Seismic assessment of deficient steel braced frames with built-up back-to-back double angle brace sections using OpenSees modelling

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SUMMARY:
Steel braced frames constructed with built-up back-to-back double angle bracing members have been used extensively in past decades, prior to the implementation of design provisions for ductile seismic response. Braced frames of this type may lack ductility in their bracing members or brace connections. In this paper, a numerical model is proposed and validated to simulate the inelastic seismic response of double angle braces including brace connection failure. The model is developed in the OpenSees platform with the force base nonlinear beam-column element and the fiber representation of the cross-section. Initial out-of-straightness and residual stress conditions are accounted for. Contact elements are used at stitch connections and zero-length elements with pinching4 material are selected to model axial and flexural inelastic deformation responses in the bolted connections. The buckling strength of single angle members is verified against code design equations for compression members. The model of built-up back-to-back double angle bracing members is validated against results obtained from full-scale quasi-static cyclic physical tests performed on double angle brace specimens in an 12 MN load frame and in a single-bay braced frame. The calibrated model is found to reliable predict the inelastic cyclic response of double angle bracing members. A sensitivity analysis is performed to define modelling parameters for future studies. The influence of the number of stitch connectors on the brace buckling strength is compared to that obtained when using code design equations.

Keywords: Bracing member, Built-up double angle section, OpenSees, Model validation, Sensitivity analysis.

1. INTRODUCTION

Built-up double angle sections have been used extensively as brace sections to construct steel braced frames in the past decades. Several steel braced frames using double angle sections were constructed prior to the implementation of seismic design provisions. Accordingly, the common problem of these structures is the lack of ductility in their bracing members and connections. In particular, failure modes such as shear failure of bolts as well as net section or block-shear failure of the braces at the connections are characterized by limited inelastic deformation capacity and sudden degradation of the brace resistance (Hartley et al. 2011). These behavioural characteristics may lead to nearly complete loss of lateral resistance and collapse of the entire structure in case of a strong seismic event. Accurate prediction of the collapse point is a key step in the seismic assessment of existing buildings.

In this study, a detailed numerical model is proposed to reproduce the seismic cyclic inelastic response of built-up back-to-back angle bracing members using the OpenSees platform. The model accounts for several factors including flexural buckling of the individual angles, angles acting in pairs, physical contact between the two angles, the influence of the stitch connections on the buckling and post-buckling responses of the angles, and the nonlinear flexural and longitudinal responses of the brace end connections. A first model is developed with the aim of studying the flexural buckling response of a single-angle member. Then, the model is extended to reproduce the cyclic inelastic response of a double-angle bracing member, including inelastic end connection behaviour. Further, the model is validated against the results from full-scale quasi-static cyclic testing performed on double angle bracing members with stitch connections. Sensitivity analyses are carried out to examine the influence of the number of fibres, number of elements, and number of integration points on the brace response predictions.

2. NUMERICAL ANGLE BRACE MODEL

2.1 Single Angle Brace Model

The single angle section L 127x76x9.5 is selected to illustrate the numerical model as this shape has been used for the specimens of the test program that is described later for validation purposes. As shown in Fig. 1a, the brace is modelled using 16 nonlinear *forced-based beam-column* elements. Each element includes 4 integration points and the cross-section of each element is discretized with rectangular fibres. In Fig. 1b, a total of 160 fibres are used, which includes 20 segments along the width of each of the two legs and 4 layers of fibres across the angle thickness. Nonlinear material response is reproduced with the *uniaxial Giuffré-Menegotto-Pinto* (*Steel02*) steel material object exhibiting both kinematic and isotropic strain hardening properties. This modelling technique has already been successfully used in past studies on HSS bracing members (Aguero et al. 2006; Uriz et al. 2008). In addition, residual stresses linearly varying across the width of the legs are assigned to the cross-section fibers, based on the measurement data collected by Adluri and Madugula (1995). Figure 1c shows the adopted residual stress distribution for the L-127x76x9.5 section. Corotational geometric coordinate transformation is used to predict the buckling response. Initial out-of-straightness is included by means of half-sine initial deformation configuration with maximum deformation equal to *L/500* specified at the brace mid-length.

