UNIVERSITY OF LEEDS

This is a repository copy of *Finite element modelling of steel connections with web openings: aseismic design and progressive collapse*.

White Rose Research Online URL for this paper:
http://eprints.whiterose.ac.uk/80991/

Version: Accepted Version

**Proceedings Paper:**

Tsavdaridis, KD and Pilbin, C (2014) Finite element modelling of steel connections with web openings: aseismic design and progressive collapse. In: The 7th European Conference on Steel and Composite Structures. The 7th European Conference on Steel and Composite Structures, 10-12 September 2014, Napoli, Italy. Eurosteel 2014 . (Unpublished)

**Reuse**

Unless indicated otherwise, fulltext items are protected by copyright with all rights reserved. The copyright exception in section 29 of the Copyright, Designs and Patents Act 1988 allows the making of a single copy solely for the purpose of non-commercial research or private study within the limits of fair dealing. The publisher or other rights-holder may allow further reproduction and re-use of this version - refer to the White Rose Research Online record for this item. Where records identify the publisher as the copyright holder, users can verify any specific terms of use on the publisher’s website.

**Takedown**

If you consider content in White Rose Research Online to be in breach of UK law, please notify us by emailing eprints@whiterose.ac.uk including the URL of the record and the reason for the withdrawal request.

White Rose university consortium
Universities of Leeds, Sheffield & York

eprints@whiterose.ac.uk
https://eprints.whiterose.ac.uk/

# FE MODELLING OF STEEL CONNECTIONS WITH WEB OPENINGS

## Aseismic Design and Avoidance of Progressive Collapse

Konstantinos Daniel Tsavdaridis and Christopher Pilbin

The University of Leeds, School of Civil Engineering, Leeds, UK
k.tsavdaridis@leeds.ac.uk, chris.pilbin@hotmail.co.uk

## INTRODUCTION

Events of extreme loading on structures such as the 1994 Northridge, California earthquake and the 2001 attack on the World Trade Centres, New York have not only lead to loss of life but highlighted structural deficiencies and lack of existing knowledge of the progressive collapse behaviour. As a result, Governmental bodies such as the GSA [1] and DoD [2] have established design codes and guidelines to reduce the possibility of progressive collapse through the promotion of redundancy, ductility and continuity in structural systems. The design codes do not provide guidelines for the complete prevention of progressive collapse but rather mitigate a 'disproportionate' area of collapse. In collapse scenarios, following the removal of a load bearing column, the ability for the beams and columns to develop catenary action is essential for the structure to resist further collapse. Beam-to-column connections should provide sufficient ductility to sustain large rotations and allow structural members to carry loads in tension.

Researchers have suggested that seismically designed structures are more resistant to progressive collapse conditions [3,4,5]. By designing seismically resistant structural elements, such as the connections, the structure's ductility, robustness and ability to perform catenary action is also enhanced. Steel structures with complete lateral force-resisting systems capable of resisting wind and seismic loads specified by building codes are able to resist credible blast loads without lateral instability and collapse. However, explosive charges detonated in close proximity to structural elements can cause extreme local damage, including complete loss of loading carrying capacities in individual columns, beams, girders and slabs [6]. GSA developed simplified guidelines for the design of such systems. These guidelines specify that certain elements of the frame be proportioned with sufficient strength to resist twice the dead and live load anticipated to be present, without exceeding inelastic demand ratios, based on theory related to the instantaneous application load on an elastic element. Under general progressive collapse design guidelines, structural members are permitted to experience flexural inelasticity based on allowable values proposed in seismic guidelines as the amplified loading occurs for a very short period, and the long-term loading following removal is a static condition. This is the reason that most of the recent studies found in the literature employ the non-linear static (push-down) analysis. However, GSA and DoD guidelines do not require the evaluation of the plastic strength of the frame supporting the weight of the structure in a static condition, while they should.

