Quasi-Static Testing and Correlative Dynamic Analysis of Concentrically Braced Frames with Hollow Steel Braces and Gusset Plate Connections

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SUMMARY
The ultimate deformation capacity of hollow section steel members subjected to cyclic axial loading is influenced by both member and cross-section slenderness. When these members are employed as bracing members in concentrically braced steel frames (CBFs) their ductility capacity is also influenced by the flexibility and strength of their gusset plate connections to beams and columns. The research project BRACED (Brace Response Assessment – Computation, Experiments and Design) investigates the ultimate behaviour of CBFs by validating recently-developed models for the ductility capacity of hollow section bracing members under low cycle fatigue conditions and assessing gusset-plate influence on global deformation capacity and energy dissipation. This paper describes an integrated experimental programme of quasi-static cyclic and shaking table tests on CBF specimens employing different combinations of brace member and gusset plate dimensions and details. These experiments evaluate the potential for improved ductility capacity offered by a balanced design approach and support model validation.

Keywords: CBFs, brace ductility capacity, gusset plate design, cyclic testing, numerical modelling

1. INTRODUCTION

The increase in popularity of concentrically braced frames (CBF) can be attributed to a combination of their desirable stiffness for reducing inter-storey drifts and their ability to provide a source of energy dissipation in structures that experience severe seismic ground motions. Extensive experimental research has addressed the seismic performance of brace members. Early cyclic tests of brace members carried out by Black et al. (1980) and Jain et al. (1980) showed that global slenderness has a significant influence on their overall hysteretic behaviour, and this remains a fundamental consideration when estimating the design ductility capacity of brace members. Tang and Goel (1989) later developed a criterion to predict the fracture life of bracing members and found that it is highly sensitive to section slenderness, particularly in compression flanges susceptible to local buckling. More recent studies have developed increasingly accurate models for ductility capacity prediction (Tremblay et al., 2003, Nip et al., 2010).

Gusset plate connections are typically used to connect brace members to the beams and columns in concentrically braced frames (CBFs). The stiffness strength properties of these gusset plates play an important role influencing the global ductility capacity of CBFs. In an experimental investigation of double angle bracing members connected to gusset plates by fillet welds, Astaneh-Asl et al. (1981) noted that correct design of these connections requires consideration of their deformation following brace member buckling and stresses. Both in-plane and out-of-plane buckling was investigated. It was found that, for out-of-plane brace buckling, permitting the formation of plastic hinges in gusset plates is crucial to avoid connection failure. More recently, the inelastic behaviour of the brace-gusset plate assembly during tensile yielding has received attention. The importance of balancing desirable yield mechanisms in both these components was investigated by Roeder et al. (2004), with analytical and experimental research studies identifying gusset plate design parameters to increase overall frame ductility.

Consideration of these developments in a European seismic design context, has led to the development of the BRACED (Brace Response Assessment – Computation, Experiments and Design) project which aims to investigate the ultimate behaviour of CBFs. The BRACED project, developed as part of the Transnational Access programme offered by the European Commission's Seventh Framework Programme (FP7) project SERIES, involves collaboration between researchers in Trinity College Dublin, Imperial College London, University of Ljubljana, National University of Ireland Galway, University of Liege and the Commissariat à l'Énergie Atomique (CEA).

The project entails shake-table tests of a full-scale single storey CBF system (Figure 1.1) that will investigate the behaviour of a series of selected brace section sizes and connection details under various loading regimes. These tests will be executed on the AZALEE shaking table located in the CEA's Seismic Laboratory in Saclay, Paris. The objective of this project is to experimentally and numerically investigate the ultimate dynamic response of CBFs under realistic earthquake loading through quantitative evaluations of ductility capacity. Hence, the validation of models for CBF ductility capacity under low cycle fatigue underpins this work. The experiments will employ distinct gusset plate connection boundary conditions (discussed in Section 3 below), alternative gusset plate geometrical design details and obtain essential experimental data required for performance-based design of CBFs and the validation of numerical models of the earthquake response of CBFs.