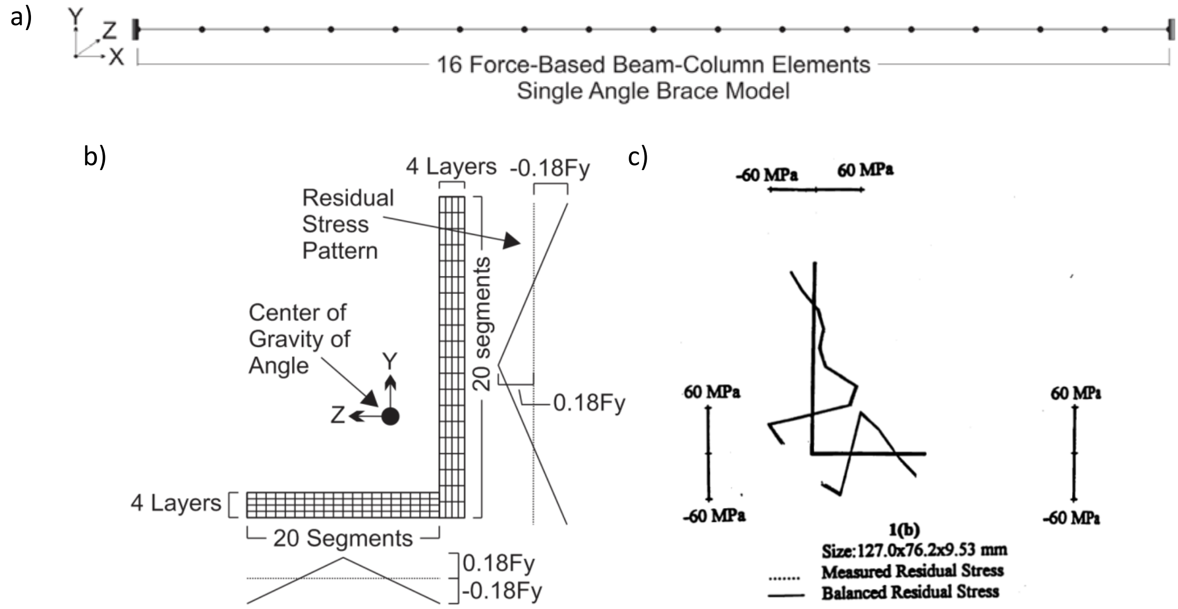

Figure 1. Single angle model: a) Beam-column elements; b) Brace cross-section with fibers and residual stress patterns; c) Residual stress distribution used for L-127x76x9.5

To check the buckling shape of a single angle brace, a fictitious 6 m wide by 4 m high braced bay was selected, which resulted in a brace length of 6095 mm. Both brace ends are assigned as fixed. The brace was analyzed under static incremental (push-over) analysis in which a 20 mm negative deformation inducing compression was applied at one end of the brace in 1000 steps. Buckling of the brace occurred about the minor principle axis of angle cross section, as expected. Figure 2 shows the buckled shape of the brace in Y and Z directions at the end of the analysis for three different initial imperfections (δ_0) conditions, which includes imperfection in both Y and Z directions, only in Y direction, and only in Z direction. Buckling deformations in the Z direction are approximately 4 times larger than the deformations in the Y direction. These observations are consistent with the direction of principal axes of the angle cross section. In addition, different imperfection conditions led to approximately 8% and 3% differences in lateral deformations along Y and Z directions, respectively. These differences are small and may be ignored. Thus, initial imperfection is only applied in the Z direction in the subsequent analyses.

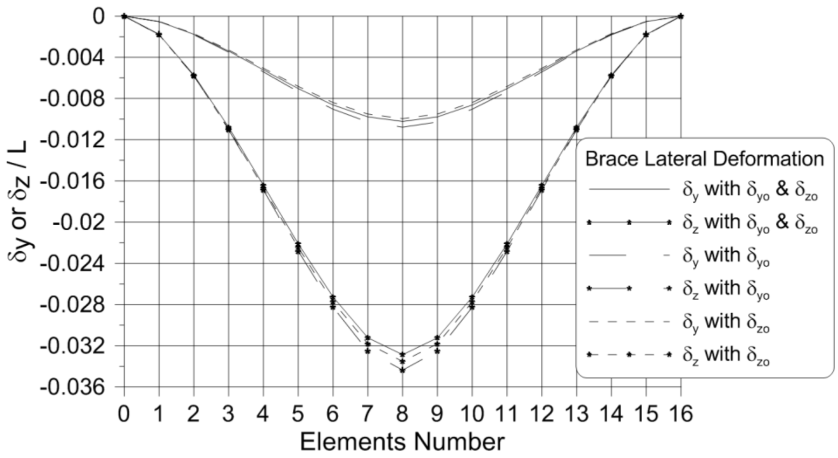

Figure 2. Lateral deformation in Y and Z directions resulting from different initial imperfection conditions

The buckling strength obtained from the analyses is validated by comparing with the nominal (unfactored) compressive resistance, C_n, as specified in CSA S16-09 (CSA 2009) Canadian design standard for flexural buckling:

$$C_n = AF_y(1+\lambda^{2n})^{-1/n} \qquad with: \lambda = \frac{KL}{r}\sqrt{\frac{F_y}{\pi^2 E}} \tag{2.1}$$

where A is the gross cross-sectional area of the member, F_y is the steel yield strength, taken equal to 345 MPa for this comparison, λ is the non-dimensional brace slenderness, $n = 1.34$ for angles, KL is the member effective length, r is the radius of gyration about minor principal axis, and E is the Young's modulus ($E = 200000\ MPa$). The calculation is performed for 8 different member lengths in order to obtain different KL/r ratios varying from 20 to 200. A factor $K = 0.5$ was used in Eqn. 2.1 to represent the fixed end conditions of the model. As illustrated in Fig. 3, the model predictions are very close to the analytical values obtained with the code equation.