In order to enhance the ductility of post Northridge connections, researchers suggested the weakening of the beam to shift the stresses away from the column's face. One method to achieve this is by removing portions of the flanges or the web weakening the beam locally, referred to as Reduced Beam Section (RBS) and Reduced Web Section (RWS), respectively. From another perspective, perforations in the webs of steel beams are widely used nowadays in building construction due to their ability to provide lighter structural members, reduced material costs, in addition to the provision for greater flexibility in structural layouts particularly in the floor-to-ceiling height. In order to enhance the structural behaviour of perforated beams, Tsavdaridis and D'Mello [7] developed novel web opening shapes which were found to be more effective than the standard ones (i.e. circular and hexagonal) in terms of beams' stress distribution and deflections experienced. Moreover, the potential of beams with isolated openings to enhance the cyclic (hysteretic) behaviour of steel connections has been recently studied by researchers [8,9,10].

However, the behaviour of such connections under progressive collapse condition has not been studied yet.

# 1 VALIDATION OF THE FE MODEL

## 1.1 Introduction

In order to accurately model the behaviour of a beam-to-column connection, a validation study was initially performed. A three-dimensional (3D) FE model was developed using shell elements in ABAQUS v6.10 and compared against both the experimental (*Fig.* 1) and the FE model found by Kim et al. [11]. The modelled connection was a welded unreinforced flange-bolted web (WUF-B) single sided moment connection, often referred to as a Pre-Northridge connection. This type of connections was found to be prone to brittle failure of the welded flanges during an earthquake. The 3D FE model was implemented with conventional shell elements S4R; a 4-node reduced integration element with displacement and rotational degrees of freedom. Five M24 fasteners were used to connect the shear tab to the web of the beam and they are modelled as rigid 'beam elements' with a physical radius of 12mm simulating the link projected between the two surfaces.

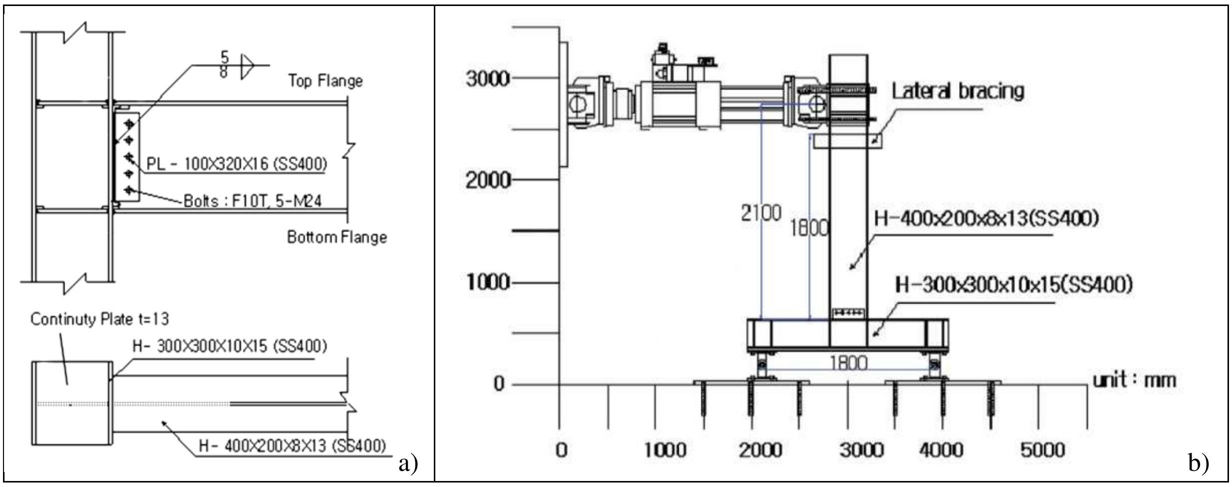


*Fig. 1.* a) Connection properties; b) Experimental setup [11]

## 1.2 Material properties

A bi-linear stress-strain relationship was used to model the non-linear behaviour of the assembly. For the linear elastic response of the beam and column a Young's modulus, *E=190GPA* and Poisson's ratio, *v=0.3* was used with the Von Mises yield criterion employed. The plastic response of the assembly components were taken from the coupon test measurements for the yield and ultimate strengths, as presented by Kim et al. [11] (Table 1). Kinematic hardening material model was assumed for the assembly under cyclic loading.