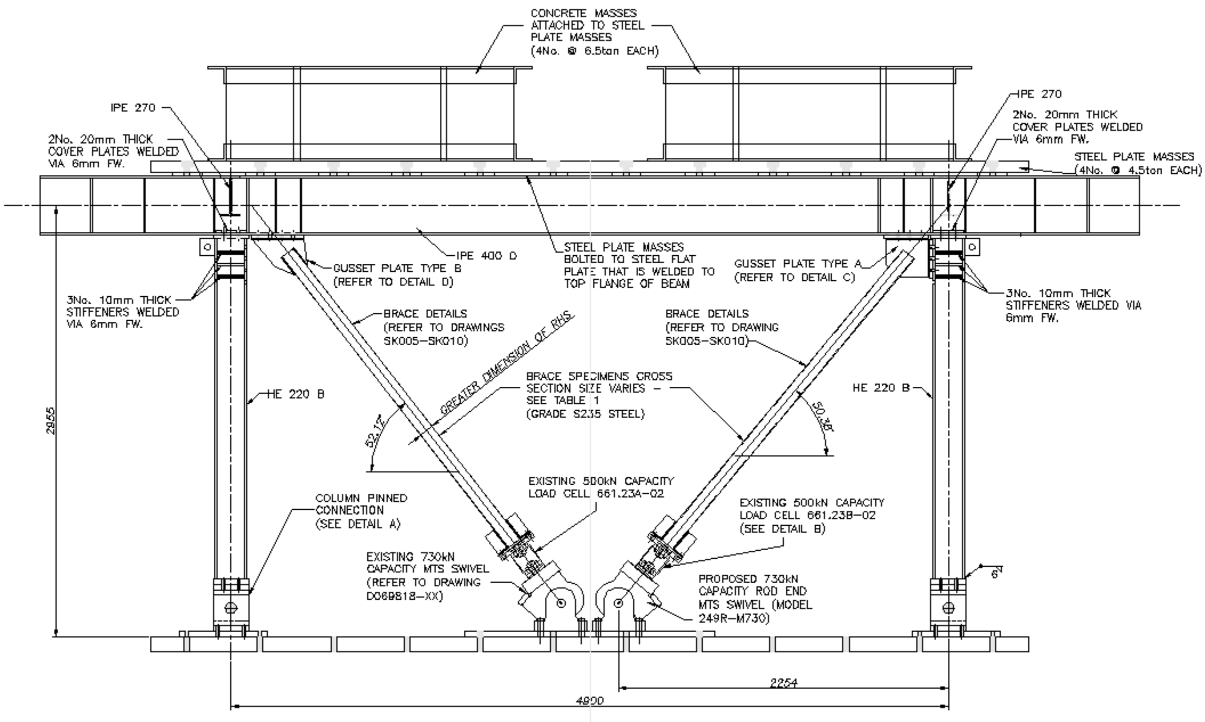


Figure 1.1. BRACED CBF test frame showing both gusset plate connection types.

Figure 1.1 shows the experimental configuration of the test CBF. Idealised pinned connections are used to connect the lower ends of the brace members to the shaking table, while the upper ends are welded to flexible gusset plates that are bolted to the beam and column members. This arrangement allows the brace-gusset plate assemblies to be replaced between experiments, allowing different combinations of member and connection properties to be investigated. The frame beam and columns are over-designed to protect against plastic deformation, limiting energy dissipation to the brace members and gusset plate connections.

The aforementioned ductility capacity model by Tang and Goel (1989), relied on prior knowledge of the load-displacement response. Tremblay (2002) established an empirical relationship between member displacement ductility, $\boldsymbol{\mu_\Delta}$, and global slenderness $\bar{\boldsymbol{\lambda}}$:

$$\mu_\Delta = 2.3 + 8.3\bar{\lambda} \tag{1.1}$$

It was later noted by Goggins et al. (2006) that the ductility capacity was influenced by member yield strength, member slenderness and local slenderness (i.e. section width-to-thickness ratio). Using experimental data from a suite of cyclic tests, the following linear relationships were developed:

$$\mu_\Delta = 26.2\bar{\lambda} - 0.7 \tag{1.2}$$

$$\mu_\Delta = 29.1 - 1.7(b/t) \tag{1.3}$$

where $\boldsymbol{b}$ is the outer section width; and $\boldsymbol{t}$ is the section wall thickness. These equations are separate linear relationships between displacement ductility and global slenderness (Eqn. 1.2) and local slenderness (Eqn. 1.3). Nip et al. (2010) later showed an overestimation of the displacement ductility in Goggins' (2006) formulations during a series of tests that used a broader range of local and global slenderness values. The following relationship was derived which incorporates both types of slenderness in the prediction of member displacement ductility for hot-rolled members:

$$\mu_\Delta = 3.69 + 6.97\bar{\lambda} - 0.5(b/t\varepsilon) \tag{1.4}$$

where $\varepsilon = \sqrt{235/f_y}$.