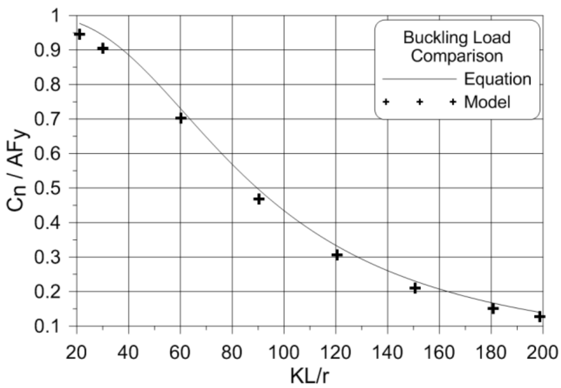

Figure 3. Buckling load comparison between the CSA equation and modelling analysis

2.2 Double Angle Brace Model

Figure 4a shows the model used to simulate the behaviour of a double angle bracing member built with L 127x76x9.5 angles with long legs back-to-back. This built-up double angle brace model is an extension of the single angle brace model. The two single angle brace models are connected back to back using *elastic beam column* elements linking together the centres of gravity of each angle. A very high stiffness was assigned to the *elastic beam column* elements, by using large cross section area, Young's modulus, and moment of inertia. the brace-to-gusset connections are modelled using *zeroLength* elements. At each brace end, the *zeroLength* elements are connected to a node located at mid-length of the *elastic beam column* elements. Three different material properties are assigned to these *zeroLength* elements: 1) *pinching4* material for nonlinear axial load- deformation response due to local yielding of the brace member and gusset plate in the connection region, bearing of the bolts against the connected elements, shear deformations of the bolts, and slippage of the bolts, 2) *uniaxial Giuffré-Menegotto-Pinto* (*Steel02*) steel material for nonlinear flexural response due to bending of the gusset plates upon buckling of the brace, and 3) *elastic* properties for torsional response. The components of the brace end connections shown in Fig. 4c are included in the connection model. The same stiff *elastic beam column* elements are used in axial direction to connect the brace-to-gusset connections to the fix end supports. In a structural model, these *elastic beam column* elements are connected to the beams or columns of the frame.