*Table 1.* Assembly coupon material properties [11]

| Member | Component | Yield Strength, $f_y$ [MPa] | Ultimate Strength, $f_u$ [MPa] |
|---|---|---|---|
| Beam | Flange | 281 | 423 |
| | Web | 332 | 438 |
| Column | Flange | 281 | 433 |
| | Web | 304 | 450 |

## 1.3 Loading sequence and analysis

The assembly was examined under cyclic loading using the SAC loading protocol outlined by FEMA-350 [12], and beam-end displacements for 32 cycles. A comparison between the experimental and FEM behaviour of the connection model is illustrated in *Fig. 2(a)*. The maximum normalised moment found from the experimental data and the FE analysis was 1.070 and 1.096, respectively. There is a good correlation between the results illustrated in the graphs with FE model presenting slightly stiffer response when compared to the experimental model. In more detail, the hysteretic behaviour of the FE model can be seen as similar compared to the experimental performance with the cycles overlapping until the penultimate load cycle 31, when failure occurs in the top and bottom welds of the experimental model. Fracture of the material was not incorporated into the FE model. Also, the bolts in the FE model are modelled as "perfect" connectors; hence they strengthen the shear capacity of the connection and ignore any slippage which may occur under large rotations.

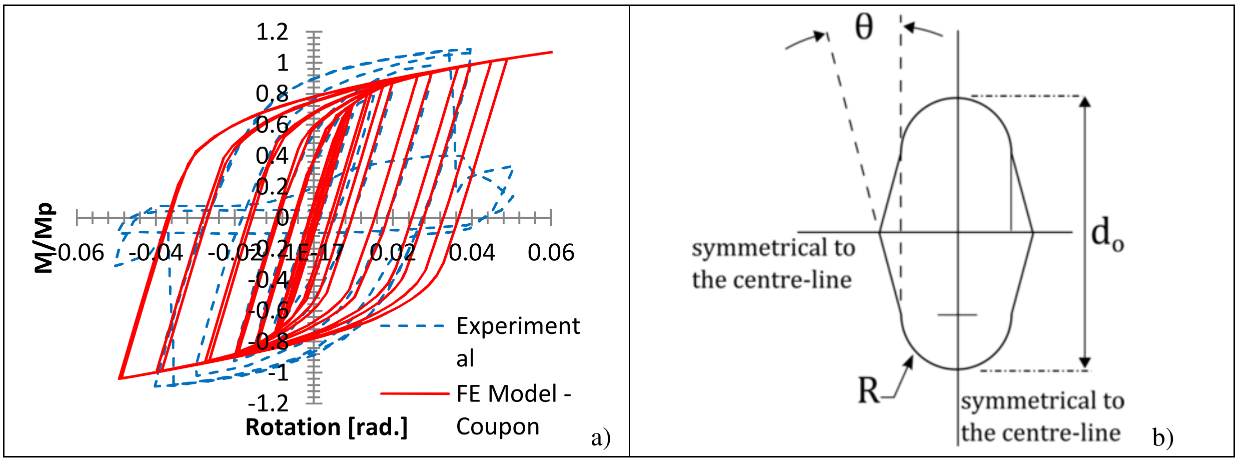


*Fig. 2.* a) Hysteretic behaviour; b) Geometric parameters of novel web openings

## 2 PARAMETRIC STUDY

### 2.1 Introduction

The ability of two different isolated novel web openings to enhance the behaviour of a pre-Northridge WUF-B connection under progressive collapse is investigated herein. These elliptically-based web openings were first introduced by Tsavdaridis and D'Mello [13] and they are incorporating straight lines to ease fabrication and curvy lines to avoid stress concentration (*Fig. 2(b)*). For the sake of comparison, a traditional circular opening has also been analysed to observe the effectiveness of the novel openings on steel beams and connections.

