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EXTENDED END-PLATE RWS CONNECTIONS WITH PERFORATED BEAMS UNDER CYCLIC LADING

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Abstract:

Researchers have developed seismic-resistant connections specially designed for moment frames through reducing the beam strength at a location adjacent to the beam-column connection to develop the so-called "weak beam - strong column" mechanism. The recently introduced reduced web section (RWS) connections is another type of such connections designed by weakening the beam section through large web openings similar to those found in perforated (cellular) beams. Up to date, various types of RWS connections have been studied through finite element (FE) analyses, yet no experimental works have been carried out. This paper presents the first three types of RWS connections with the aim to determine the influence of the web opening position and spacing on the cyclic behaviour of RWS connections. The four tested RWS specimens sustained throughout the entire FEMA 350 cyclic loading test except for one specimen which failed by weld cracking - similar to the failure of connections in the 1994 Northridge earthquake. Findings of this paper can be used to validate FE studies and to be the basis of design guidelines for pre-engineered RWS connections.

Introduction

Historically, extensive studies had been carried out for the Reduced Beam Section (RBS) since the 1994 Northridge earthquake. The concept of RBS connection is to create a "weakened" area in the beam at a short distance away from the column flange. The main failure of steel structures in the 1994 Northridge earthquake was due to the stress concentration in the welded connection and hence the fracture of welds. The RBS connection was developed to increase the beam's elasticity in comparison to the CJP weld's low inelasticity and shift the stress concentration from column face to the "weakened" area in order to achieve the "weak beam - strong column" mechanism. The RBS connection was created by cutting the top and bottom flange in the designated "weakened" area. However, it was reported by researchers that reducing the cross-sectional area of the compressive flange, buckling is more likely to occur. Therefore, to eliminate the instability of RBS connection, a plethora of new seismic connections are proposed in recent years.

Reduced Web Sections (RWS)

The Reduced Web Section (RWS) connection is a new type of seismic-resistant connection developed recently by Tsavdaridis et al. The idea of RWS connection is to create a weakened zone in the beam without significant reduction in the moment capacity and hence decrease the chances of web buckling. Due to the presence of multiple web openings along the beam, the RWS connection performs more elastically which is beneficial to the performance of steel frames under seismic actions.

Simultaneously, perforated (aka cellular) steel beams are getting popular in the industry due to their lightweight mature and their capacity to span much longer than the traditional solid webbed beams. Also, lightweight structures are preferred options in seismic regions as they experience lower seismic forces when comparing to the heavy concrete structures. Therefore, perforated beams had been employed to develop a series of RWS connections with various configurations.

Although perforated beams are more susceptible to LTB when compared to the traditional solid beams, they tend to fail first under local failure modes appear in the perforated webs through the Vierendeel mechanism and/or the web-post buckling mechanism [Tsavdaridis and D'Mello, 2012].

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To understand the the Vierendeel mechanism, the factors that affect the moment capacity of the Tees and the formation of plastic hinges had to be understood. [Tsavdaridis and D'Mello, 2012] comprehensively studied the Vierendeel mechanism and performed FE analyses on perforated beams with single and multiple web openings. It was discovered that the first plastic hinge often forms at the low moment side (LMS) with an angle φ_p taken from the center line of the opening. The load is then slowly redistributed across the opening and other plastic hinges are formed. At this time, the angle φ_p began to increase and the stress in steel was transformed from elastic state to plastic state. When the locations of plastic hinges are known, the critical opening length can be determined. The angle φ_p of cellular (circular) web openings is around 23^{o} when under high shear. [Yang et al., 2009] performed a test on perforated beam and came up with a similar result; the angle φ_p was approximately 24^{o}.

Vierendeel mechanism

Vierendeel mechanism failure is one of the most critical failure modes that are considered in the design of perforated beams. It often happens due to the high shear forces in the vicinity of large web openings, especially after the formation of the four plastic hinges at the "corners" of the web opening.