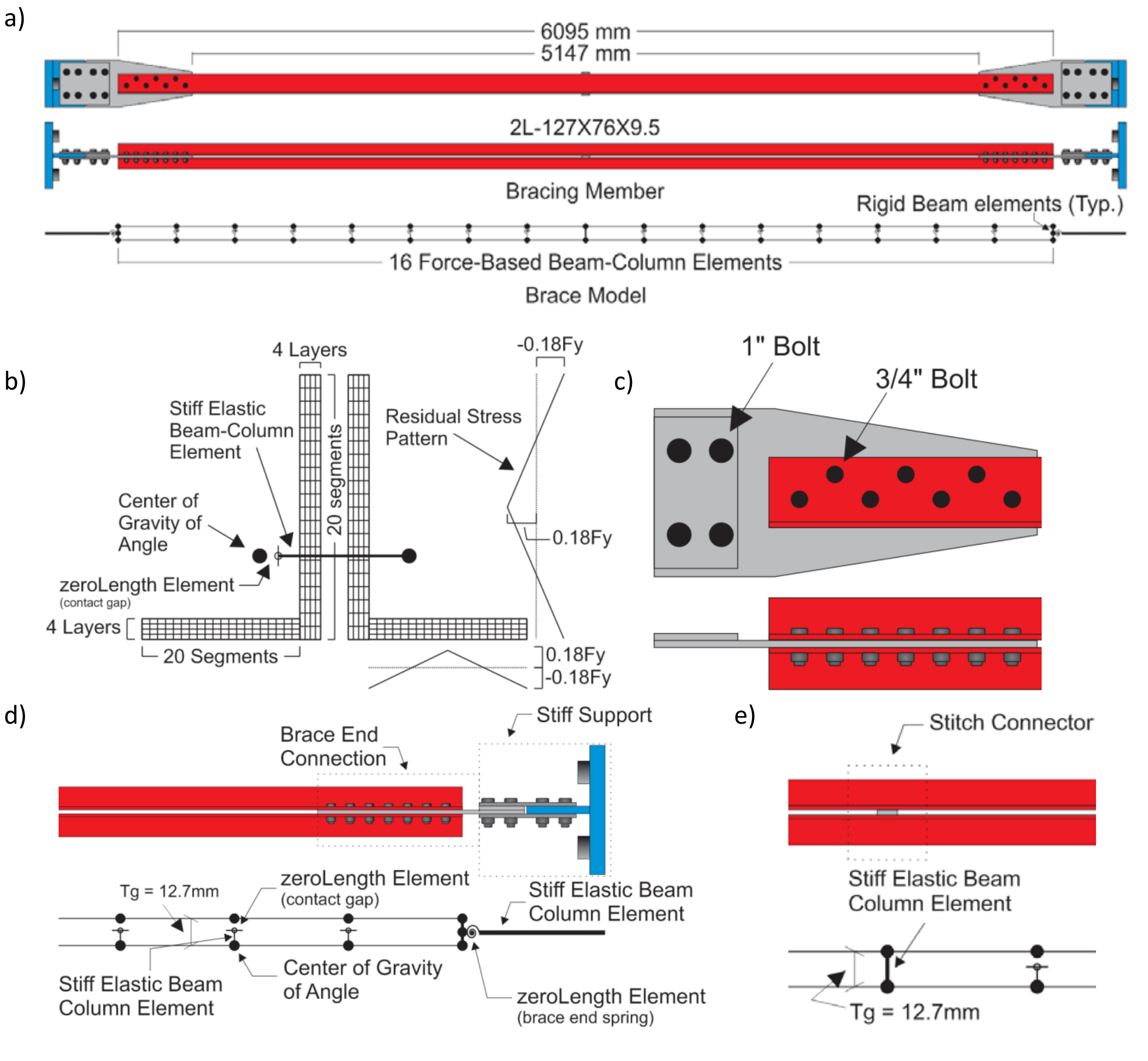

Figure 4. Brace model: a) Brace configuration and overall model; b) Brace cross-section with fibers and residual stress patterns; c) Brace end connection; d) Contact elements and brace end connection modelling; and e) Stitch connection at the brace mid-length

In Figs. 4b and 4d, stiff *elastic beam column* elements and *zeroLength* elements with *elastic-perfectly plastic* gap material are used at each pair of nodes of the brace member to reproduce the contact behaviour between the two angles when they buckle. Zero stiffness and strength properties were specified for the gap elements in tension. In compression, these elements were assigned 12.7 mm initial clear distance, corresponding to the thickness of the gusset plates, as well as high stiffness and strength properties. In this brace example, only one stitch welded connection located at the brace mid-length is used to connect the two angles. As shown in Fig. 4e, one simple 3D stiff *elastic beam column* element is used to simulate the stitch connector.

2.3 Experimental Validation

Two quasi-static cyclic tests were performed to validate the proposed numerical model: one in a single bay, single-storey braced frame setup and another one in a 12MN load frame. Experimental setups are shown in Fig. 5. The displacement loading protocol with stepwise incremented displacement amplitudes illustrated in Fig. 6 was applied to the specimen. The displacement amplitudes were based on storey drift displacements anticipated in buildings. The model material strength properties were based on the results of standard coupon tensile tests performed on the angle legs: $F_y = 338\ MPa$, $F_u = 490\ MPa$. The strain hardening parameters were adjusted to obtain good match with the cyclic test data.