### 2.2 Parameters

The geometric parameters which define these novel opening shapes are the opening depth, $d_o$, the angle of the straight edge lines, θ and the semi-circle radius, R. Another connection type with similar perforated beams was introduced by Tsavdaridis et al. [10], while examining the effect of the opening depth, $d_o$, and the distance from the column's face to the web opening centre line, S, on connections' performance. Three varying values were selected for each case and they are dependent on the total beam height, h, of the connection: $d_o = 0.5h; 0.65h; 0.8h$ and $S = 0.87h; 1.3h; 1.7h$. The name of the specimens are given in the following manner: a connection with a RWS containing a novel web opening 'A' with a depth of 0.65h (i.e. 260mm), located 700mm from the column face would correspond to the classification: A2-700. In total, 27 connection specimens with varying geometric characteristics are examined herein. The material properties for all of the connection components are changed to steel grade of S500 to provide material uniformity.

## 3 CYCLIC LOADING

### 3.1 Results

A summary of the results is presented in *Table 2*. The transfer of shear forces in the vicinity of the web opening induces local bending moments, known as the Vierendeel mechanism. *Fig. 3* shows the development of the four local plastic hinges, for the same opening size ($d_o$=$0.8h$) at different locations along the beam. Connections with small openings are unable to develop plastic hinges as the effective depth of the web is still large enough to resist the global vertical shear stresses. As a result, higher stresses are developed in the panel zone and in the shear tab as the beam rotates.

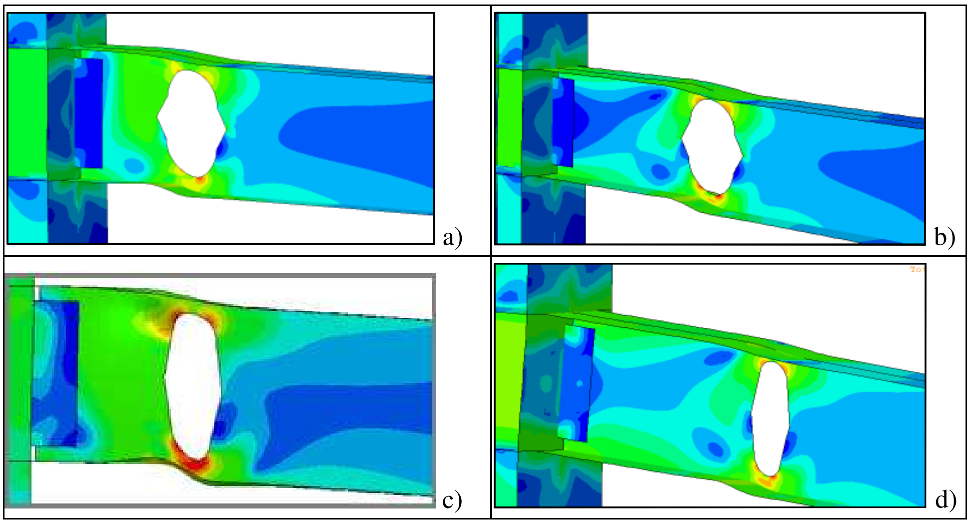


*Fig. 3.* Von Mises stress distribution and Vierendeel mechanism a) A3-350; b) A3-700; c) B3-350; d) B3-700

Openings of size $d_o$ equal to 0.8h found to be the most effective in reducing the stress on the top flange weld. When the web opening is placed closer to column's face (*S=350mm*), high deflections are observed at the top flange and the tee-section of the beam near the opening. When large web openings placed further away (*S=700mm*), four plastic hinges are formed again, but a very small deformation is noted in the flanges, resulting in a greater stressed area in the top flange weld of the connection. On the other hand, connections using small web openings do not influence the stress state of the flange weld. In fact, high stresses are developed on the top flange in a similar fashion to the solid beam. This suggests that connections with smaller openings would fail via brittle facture as experienced when the solid beams are used. Overall, connections with smaller web openings are not influenced by the change in distance from the face of the column, S. Furthermore, no differences between opening types A and B are acquired.

### 3.2 Connection strength

The connection specimen with the solid beam managed to carry the maximum moment of 391.7kNm. Specimens with small web openings carried high loads while the increase in the opening depth result the decrease of the ultimate moment. Also, the connections with novel opening type B are the strongest, whereas connections with traditional circular (C) openings are the weakest. It is also found that the wider the critical opening length, the lower is the cyclic degradation that the connection experiences due to the Vierendeel mechanism. Connections using opening type B are characterised as more controllable, as a linear behaviour was observed when changing geometric parameters.