[Tsavdaridis and D'Mello, 2012] carried out a FE analysis study of perforated beams and concluded that the top and bottom Tee-sections of a web opening carries the applied shear, primary and secondary moments. It was mentioned that the primary moment is the traditional moment results from bending along the beams, while the secondary moment (Vierendeel moment) results from the bending action in the Tees. As a result of this, the length (i.e., critical opening length) of the Tee-section determines the secondary moment. For example, when the Tee-section (web opening) is longer, the secondary moment is therefore greater.

Recently, [Erfani and Akrami, 2016] carried out a numerical study on welded RWS connections and examined the effect of different parameters on their seismic behaviour. FE analyses were conducted on three different series of web openings. Based on the results, it was discovered that the stress concentration increased at the top and bottom Tee-sections when the critical opening length increases, as it can be seen from Figure 1. For web openings with adequate opening length or depth, four plastic hinges were formed at the corners. This, in fact, indicated that the geometrical properties of the web openings have significant influence on perforated beam's critical failure modes, as stated before by [Tsavdaridis and D'Mello, 2011a], and hence the connection's moment capacity.

Web-post buckling

Web-post buckling is another critical failure mode in perforated beams, especially for those with closely spaced web openings. [Tsavdaridis et al., 2015] mentioned that the failure mechanism depends on different parameters, such as web opening spacing and effectively web-post dimensions, additional vertical deflections, and out-of-plane displacement. To verify the possibility of multiple buckling mode shapes, [Tsavdaridis and D'Mello, 2011b] performed FE analyses and extracted four buckling modes. The FE results were then compared against the experimental tests. Overall, the predicted failure buckling modes from the FE analyses appeared the same in the experimental tests. Moreover, [Tsavdaridis and D'Mello, 2011b] carried out another study and concluded that perforated beams are governed by Vierendeel bending capacity instead of web-post buckling failure when the web thickness and web-post width were within a certain range.

Connection type

High stresses around CJP weld, particularly in the top flange, increase the possibility of brittle failure in the connection, presented as brittle fracture occurred because of the high tri-axial stress-state in the CJP weld region that can induce micro-cracking and trigger rapid crack propagation [Tsavdaridis et al., 2017]. Therefore, [Tsavdaridis and Papadopoulos, 2016] recommended the use of bolted connections due to their cost and safety. Bolted connections are more efficient in energy dissipation as they dissipate energy though yielding of the connection components. [Mashaly et al., 2010] also stated that extended end-plate is ideal for special moment-resisting frames subjected to lateral loads due to their ductility and energy dissipation. A FE analysis was carried out on four-bolt extended end-plate connections and the studied models achieved a minimum storey drift value of 5% under cyclic loading, i.e., a ductile behaviour.