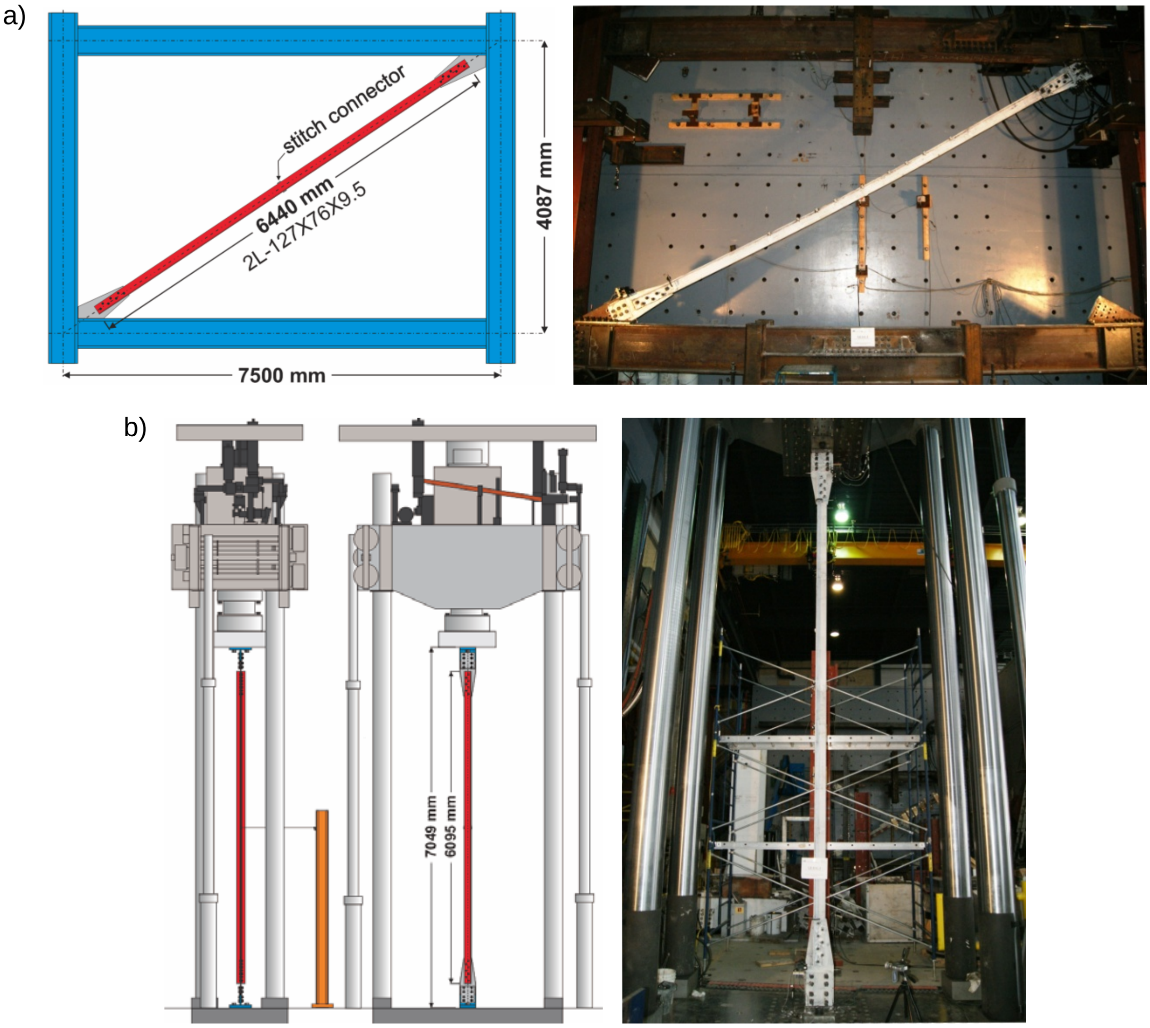

Figure 5. Test setup: a) 12MN load frame test; b) Single Brace braced frame test

As illustrated in Fig. 7, in both tests, the brace specimens buckled about the axis of symmetry of the brace cross-section, as expected. The predicted and measured brace hysteretic responses are compared in Fig. 8. The compressive resistance of the brace is very small, which is anticipated for a brace designed as a tension-only member with high slenderness (*KL/r* of the brace specimen is 173). The predicted and measured brace lateral deformation *vs* axial displacement responses are compared in Fig. 9. Both the brace axial strength and brace lateral deformation could be correctly reproduced with the proposed brace numerical model. The axial load versus axial deformation response of the brace connections are illustrated in Fig. 10 for both test configurations. The portion of the numerical model predicting the axial inelastic response of the brace end connections was calibrated against the braced frame test data. Then, the calibrated model was used to predict the connection response. Very good correlation is obtained by comparing the OpenSees model with the experimental results obtained in the 12 MN load frame test.

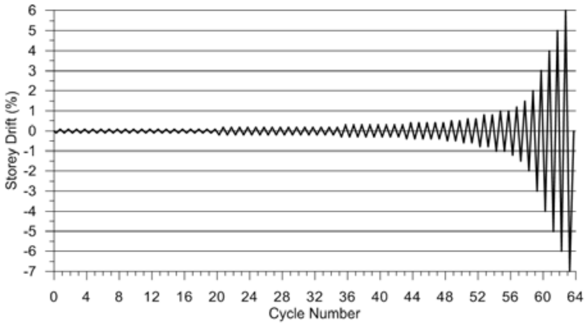

Figure 6. Imposed loading protocol

a) b)

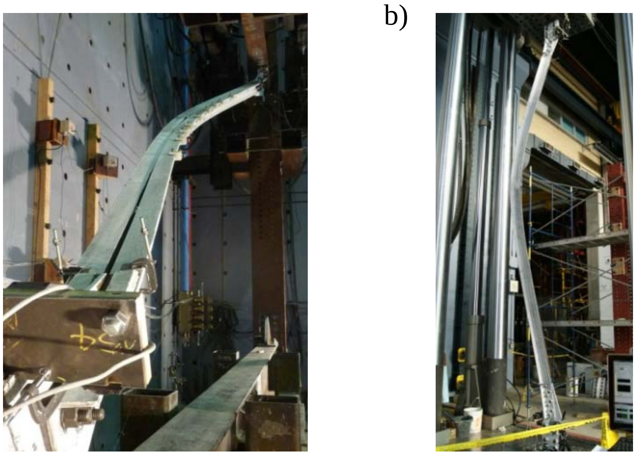

Figure 7. Buckling response of the brace specimens in: a) Braced frame test; b) 12MN load frame test

a) b)

