to the procedure described for the BRB.

![Figure 8: Hysteresis loop of beam in moment frame (Krawinkler and Popov, 1982)](image)

4. Results

For the first example, consider a portion of a brace frame shown in the right side of Figure 9, consisting of W14 X 120 columns, a W18 X 55 beam, and a BRB with a core area of 9in$^2$. The frame is 14ft tall and 20ft wide. Lateral force imposed on the structure will be transferred to the top left corner of the brace frame segment.
To illustrate the effects various assumptions regarding material behavior, representative excursions from the test data from the beam and the brace were used to derive two simplified models. Figure 11 shows the curve from test data in blue, an elastoplastic curve in orange, and a bilinear curve in gray. The elastoplastic and bilinear models were chosen because they are commonly used in practice. Analysis results using these curves will be presented to determine whether simplified models are sufficient to predict the brace frame’s behavior.
By subjecting the brace frame segment to the loading protocol shown in Figure 10, interactions between internal forces in the beam and the brace can be monitored at several key stages, each of which will be described in greater detail. Lateral resistance versus drift at each stage is shown in Figure 11 and moment at the joint centroid versus drift at each stage is shown in Figure 12. These plots were created assuming the beam and the brace exhibit elastoplastic behavior.

1) Brace Yield
When the brace frame is initially loaded, all components are still elastic up until the yield drift of the brace. The brace is stiffer compared to the frame, so it resists more lateral load. At the yield drift of the brace, the brace places more moment demand on the joint than the beam due to a combination of differences in stiffness and geometry of the frame. From the moment demand plot, it can be observed that moment from the brace acts in the opposite direction compared to moment from the beam.

2) Frame Yield
Past the yield drift of the brace, the brace is assumed to be perfectly plastic and does not resist any additional load. The frame continues to resist more lateral load, up to a maximum at its yield drift. At this point, the frame resists a smaller lateral load compared to the brace but places a significantly larger moment demand at the joint. As a result, the total moment demand at the joint reverses direction between stage 1 and stage 2.

3) Maximum Drift
Past the yield drift of the frame, no further changes occur in each component until the maximum drift is reached. During the first three stages, the free body diagram at the joint is shown in the left figure from Figure 13, where moment from the brace and the beam act in opposite directions. The direction of the total moment at the joint depends on the relative size of each component.
4) Brace Unloads

Since the yield drift of the brace is much smaller than that of the frame, the brace unloads completely while the frame continues to resist lateral load. For this specific example, the slope of the moment versus drift curve for the brace and the beam are similar in magnitude. As a result, the total moment demand at the joint shows little change between stages 3 and 4. The moment in the beam decreases compared to the previous stage, but the offsetting effect provided by the brace is no longer available. However, if the slope of the brace curve is larger due to a larger stiffness or a larger lever arm, total moment at the joint would increase rather than decrease slightly. This illustrates the effect of brace size and column depth on the moment demand at the joint.

5) Brace Yield in Opposite Direction

As the brace frame continues to unload, the brace yields in the opposite direction. Between stage 4 and 5, the brace and frame resist lateral loads in opposite directions. During this interval, moment demand from the brace reverses direction and acts in the same direction as moment from the beam as shown in the middle figure in Figure 13. At this stage, two scenarios are possible:

a) If the moment from the frame decreases quickly, or the axial force from the brace grows slowly, this stage may not be as severe compared to the previous stage (no brace axial force and maximum beam moment).

b) If beam moment decreases slowly and the axial force grows quickly, the total moment demand at the connection will be more severe than the previous stage.
6) Frame Unloads
As the brace frame continues to unload, the frame eventually unloads completely. Past this stage the behavior of the brace frame is similar to the initial stages, where moment demand from the brace and the frame act in opposite directions. This is shown in the right figure from Figure 13.

7) Frame Yield in Opposite Direction
With additional unloading, the frame yields in the opposite direction. Note that for this example, the moment this stage reaches a maximum at this point as the moment from the beam grows to become much larger than moment from the brace.

![Figure 10: Loading protocol](image)

![Figure 11: Lateral resistance of components during load cycle](image)
From the simple example, interactions between the brace and the frame can be observed at various stages of loading. The maximum moment at the joint did not exceed the moment capacity of the frame, thus the joint can be designed for a lower moment demand as follows:

\[ M_{total} = |M_{beam} - M_{brace}| \]  

Where \( M_{beam} \) is the beam’s expected moment capacity and \( M_{brace} \) is the moment demand at the expected strength of the brace.
If the same example is repeated with a larger brace area, contributions to moment demand from the brace becomes much larger (Figure 14). The maximum moment demand at the joint in this scenario is larger than the individual moments from the brace or the beam. In this scenario, the following equation can be derived to estimate total moment demand:

\[ M_{total} = M_{brace} + \left(1 - \frac{2\Delta y_{brace}}{\Delta y_{frame}}\right)M_{frame} \]  

(14)

Where \( M_{brace} \) and \( M_{frame} \) are each component’s moment capacities and \( \Delta y_{brace} \) and \( \Delta y_{frame} \) are the yield drift of the brace and the frame respectively.

Figure 14: Moment from components – large brace

Comparing results using an elastoplastic model to a model using hysteresis curves described earlier, the model using hysteresis curves produces a smaller moment demand at the joint (Figure 15). This means the elastoplastic model is more conservative. Consequently, equations 13 and 14 can be used to obtain a somewhat conservative design moment at brace connections. However, despite being somewhat conservative, the proposed equations give design moments that are much more economical compared to the naïve method of adding the maximum moment from the brace and the frame in the same direction.
While this study proposed new mechanics-based relationships to predict moment demand at brace connections, the short timeframe allocated to the study means there are areas that are not yet investigated. An important area of future work is validating the accuracy of the parallel spring model against more detailed models. A more sophisticated model can account for certain assumptions made in the current model, such as assuming inflection points occur at midspan of beams and mid-height of columns, that are not necessarily accurate under all circumstances. Another area of work is testing the relationship developed for a variety of brace frame configurations to determine a range of possible values for the magnification factor of moment demand at the joint. By doing so, design engineers will not be required to determine the yield drift of the frame and the brace when calculating the magnification factor. While the current study employed a target drift of 3%, it is only an estimate of the drift typical frames are expected to experience. Future work could investigate the relationship between drift and magnification factor.

Figure 15: Moment from components using hysteresis relationships – large brace

5. Future Work

While this study proposed new mechanics-based relationships to predict moment demand at brace connections, the short timeframe allocated to the study means there are areas that are not yet investigated. An important area of future work is validating the accuracy of the parallel spring model against more detailed models. A more sophisticated model can account for certain assumptions made in the current model, such as assuming inflection points occur at midspan of beams and mid-height of columns, that are not necessarily accurate under all circumstances. Another area of work is testing the relationship developed for a variety of brace frame configurations to determine a range of possible values for the magnification factor of moment demand at the joint. By doing so, design engineers will not be required to determine the yield drift of the frame and the brace when calculating the magnification factor. While the current study employed a target drift of 3%, it is only an estimate of the drift typical frames are expected to experience. Future work could investigate the relationship between drift and magnification factor.
6. Conclusion

This study showed how moment demand from the brace and the frame partially offset when the brace frame is loading, and compound when the brace frame is unloading. Simplified equations were developed to predict the moment at the joint, making it possible for engineers quantify the magnitude of moment demand to consider for seismic analysis of brace frames with moment connected beams. The magnitude of moment demand is a function of the size of each component in the brace frame as well as the frame’s geometry.
References