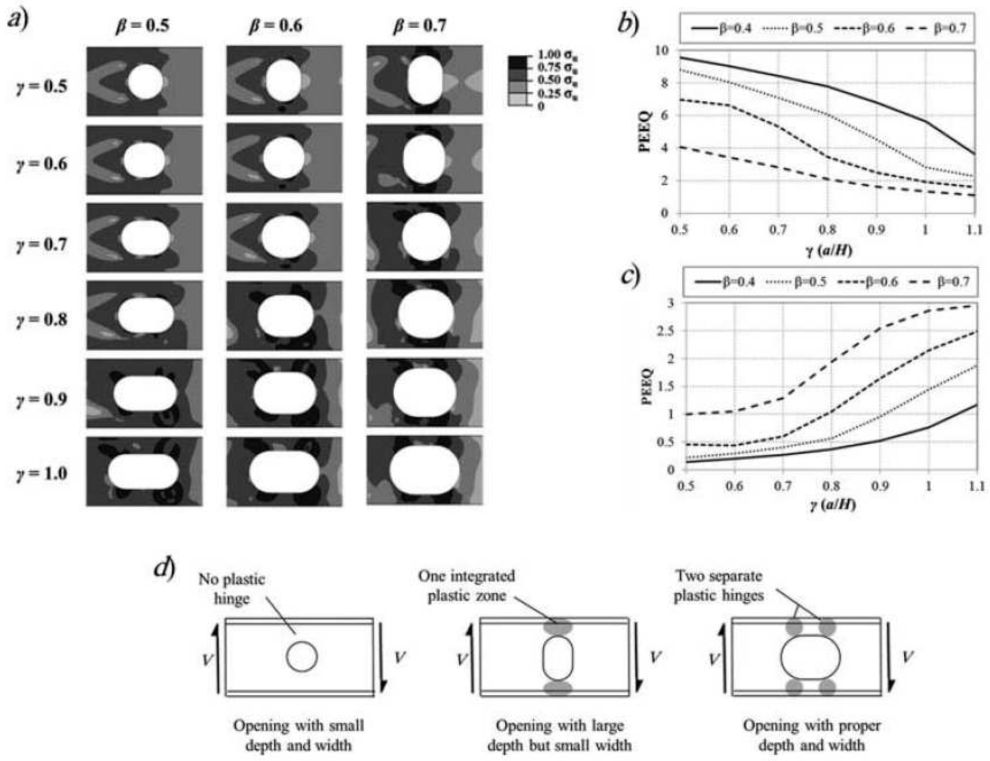

Figure 1 Effect of opening depth and length on: (a.) stress-strain distribution (b.) Equivalent plastic strain at connection welds (c.) equivalent plastic strain at opening corners (d.) Critical failure modes (Erfani and Akrami, 2016)

Methods

This present study focuses on the behaviour of extended end-plate reduced web section connections (RWS) under cyclic loading. The failure modes of RWS connections were demonstrated through a full-scale experimental test carried out in the St. George's laboratory at the University of Leeds. The specimens were designed according to and in line with [SCI P398, 2013] and a previous FE study by [Tsavdaridis and Papadopoulos, 2016].

Figure 2 General arrangement of test specimen (RWS_3)

The RWS connection specimens selected for this study were RWS_1 (single web opening as shown in Figure 3), RWS_2/RWS_3 (multiple closely spaced web openings as shown in Figures 2 and 4), and RWS_4 (multiple closely spaced web openings with the 1st opening being isolated as shown in Figure 5). The intention was to examine the performance of typical UK extended end-plate connections while using a standard cellular beams designed according to [SCI P355, 2011]. The beams were altered having only one opening (RWS_1) while simulating the RBS connection (the same amount of material removed from the web opening as the four flange cuts of the tested RBS). RWS_4, was created to simulate a fuse-connection type with an isolated opening which will ideally absorb the seismic energy by concentrating high shear stresses in its vicinity.

Linear variable differential transformers (LVDTs) were mounted on these specimens to monitor the deformations of different parts of the RWS connection. For comparison, an RBS connection with identical steel sections is tested with the same loading protocol; the comparison results are presented in a complimentary study. To achieve the governing failure mode of RWS connections, monotonic loading was applied to each specimen until the actuator reached its capacity.