### 3.3 Rotational capacity

All connection specimens illustrated very small variations in ultimate rotation capacity with the largest difference presented from specimens using openings C3. The specimens with novel openings are able to achieve greater ultimate rotation in comparison to the specimens with traditional circular openings. In addition, the use of the largest circular opening proved very detrimental to the rotation ability as a result of the Vierendeel mechanism formation since the early loading cycles. The ultimate rotations recorded are higher, mainly under monotonic loading, than the DoD limit for low and high level of protection (0.047 and 0.035, respectively) and the GSA limit (0.035). It is worth to note that DoD [2] is based on design codes and guidelines, in particular, those by GSA (2008), ASCE 41 (2007) and DoD (2005). In DoD it is stated that "the acceptance criteria in ACSE 41 are based upon cyclic loadings with bending moment only, and rotational capacities are often limited because of degradation and premature loss of strength". This limitation can be demonstrated as prejudicial in the design of avoiding progressive collapse.

*Table 2.* Summary of results for various connections with isolated openings

| Connection type | Connection (Cyclic) Ultimate moment, $M_u$ [kNm] | Ultimate rotation, $\varphi_u$ [rad.] | Panel Zone (Monotonic) Rotation at maximum panel zone deflection [rad.] | Maximum panel zone deflection [mm] | Web Opening Area [$mm^2$] |
|---|---|---|---|---|---|
| Solid | 391.7 | 0.0500 | 0.2320 | 20.17 | N.A. |
| A1-350 | 390.4 | 0.0500 | 0.1260 | 14.58 | 22758 |
| A1-520 | 390.5 | 0.0500 | 0.1850 | 17.57 | 22758 |
| A1-700 | 390.7 | 0.0500 | 0.2170 | 18.44 | 22758 |
| A2-350 | 385.5 | 0.0497 | 0.0790 | 9.16 | 38457 |
| A2-520 | 389.7 | 0.0500 | 0.0960 | 12.36 | 38457 |
| A2-700 | 389.5 | 0.0500 | 0.1180 | 13.96 | 38457 |
| A3-350 | 331.7 | 0.0499 | 0.0240 | 1.70 | 58259 |
| A3-520 | 320.8 | 0.0500 | 0.0290 | 2.41 | 58259 |
| A3-700 | 335.8 | 0.0500 | 0.0620 | 3.34 | 58259 |
| B1-350 | 390.6 | 0.0499 | 0.1200 | 15.23 | 12949 |
| B1-520 | 390.5 | 0.0500 | 0.1980 | 17.40 | 12949 |
| B1-700 | 390.4 | 0.0500 | 0.2190 | 18.50 | 12949 |
| B2-350 | 389.2 | 0.0500 | 0.0970 | 11.27 | 21895 |
| B2-520 | 390.1 | 0.0499 | 0.1080 | 14.34 | 21895 |
| B2-700 | 390.0 | 0.0498 | 0.1370 | 17.82 | 21895 |
| B3-350 | 347.4 | 0.0500 | 0.0390 | 4.17 | 35062 |
| B3-520 | 371.5 | 0.0498 | 0.0560 | 6.05 | 35062 |
| B3-700 | 384.9 | 0.0500 | 0.0740 | 9.48 | 35062 |
| C1-350 | 382.0 | 0.0487 | 0.1110 | 12.99 | 31410 |
| C1-520 | 382.7 | 0.0493 | 0.1699 | 16.89 | 31410 |
| C1-700 | 389.7 | 0.0498 | 0.1466 | 19.99 | 31410 |
| C2-350 | 369.5 | 0.0495 | 0.0577 | 6.41 | 53083 |
| C2-520 | 380.9 | 0.0498 | 0.0710 | 9.16 | 53083 |
| C2-700 | 384.8 | 0.0479 | 0.1032 | 13.03 | 53083 |
| C3-350 | 260.8 | 0.0186 | 0.0192 | 0.77 | 80410 |
| C3-520 | 275.5 | 0.0198 | 0.0220 | 1.00 | 80410 |
| C3-700 | 295.1 | 0.0249 | 0.0257 | 1.49 | 80410 |