The above equations were derived from experiments on brace members with idealised boundary conditions, and do not account for the possible contribution of connection yielding to system ductility. Existing methods for gusset plate connection design are reviewed in Section 2. As a complement to the BRACED shaking table tests, a series of cyclic tests investigating the interaction of some brace-gusset plate connections and proposed design alternatives are discussed in Section 3. Simulation models of the BRACED frame developed using the open source earthquake simulation software OpenSees (McKenna 1997) are detailed in Section 4. This section features the process of obtaining ultimate displacement values using OpenSees earthquake time-history analyses and scaling the experimental shake table excitations.

2. EXISTING DESIGN METHODS FOR CBF FRAMES

In the seismic design of CBFs to Eurocode 8 (CEN, 2004) normally only the resistance of the tension braces is included in the analysis of seismic action effects. Because the brace compression resistance need not taken into account, European design practice tends to employ bracing members that are more slender than those encountered in other regions. Yielding should occur in the brace members before failure in the connections and before yielding or buckling of the beams and columns.

System ductility is achieved through individual brace member and connection design.
When subjected to strong ground motions the braces are expected to undergo large inelastic deformations in the post-buckling range leading to the formation of plastic hinge regions. As bracing members are susceptible to local buckling, the large flexural stains at the plastic hinge locations often lead to brittle failure due to fracture. This behaviour can occur at low storey drifts and results in reduced system ductility and can induce excessive ductility demands on beams and columns.

This is the premise of the design principles for the gusset plate connections of 'Special Concentrically Braced Frames' (SCBF) as set out by the AISC Seismic Design Provisions (AISC, 2005a) and the AISC Uniform Force Method (UFM) (AISC, 2005b). Gusset plates are the predominant method used to connect brace members to the structural frame. Typically, the gusset plate is aligned in-plane with the frame in a vertical direction. The direction in which compression braces buckle is dependent on the orientation of the section shape and the brace end restraints provided by the gusset plate. For out-of-plane brace buckling, member end rotations induce weak axis bending in the gusset plate due to the reduced stiffness of the plate in the out-of-plane direction. At larger storey drifts, the end rotation in the post-buckled brace is accommodated by the formation of plastic hinges in the gusset plates. To permit this, a free length is incorporated in the gusset plate design perpendicular to the end of the brace and the assumed line of restraint (Figure 2.1). This is achieved in the gusset plate design by allowing the brace-gusset connection to terminate before the line of restraint. This is the first gusset design method investigated in this paper, and is known as the Standard Linear Clearance (SLC) model. The recommended size of the free length is between $2t_p$ to $4t_p$ where $\boldsymbol{t_p}$ represents gusset plate thickness.

Gusset connections are typically designed with initial overall dimensions $\boldsymbol{l_h}$ and $\boldsymbol{l_v}$ governed by the alignment of the brace centreline with the intersection of the beam and column centrelines (known as the work point, in Figure 2.1). Once the maximum forces to be transferred through the braces are established the welds or bolts used to connect the brace to the gusset plate can be specified. If welds are used, their lengths will be determined by the initial sizing of the gusset plate dimensions $\boldsymbol{l_h}$ and $\boldsymbol{l_v}$ and the specified clearance length.

The gusset plate yield and buckling strengths are then calculated. This part of the design procedure is developed on the concept of the Whitmore section (Whitmore, 1950), which proposes that the axial force in the brace member is transferred through a section with a predefined width. This axial force can be distributed as a uniform stress over the section, which is sized to remain elastic. The width is defined with projection lines from the brace-edge of the gusset plate to the end of the brace at 30° (Figure 2.2a). For the buckling strength of the gusset plate, the Thornton (Thornton, 1991) model augments this design concept, by treating the gusset plate as a slender strut element with an effective length and Whitmore section area. The strut length is taken as the average of three lengths projected from the Whitmore width to the beam and column flanges (Figure 2.2b). Given the restrained boundary conditions on two sides of the plate, an effective length coefficient of $K = 0.65$ can be justified. This assumes that the effective length of the gusset plate is nearly fully restrained against rotation at each end and sidesway buckling is prevented (Roeder et al., 2004). Other methods suggested by Yam and Cheng (1994) include the Modified Thornton method which recommends a 45° plate stress distribution angle, introduced to take into account the effects of thin plate behaviour. When the column buckling formula is used, the load redistribution due to yielding is neglected as the formula considers a rectangular column directly beneath the Whitmore section. Following this the bolts and welds connecting the plate to the beam and columns of the CBF are sized according to the maximum tensile force to be transferred from the brace with overstrength factors applied as appropriate for capacity design.