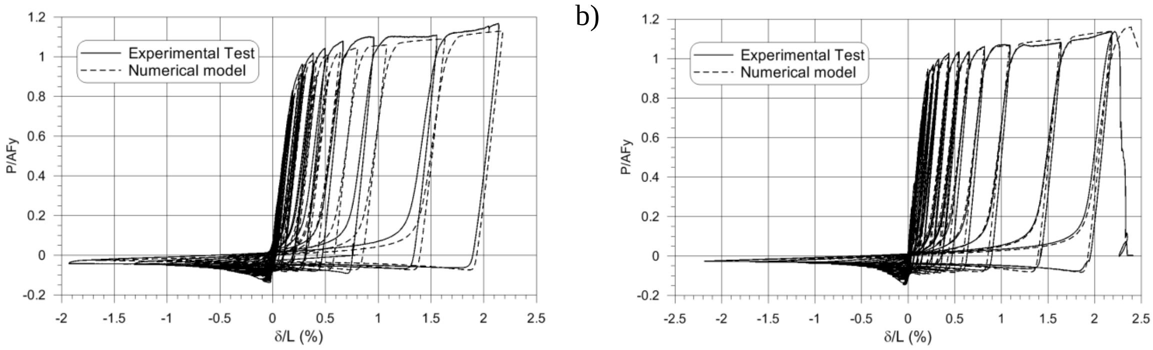

Figure 8. Brace hysteretic behaviour from numerical model and test measurements in quasi-static cyclic testing: a) Braced frame test; b) 12MN load frame test

a) b)

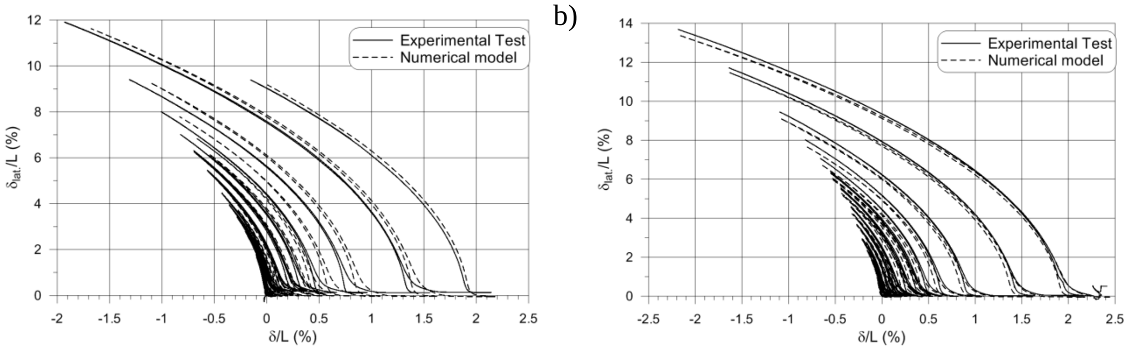

Figure 9. Brace lateral deformation from numerical model and test measurements in quasi-static cyclic testing: a) Braced frame test; b) 12MN load frame test

a) b)

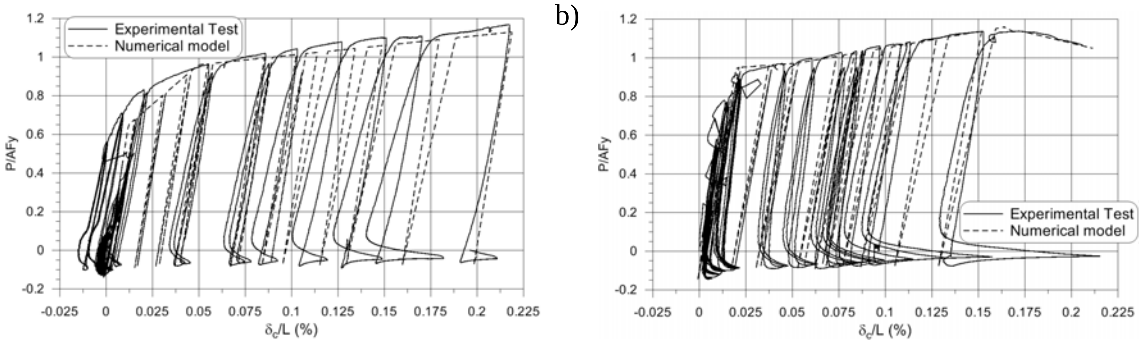

Figure 10. Brace connection response from numerical model and test measurements in quasi-static cyclic testing: a) Braced frame test; b) 12MN load frame test

2.4 Sensitivity Analysis

Model sensitivity analysis can provide useful data to improve the accuracy and effectiveness of the numerical model for future numerical simulations. The number of elements along each angle member, the number of fibre layers across the angle thickness, and the number of integration points were varied to examine the influence of these parameters on the brace response. In this sensitivity analysis, a stiff *elastic* material was used for the axial response of the *zeroLength* element simulating the brace end connections, instead of the *pinching4* nonlinear material. This simplification aimed at focusing on the brace inelastic response rather than on the combined brace and connection nonlinear responses. The loading protocol used in the test program was also employed in the sensitivity analysis. Buckling load P_u, hysteretic energy dissipated over the entire loading protocol, E_H, and maximum lateral displacement at brace mid span, δ_{lat} are the response parameters selected for comparison.