## 3.4 Panel zone

The shear deformation of the panel zone is recorded under monotonic loading in order to determine the effect of the presence of an opening. The energy dissipated in the connection is absorbed in the panel zone region until the maximum moment is achieved and before any local buckling occurs in the vicinity of the web opening of the flanges of the beam. From the results it is shown that connections with openings are effectively reducing the shear deformation of the panel zone, due to the formation of plastic hinges in the beam. In particular, specimens with novel opening A are the most effective in reducing the shear deformation, oppositely to specimens with circular opening C. For instance, when S is equal to 350mm, the increase in the web opening area is almost directly proportional to the reduction of the deformation of the panel zone for all opening types.

## 4 CONCLUDING REMARKS AND DISCUSSION

The study presented examined the behaviour of isolated web openings in enhancing the ability of a pre-Northridge connection under progressive collapse conditions. From the study, the following main conclusions are drawn:

- Introduction of the web openings resulted in reduction of strength of the connection in relation to the solid beam connection. When small web openings were used, there was not any effect.
- Specimens with novel openings, compared to the traditional circular openings, were found to be stronger and attain a higher ultimate rotation for the same opening depth.
- The use of large openings was found to be the most effective at moving the plastic region away from the column's face and reducing the shear zone panel deflection.
- It can be verified that the cyclic limits given by GSA and DoD may underestimate the demand of rotational capacity and strength, while they can be achieved by the novel connections.
- Isolated large web openings can prevent excessive shear deformation as well as reduce the stress intensity in the vicinity of the beam-to-column weld in contrast to current reinforcing methods such as using stiffeners and double plates. Consequently, the use of certain novel web openings serve as a cheaper and more feasible method of retrofitting existing structures.

As it is known from the literature, it is not necessary to provide substantial flexural capacity in the horizontal framing to provide collapse resistance, however, it can be provided as long as connections with sufficient tensile capacity to develop catenary behaviour are designed. This concept is now promoted using perforated beams at beam-to-column connections, as the rotational capacity is increased and the connection components are lightly stressed. Hamburger and Whittaker [6] suggests that the mobilization of catenary action in framing could require plastic rotations on the order of 0.07 radians or more. The connections developed in the present study meet this criterion. Earthquake demands are cyclic and induce low-cycle fatigue failure of connections whereas, for instance, air-blast loadings can produce high strain rates of larger magnitudes and will occur simultaneously with large axial tension demands. However, under conditions of high strain rate, steel framing becomes stronger; hence not requirement of extra strength is necessary, but also becomes more brittle; something which is dealt very well with the new proposed RWS connection type while developing large inelastic rotations, ductility and tensile strains through large deformation behaviour.

## REFERENCES


[1] General Services Administration (GSA). 2003. Progressive collapse analysis and design guidelines for *The U.S. General Services Administraion*. Washing., DC.

[2] Department of Defense (DoD). 2009. Design of Buildings to Resist Progressive Collapse, *U.S. Army Corps of Engineering,* Washington, DC.

[3] Carino, N. and Lew, H.S. 2001. *Proc., National Institute of Technology and Standards*, Washington, D.C., No. NISTIR 6831.

[4] Khandelwal, K. and El-Tawil, S. 2007. Collapse Behavior of Steel Special Moment Resisting Frame Connections. *J. Struct. Eng.*,133(5), 646–655.

[5] Kim, T. and Kim, J. 2009. Collapse analysis of steel moment frames with various seismic connections. *J. of Constr. Steel Res.,* 65(6), 1316-1322.

[6] Hamburger, R. and Whittaker, A. 2004. Design of Blast-Related Progressive Collapse Resistance. *North American Steel Constr. Conf.*

[7] Tsavdaridis, K.D. and D'Mello, C. 2012. Optimisation of Novel Elliptically-Based Web Opening Shapes of Perforated Steel Beams. *J. of Constr. Steel Res.*, 76(0), 39-53

[8] Hedayat, A.A. and Celikag, M. 2009. Post-Northridge connection with modified beam end configuration to enhance strength and ductility. *J. of Constr. Steel Res.,* 65(0), 1413-1430.