These seismic design methods are based on the concept that CBF response depends largely on the behaviour of the brace member. Consequently, the connection is required to be much stronger than the brace and the connection is not considered as a potential dissipative zone. The requirement that the connection remain elastic usually leads to large gusset plates that are often uneconomical. Moreover, an examination of the seismic provisions by Roeder et al. (2005) showed that under large inelastic deformation demands, local damage frequently occurred in beam and column members adjacent to gusset plates. Improved and more reliable overall CBF behavior could be achieved by allowing some limited tensile yielding in carefully sized and detailed gusset plates.

This has led to the development of the balanced design approach by Roeder et al. (2004). Traditional seismic design methods utilize plastic design where beams and columns are designed to remain elastic and the brace members are the primary yielding mechanisms that achieve sufficient energy dissipation for the frame through inelastic deformation. The balanced method permits a secondary yielding mechanism to occur in the connection after the primary yield has occurred. While investigating three different types of moment resisting frames Roeder (2002) found that balance conditions between the yield mechanisms provided optimal connection performance. When applied to the design of CBFs, balanced design results in smaller, thinner gusset plates, for which alternative detailing rules are required to avoid plate buckling. This is one of the primary design considerations investigated in the cyclic tests described in Section 3 below.

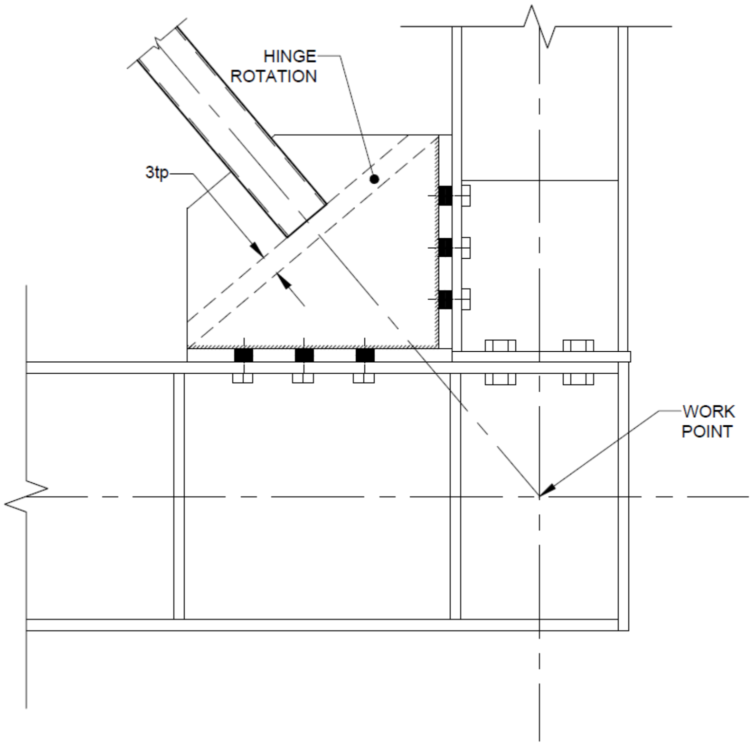


Figure 2.1. Standard Linear Clearance design method with clearance length of $3t_p$.