The effects of varying the number of elements and number of fibre layers across the angle thickness are examined first. For this study, the elements were modelled with 3 integration points. The analysis results, P_{ui}, E_{Hi} and $\delta_{lat.i}$, are given in Fig. 11. The results are normalized with respect to the values obtained when 32 elements and 16 fibre layers are considered (P_{u0}, E_{H0} and $\delta_{lat.0}$). Accurate prediction is generally obtained when only 8 elements are used along the brace length. This finding is consistent with the results obtained by Aguerro et al. (2006). However, buckling load prediction can be improved slightly when using 16 elements instead of 8. The number of fibre layers seems to have relatively smaller influence on the brace response. Using 4 layers in combination with 8 elements appears to be sufficient for achieving good predictions.

The influence of the number of integration points was examined for a model with 8 elements and 4 layers of fibres, based on previous analysis results. In this investigation, one stitch connector between the two angles was considered and other model parameters were assigned the same values as employed in the model used to replicate the brace specimen in the 12 MN load frame test. In Fig. 12, it is shown that 3 integration points are sufficient to properly predict the buckling load and energy dissipation capacity of the brace. No definite trend is observed for the lateral displacements but the results indicate that accurate prediction can be obtained with 2 to 5 integration points. Thus, using 3 integration points, as considered in the above case study, is acceptable.

The buckling resistance and energy dissipation capacity of double angle bracing members are both expected to increase as the number of stitch connectors is increased. This influence is examined in Fig. 13. The numerical model used to perform this study is the same model that was used in the sensitivity analysis with 16 elements, 3 integration points per element and 4 fibre layers across the flange thickness. Both the buckling load and energy dissipation capacity exhibit the anticipated trend as the number of stitches is increased. Lateral displacements at the brace mid-span tend to increase when using more closely spaced stitch connectors. The apparent discrepancy observed for the brace with 3 stitches is due to the position of the stitches relative to the buckled shape: as illustrated in Fig. 14, the deformed shape of the brace with 3 stitch connectors is flatter between the first and the third stitch connectors.

In Fig. 13a, the positive effect of increasing the number of stitch connectors on the brace compressive strength, as predicted when using the AISC 360-10 Specification (AISC 2010) requirements for built-up sections, is compared to that obtained with the numerical simulation results. For the calculations of the AISC member strength values, a *K* factor of 0.92 was used for brace overall buckling about the axis of symmetry, as determined by comparing the AISC prediction with the buckling load measured in the 12 MN load frame test with one stitch connector. An effective length factor of 0.5 was used for local buckling of the individual brace angles between the stitches, as recommended in the Specification for welded stitches. As shown, the influence of the number of stitch connectors on the brace compressive strength from the numerical simulations is comparable to that resulted from using the AISC 360-10 Specification.

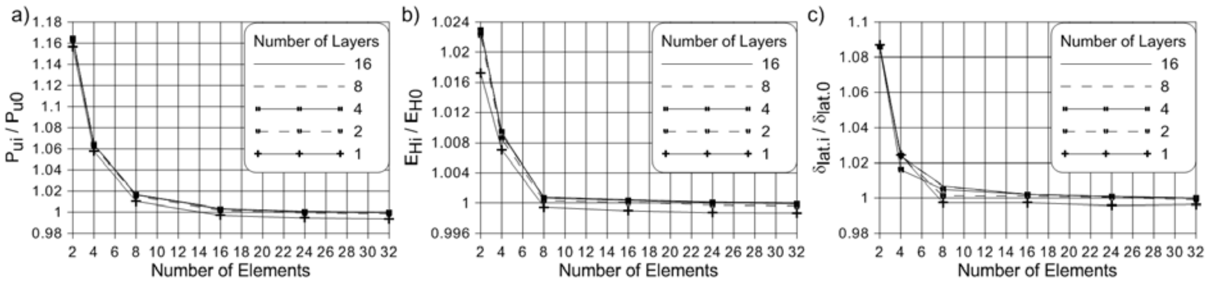

Figure 11. Influence of the number of elements and fibre layers across the thickness on: a) Buckling load; b) Energy dissipation; and c) Lateral displacement at brace mid-span.