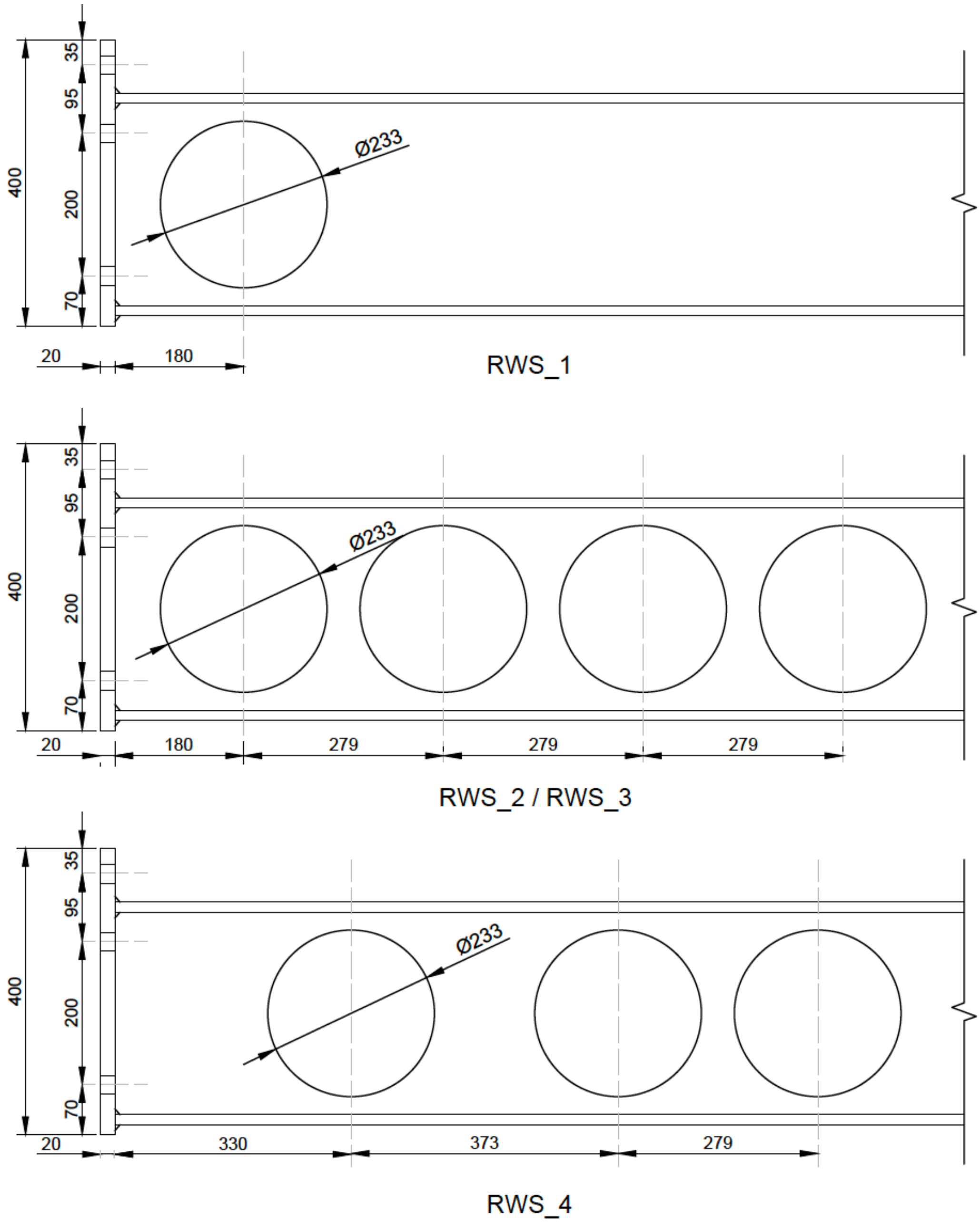

Figure 3 Cross-section of RWS specimens

Results

RWS_1 achieved an ultimate moment capacity of 210.8kNm under cyclic loading. In comparison, RWS_2 and RWS_3 are identical specimens and they achieved similar ultimate moment capacity of 215.62kNm and 212.4kNm respectively. The difference between RWS_1, RWS_2, and RWS_3 are approximately within 5%, meaning that the number of web opening does not directly influence the moment capacity of an RWS connection. Based on the observations, most of the yielding of these specimens occurred in the 1st web opening. Despite most yielding of these specimens initiated in the 1st web opening at the same location, the stress concentration varied differently throughout the loading test as shown in Figure 6**Error! Reference source not found.**. The deformation across the 1st web opening in different specimens also vary where the deflections across the 1st web opening is shown in Figure 7. Despite RWS_2 and RWS_3 experienced web-post buckling last few cycles, they still achieved an ultimate moment capacity similar to RWS_1, which suggests that the Vierendeel mechanism was the governing failure mode. In contrast, RWS_4 achieved a lower ultimate moment capacity of 179.66kNm, as it failed under cracking of the weld. This indicates that the ultimate moment capacity of RWS_4 was mainly dependent on the strength of the end-plate connection as most of the yielding occurred in the end-plate connection.