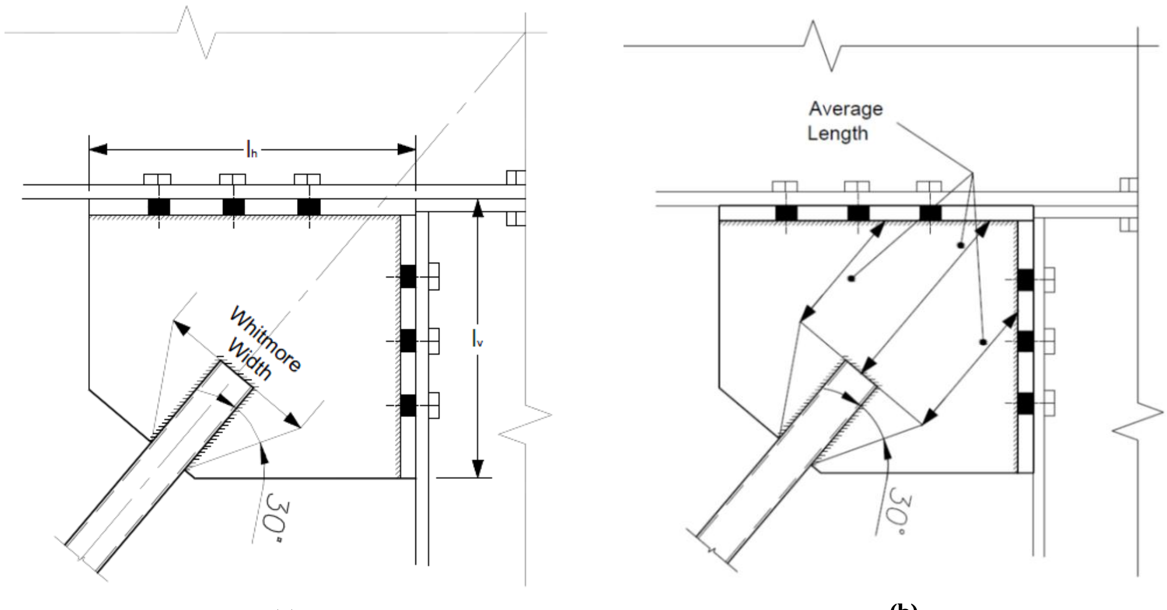


Figure 2.2. Schematic of (a) Whitmore width and (b) Thornton method gusset plate effective length.

3. CYCLIC TESTING SPECIMENS AND PROGRAMME

As a complement to the BRACED shake table tests, the behaviour of two connection types designed using two gusset plate design methods for selected hollow section brace sizes is investigated in a programme of quasi-static cyclic tests at Trinity College Dublin.

The test programme comprises six full scale tests which each feature distinct gusset plate designs, connection constraints and brace section sizes. The brace-gusset plate specimens are installed within a single storey plane frame of overall dimensions similar to the CBF shown in Figure 1.1, creating a single brace CBF structure (Figure 3.1) in which the brace specimen is the only significant source of lateral resistance. Horizontal cyclic displacements of increasing amplitude are applied to this test frame through a 150 kN actuator. The horizontal loading was applied based on the cyclic displacement history guidelines set out by the Recommended Testing Procedure by ECCS (1986). The displacement history was applied at increasing ductility levels based on the initial yielding of the brace cross section. The displacements are increased until the specimen reaches failure by fracture.

As already mentioned, the conventional SLC gusset plate design method can lead to excessively large and stiff gusset plates. An alternative method proposed by Roeder et al. (2006) theorises an elliptical yield line shape occurring in the gusset plate, thus permitting smaller gusset plate dimensions. This elliptical clearance (EC) is shown in Figure 3.2 and is implemented in half of the gusset plate test specimens using a free length of $8t_p$. For the other specimens, the SLC method with a free length of $3t_p$ is used. The properties of all 6 test specimens are presented in Table 3.1.