[9] Qingshan Y., Bo, L. and Na, Y. 2009. Aseismic behaviors of steel moment resisting frames with opening in beam web. *J. of Constr. Steel Res.,* 65(6), 1323-1336.

[10] Tsavdaridis, K.D. Faghih, F. and Nikitas, N. 2014. Forthcoming. Assessment of Perforated Steel Beam-to-Column Connections Subjected to Cyclic Loading. *J. of Earthquake Eng.* In Press.

[11] Kim, T., Kim, U.S., and Kim, J. 2012. Collapse resistance of unreinforced steel moment connections. *The Struct. Design of Tall and Special Build.,* 21(10),724-735.

[12] Federal Emergancy Management Agency (FEMA) 350. 2000. Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings, Prepared by the SAC Joint Venture for FEMA. Washington, DC.

[13] Tsavdaridis, K.D. and D'Mello, C. 2011. Web Buckling Study of the Behaviour and Strength of Perforated Steel Beams with Different Novel Opening Shapes. *J. of Constr. Steel Res.* 67(10),1605-1620

# FE MODELLING OF STEEL CONNECTIONS WITH WEB OPENINGS
## Aseismic Design and Avoidance of Progressive Collapse

Konstantinos Daniel Tsavdaridis and Christopher Pilbin

The University of Leeds, School of Civil Engineering, Leeds, UK
k.tsavdaridis@leeds.ac.uk, chris.pilbin@hotmail.co.uk

**KEYWORDS:** WUF-B connections; perforated beams; cyclic loading; progressive collapse; FEA.

## ABSTRACT

Events of extreme loading on structures such as the 1994 Northridge, California earthquake and the 2001 attack on the World Trade Centres, New York have not only lead to loss of life but highlighted structural deficiencies and lack of existing knowledge of the progressive collapse behaviour. As a result, Governmental bodies such as the GSA [1] and DoD [2] have established design codes and guidelines to reduce the possibility of progressive collapse through the promotion of redundancy, ductility and continuity in structural systems. The design codes do not provide guidelines for the complete prevention of progressive collapse but rather mitigate a 'disproportionate' area of collapse. In collapse scenarios, following the removal of a load bearing column, the ability for the beams and columns to develop catenary action is essential for the structure to resist further collapse. Beam-to-column connections should provide sufficient ductility to sustain large rotations and allow structural members to carry loads in tension.

Researchers have suggested that seismically designed structures are more resistant to progressive collapse conditions [3,4,5]. By designing seismically resistant structural elements, such as the connections, the structure's ductility, robustness and ability to perform catenary action is also enhanced. Steel structures with complete lateral force-resisting systems capable of resisting wind and seismic loads specified by building codes are able to resist credible blast loads without lateral instability and collapse. However, explosive charges detonated in close proximity to structural elements can cause extreme local damage, including complete loss of loading carrying capacities in individual columns, beams, girders and slabs [6]. GSA developed simplified guidelines for the design of such systems. These guidelines specify that certain elements of the frame be proportioned with sufficient strength to resist twice the dead and live load anticipated to be present, without exceeding inelastic demand ratios, based on theory related to the instantaneous application load on an elastic element. Under general progressive collapse design guidelines, structural members are permitted to experience flexural inelasticity based on allowable values proposed in seismic guidelines as the amplified loading occurs for a very short period, and the long-term loading following removal is a static condition. This is the reason that most of the recent studies found in the literature employ the non-linear static (push-down) analysis. However, GSA and DoD guidelines do not require the evaluation of the plastic strength of the frame supporting the weight of the structure in a static condition, while they should.

In order to enhance the ductility of post Northridge connections, researchers suggested the weakening of the beam to shift the stresses away from the column's face. One method to achieve this is by removing portions of the flanges or the web weakening the beam locally, referred to as Reduced Beam Section (RBS) and Reduced Web Section (RWS), respectively. From another perspective, perforations in the webs of steel beams are widely used nowadays in building construction due to their ability to provide lighter structural members, reduced material costs, in addition to the provision for greater flexibility in structural layouts particularly in the floor-to-ceiling height. In order to enhance the structural behaviour of perforated beams, Tsavdaridis and D'Mello [7] developed novel web opening shapes which were found to be more effective than the

standard ones (i.e. circular and hexagonal) in terms of stress distribution and deflections experienced. The potential of beams with isolated openings to enhance the cyclic (hysteretic) behaviour of steel connections has been recently studied by researchers [8,9,10]. However, the behaviour of such connections under progressive collapse condition has not been studied yet.