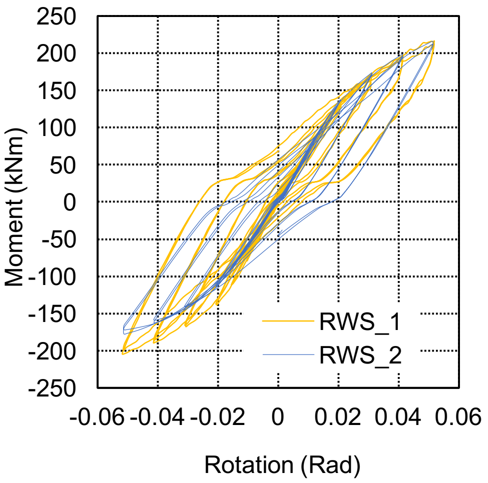

Figure 4 Hysteretic graph of RWS_1/RWS_2

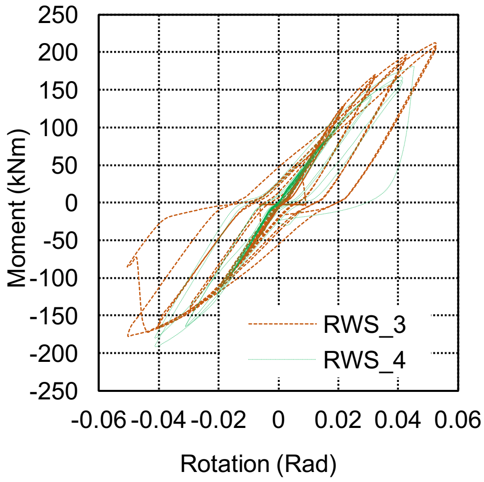

Figure 5 Hysteretic graph of RWS_3/RWS_4

RWS_2 and RWS_3 had almost identical behaviours where both of these specimens displayed strength degradation when loaded downward and strain hardening when loaded upward. In contrast, RWS_1 had a different behaviour where it displayed strength degradation when loaded upward and strain hardening when loaded downward. These observations suggest that RWS_1 deformed differently than RWS_2 and RWS_3 where both groups of specimen dissipated their energy through different part of the structural components.

RWS_1 dissipated the energy through yielding of the end-plate as the end-plate connection is relatively elastic than the RWS connection which only has one web opening. The stress was mostly concentrated in the end-plate rather than the 1st web opening. Due to the unsymmetrical extended end-plate connection, the end-plate has a non-uniform stress distribution along the surface. The top part of the end-plate contains more bolts and plate area, therefore in comparison the lower part of the end-plate generally has higher residual stress and plasticity. This caused the strength degradation of RWS_1 when loaded upward and strain hardening when loaded downward.

RWS_2 and RWS_3 had a more elastic RWS connection which has multiple web openings along the beam. Due to the lower stiffness of the RWS connection, the stresses were mostly concentrated in the web openings rather than the end-plate connection. Hence, mostly yielding of RWS_2 and RWS_3 occurred in the RWS connection rather than the end-plate connection. The lateral deformation of the web posts suggested the web post buckling which caused the strength degradation of the specimen. In addition, due to the unsymmetrical stiffness of the extended end-plate connection, the bottom flange of the RWS connection experienced higher stress when comparing with the top flange and ultimately the early development of plasticity in the bottom flange of the first few web openings. Therefore, strength degradation occurred in RWS_2 and RWS_3 when the beam was loaded downward and strain hardening when loaded upward.

Moreover, it was observed that most deformation was found at the bottom flange for specimen RWS_2 and RWS_3. The results indicate that the strength degradation of RWS_2 and RWS_3 was due to fatigue of the beam section; particularly the bottom flange at the 1st web opening. RWS_1 was mainly deformed through the end-plate connection; particularly at the bottom section of the end-plate due to the elongation of the bolts and the prying action of the end-plate.