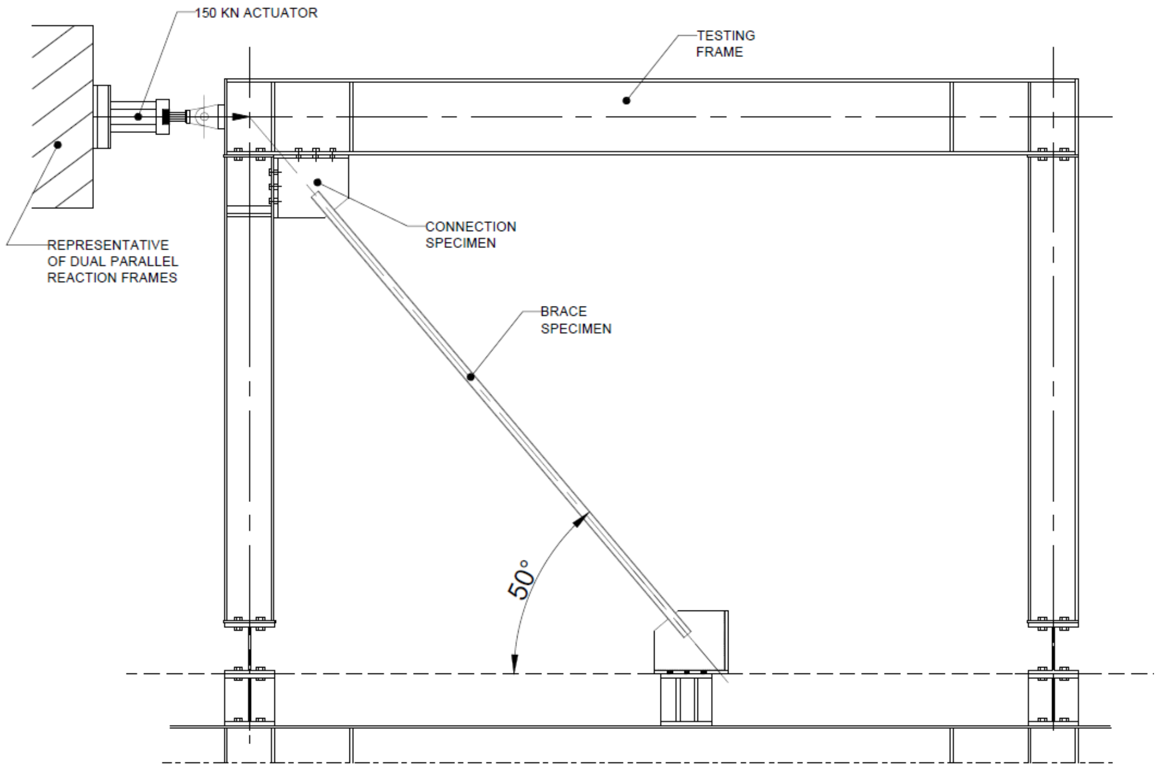


Figure 3.1. Cyclic Test Configuration Setup.

In the SLC specimens, the yield resistance of the connection is designed to be stronger than that of the brace member, leading to a t_p value of 8 mm to be chosen. On the other hand, the EC specimens, are designed with thinner gusset plates ($t_p = 4\,mm$) to achieve a balanced design. Lehman et al. (2008) quantified the benefits of balanced design by comparing the frame drift capacities of test specimens designed with the SLC and EC methods in terms of their primary and secondary yield strength balance ratios. The balance ratio, $\boldsymbol{\beta}$, is calculated as the factored tensile yield strength of the brace divided by the gusset plate yield strength based on the Whitmore width. Smaller drift ranges are exhibited by specimens with smaller $\boldsymbol{\beta}$ ratios. The $\boldsymbol{\beta}$ values for the test specimens in this study are presented in Table 3.1.

The test programme also considers two different types of gusset plate boundary conditions (Figure 3.3). In the first type, the gusset plate is bolted to both the beam and column flanges (CA) effectively restraining the gusset plate on two sides. This type of connection increases the stiffness of the beam-column connection. In the second type, the gusset is connected to the beam flange only (CB), which allows free plate rotation in the out-of-plane direction.

Table 3.1. Schedule of brace and gusset plate specimen details.

Specimen	**Brace Section Size (all SHS)**	**Gusset Boundary Conditions**	**Clearance Model**	**t_p (mm)**	**a (mm)**	**b (mm)**	**Brace Length, L_b (mm)**	**β**
S40-CA-G1	40x40x2.5	CA	Standard ($3t_p$)	8	285	240	2368	0.35
S40-CA-G2	40x40x2.5	CA	Elliptical ($8t_p$)	4	270	230	2503	0.70
S60-CA-G1	60x60x2.5	CA	Standard ($3t_p$)	8	285	240	2368	0.48
S60-CA-G2	60x60x2.5	CA	Elliptical ($8t_p$)	4	270	230	2467	0.96
S40-CB-G1	40x40x2.5	CB	Standard ($3t_p$)	8	265	240	2368	0.35
S40-CB-G2	40x40x2.5	CB	Elliptical ($8t_p$)	4	250	230	2503	0.70