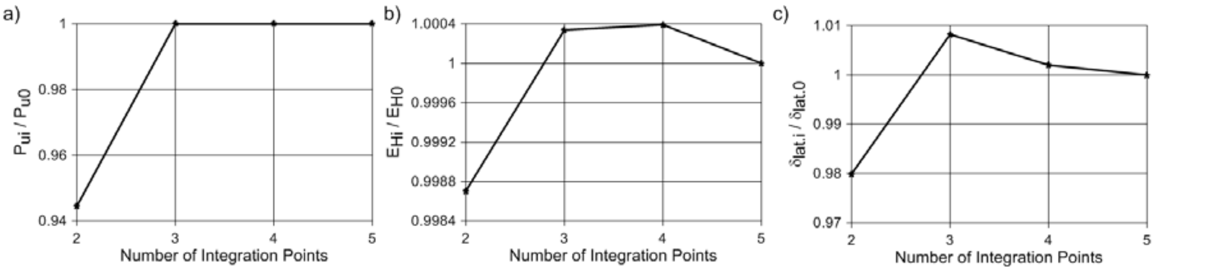

Figure 12. Influence of the number of integration points on: a) Buckling load; b) Energy dissipation; and c) Lateral displacement at brace mid-span.

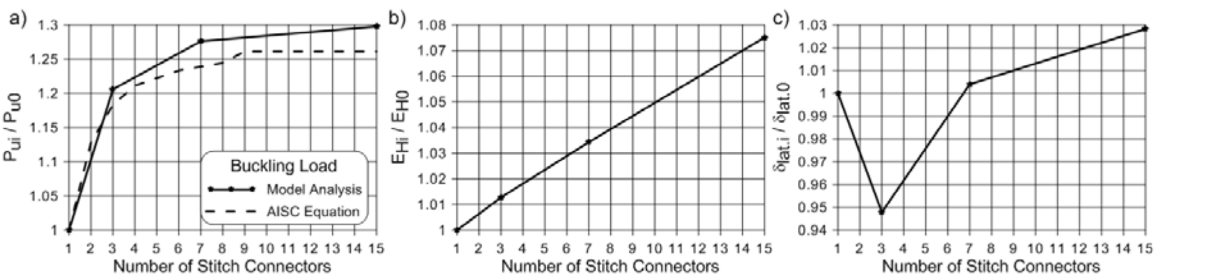

Figure 13. Influence of the number of stitch connectors between the two angles on: a) Buckling load; b) Energy dissipation; and c) Lateral displacement at brace mid-span.

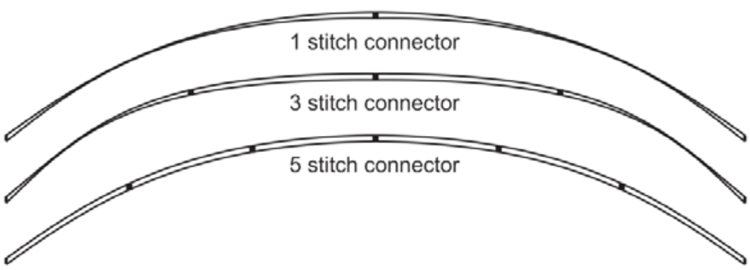

Figure 14. Double angle brace buckling shape.

3. CONCLUSION AND FUTURE WORK

A numerical model is proposed to study the buckling behaviour of a single angle and double angles steel bracing members using the OpenSees computer framework. In the numerical models, the brace was modelled using *forced-based beam-column* elements with fibre discretization of the cross-section. The *Giuffré-Menegotto-Pinto* (Steel02) material was used with isotropic and kinematic strain hardening properties. Initial out-of-straightness and residual stresses were considered in both models. The single brace model was used to predict the flexural buckling response of a fictitious brace with fixed end conditions. The accuracy of the proposed model was verified against the predictions resulted from using the code design equations. For the double angle brace model, contact elements were used to simulate the stitch connections along the length of the bracing member and *zeroLength* elements with nonlinear axial and flexural responses were used in the double angle member model to simulate the responses of the end connections. The accuracy of the double angle model was verified through comparisons with quasi-static cyclic tests performed on a single bay braced frame test and a 12MN load frame test on an individual bracing member. The study showed that the model was able to accurately predict the measured brace inelastic cyclic response, including the nonlinear behaviour of the end connections. A sensitivity analysis permitted to determine the required number of elements, integration points and fibre layers across the angle flange thickness. The model was also found to adequately predict the benefits of adding stitch connectors on the brace buckling loads. Energy dissipation and lateral displacements were found to increase when the number of stitch connectors was increased.

Future work will include the development of numerical models capable of predicting and reproducing all failure modes anticipated in steel concentrically braced frames designed prior to the implementation of current seismic design and detailing provisions. In particular, the work will focus on the development of accurate models for the prediction of the nonlinear behaviour and non ductile failures anticipated in brace connections.

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