## CONCLUSIONS

This study presented examines the behaviour of isolated web openings in enhancing the ability of a WUF-B connection under progressive collapse conditions and the main conclusions are drawn:

- Introduction of the web openings resulted in reduction of strength of the connection in relation to the solid beam connection. When small web openings were used, there was not any effect.
- Specimens with novel openings, compared to the traditional circular openings, were found to be stronger and attain a higher ultimate rotation for the same opening depth.
- The use of large openings was found to be the most effective at moving the plastic region away from the column's face and reducing the shear zone panel deflection.
- It can be verified that the cyclic limits given by GSA and DoD may underestimate the demand of rotational capacity and strength, while they can be achieved by the novel connections.
- Isolated large web openings can prevent excessive shear deformation as well as reduce the stress intensity in the vicinity of the beam-to-column weld in contrast to current reinforcing methods such as using stiffeners and double plates. Consequently, the use of certain novel web openings serve as a cheaper and more feasible method of retrofitting existing structures.

As it is known from the literature, it is not necessary to provide substantial flexural capacity in the horizontal framing to provide collapse resistance, however, it can be provided as long as connections with sufficient tensile capacity to develop catenary behaviour are designed. This concept is now promoted using perforated beams at beam-to-column connections, as the rotational capacity is increased and the connection components are lightly stressed. Hamburger and Whittaker [6] suggests that the mobilization of catenary action in framing could require plastic rotations on the order of 0.07 radians or more. The connections developed in the present study meet this criterion. Earthquake demands are cyclic and induce low-cycle fatigue failure of connections whereas, for instance, air-blast loadings can produce high strain rates of larger magnitudes and will occur simultaneously with large axial tension demands. However, under conditions of high strain rate, steel framing becomes stronger; hence not requirement of extra strength is necessary, but also becomes more brittle; something which is dealt very well with the new proposed RWS connection type while developing large inelastic rotations, ductility and tensile strains through large deformation behaviour.

## REFERENCES


[1] General Services Administration (GSA). 2003. Progressive collapse analysis and design guidelines for *The U.S. General Services Administraion*. Washing., DC.

[2] Department of Defense (DoD). 2009. Design of Buildings to Resist Progressive Collapse*, U.S. Army Corps of Engineering,* Washington, DC.

[3] Carino, N. and Lew, H.S. 2001. *Proc., National Institute of Technology and Standards*, Washington, D.C., No. NISTIR 6831.

[4] Khandelwal, K. and El-Tawil, S. 2007. Collapse Behavior of Steel Special Moment Resisting Frame Connections. *J. Struct. Eng.*,133(5), 646–655.

[5] Kim, T. and Kim, J. 2009. Collapse analysis of steel moment frames with various seismic connections. *J. of Constr. Steel Res.,* 65(6), 1316-1322.

[6] Hamburger, R. and Whittaker, A. 2004. Design of Blast-Related Progressive Collapse Resistance. *North American Steel Const. Conf.*

[7] Tsavdaridis, K.D. and D'Mello, C. 2012. Optimisation of Novel Elliptically-Based Web Opening Shapes of Perforated Steel Beams. *J. of Constr. Steel Res.*, 76(0), 39-53

[8] Hedayat, A.A. and Celikag, M. 2009. Post-Northridge connection with modified beam end configuration to enhance strength and ductility. *J. of Constr. Steel Res.,* 65(0), 1413-1430.

[9] Qingshan Y., Bo, L., and Na, Y. 2009. Aseismic behaviors of steel moment resisting frames with opening in beam web. *J. of Constr. Steel Res.,* 65(6), 1323-1336.

[10] Tsavdaridis, K.D. Faghih, F. and Nikitas, N. 2014. Forthcoming. Assessment of Perforated Steel Beam-to-Column Connections Subjected to Cyclic Loading. *J. of Earthquake Eng.* In Press.