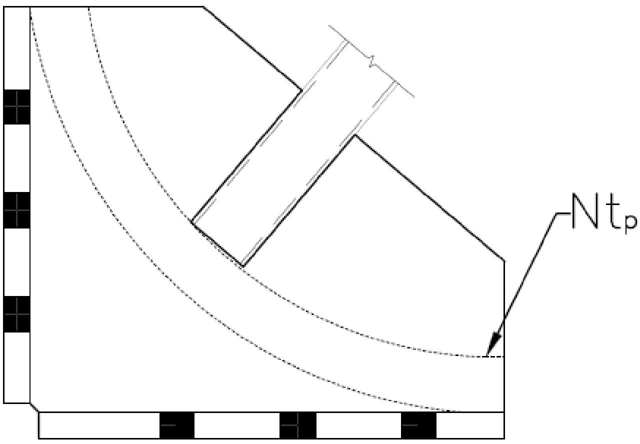
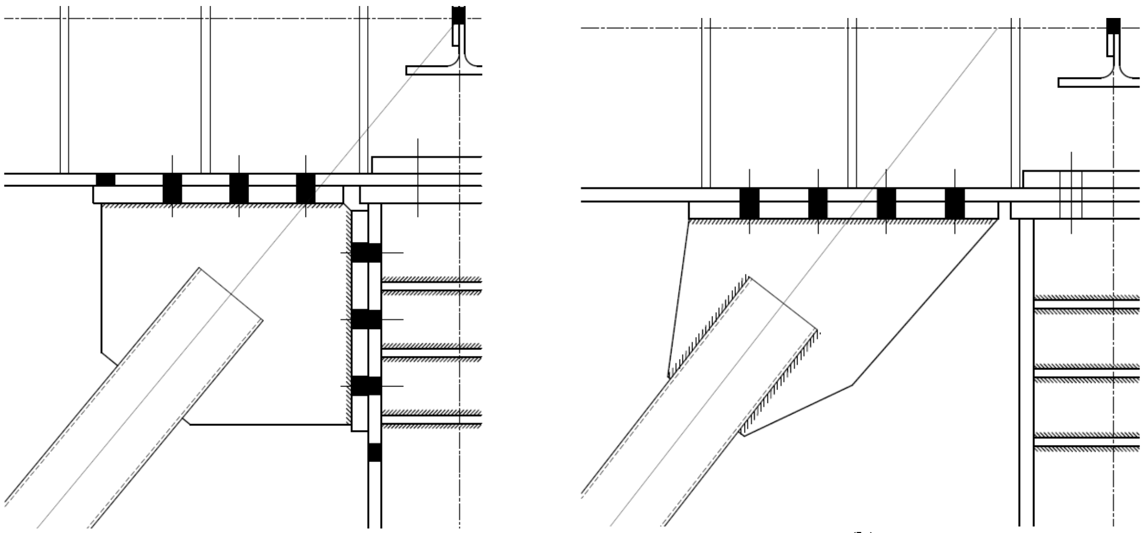


Figure 3.2. Elliptical Clearance design method with a clearance length N times the plate thickness t_p.

3.1 Experimental Response

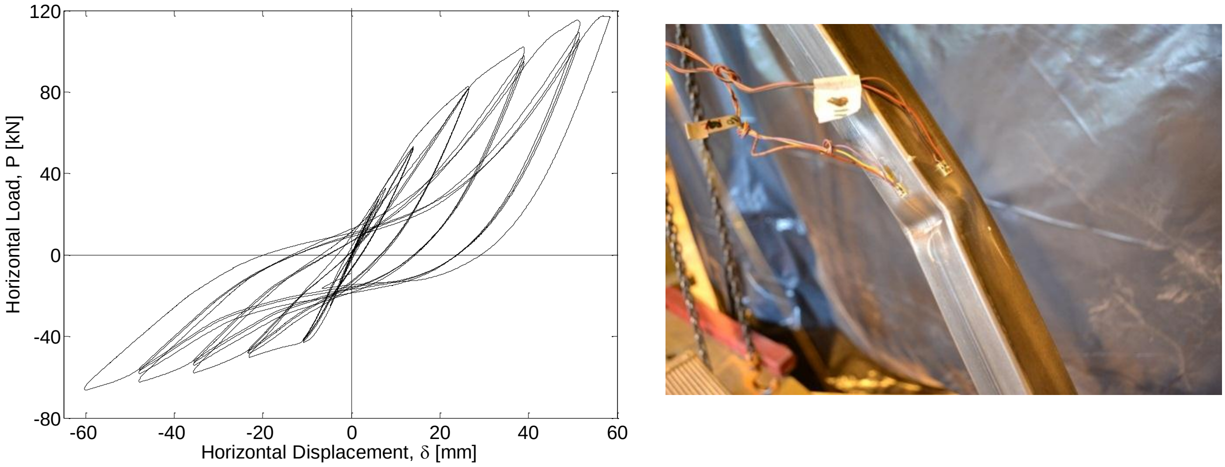
Results from the testing of S40-CA-G1 are presented here. The observed system response in the form of the hysteretic lateral load-displacement data is shown in Figure 3.4(a). In this test, buckling of the brace member occurred at a displacement of approximately 6.1 mm. The weak out-of-plane stiffness of the gusset plates allowed the brace member to form a buckled shape that can be approximated with a half sine curve in the out-of-plane direction with points of inflection located in the free length zones of the gusset plates. This indicates that the effective length can be taken as the full clear length between the brace member termination points in the gusset plates.



(a)

(b)

Figure 3.3. Schematic of (a) connection type CA (b) and connection type CB.



(a)

(b)

Figure 3.4. (a) Cyclic load-displacement history of test S40-CA-G1 (b) Local buckling at brace mid-length.

In the post-buckling range a plastic hinge formed at brace mid-length. The location of this plastic hinge and its direction of buckling concurred with pre-test measurements of the initial brace imperfections. Local buckling occurred at this plastic hinge location in the cycles towards the end of the final $4\delta_y$ cycle (Figure 3.4(b)), and evidence of brace fracture was observed at the beginning of the $10\delta_y$ cycles.

The gusset plates performed as expected with plastic hinges forming along the free length to permit out-of-plane deformation during brace buckling, and elastic gusset plate response during brace tensile yielding. Hinge rotation occurred about an axis that was normal to the axis of the brace member. No gusset plate damage was observed, even at the largest displacement, and no plastic yielding (or local buckling) occurred in the ends of the brace member. All other bolted and welded connections remained elastic throughout the test with the exception of some minor yielding in the connection of the lower end of the brace to the test frame.

4. SHAKE TABLE TEST SIMULATIONS

4.1 Numerical Modelling

Numerical models of the quasi-static and shaking table test frames have been developed in the finite-element framework OpenSees. The nonlinear beam-column element in OpenSees is utilised on account of its ability to accurately represent the behaviour of steel bracing members under earthquake actions (Hunt and Broderick, 2010). The axial force and bending moment interaction is captured through integration of the specified Giuffré-Menegotto-Pinto uniaxial material model (Menegotto and Pinto, 1973) across a fibre-based cross section. Plastic hinge formation is induced at brace mid-length by prescribing an initial geometrical imperfection or 'pre-camber' along the length of the member. There exist some limitations to the developed model, in particular its basis in small deformation theory which, in physical terms, implies that local buckling of braces is not modelled.

4.2 Time-History Tests

Nonlinear pushover analyses of the BRACED shaking table test frame have been carried out to identify the horizontal yield displacement, $\boldsymbol{\Delta_y}$, of the structure. By combining these numerical predictions of yield displacement with ductility capacity values, $\boldsymbol{\mu_\Delta}$, calculated using Eqn. 1.4, an estimate of the ultimate displacement, $\boldsymbol{\Delta_u}$, is obtained. Time-history analyses were carried out with a selection of earthquake ground motion records. These provide an estimate of the shaking table motions required to reach the previously-established values of $\boldsymbol{\Delta_u}$, which may then be employed in shaking table testing to represent the collapse (rather than design) earthquake.

The simulated horizontal roof displacement response of the test frame (with 60x60x3.0 SHS brace members) to the Northridge 1994 record is shown in Figure 4.1. An important feature of this modelling is the need to represent accurately the inelastic compression-tension response of the gusset plate connections. Inaccurate or simplified modelling of these components lead to errors in the calculated strain response in the brace member and, in turn, misrepresentations of brace ductility demand.

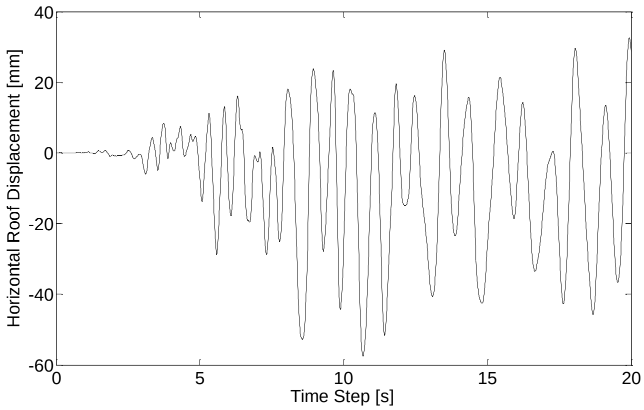


Figure 4.1. Horizontal roof displacement of the 60x60x3.0 SHS specimen.

5. CONCLUSIONS

This paper addresses the evolving seismic design methodology of CBFs employing for gusset plate connections. A controlling feature in the conventional design of these elements is provision for out-of-plane deformation of a buckled brace member through the specification of a free length within the gusset plate to allow plastic hinge formation. However, alternative methods have been proposed to allow for a balanced design approach that offers improved ductility capacity and more reliable system behaviour. An integrated programme of quasi-static testing, shake table testing and numerical simulation that investigates the application of these proposals to European design practice has been described. Various combinations of brace member section size and gusset plate designs are investigated. As well as allowing the physical behaviour and seismic performance of these combinations to be investigated, the quasi-cyclic and shaking table tests provide data for the important validation for numerical models. Successful modelling using the OpenSees computational framework must capture the asymmetric inelastic cyclic response of the gusset plate connections as well as the brace members themselves. In particular, modelling of the response of the brace-connection assembly must be able to represent the different behaviours displayed by the conventional strong gusset-weak brace approach and the balanced brace-gusset strength more recently proposed.

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