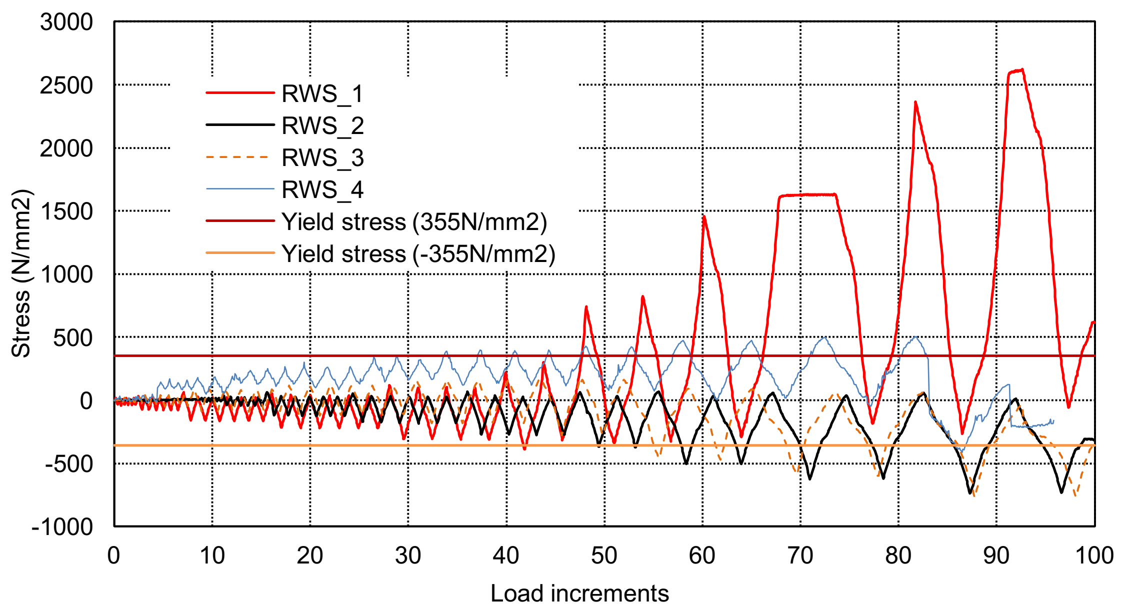

Figure 6 Stress concentration in the first plastic hinge

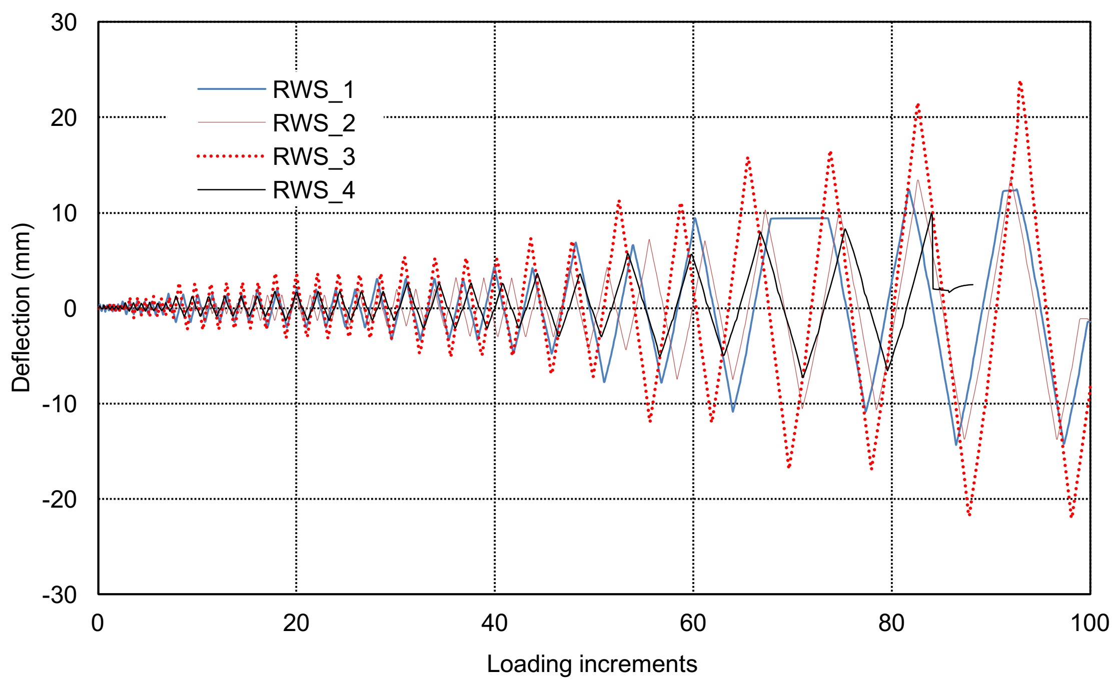

Figure 7 Vertical deflection across the 1st web opening

Although RWS_4 did not sustain throughout the whole cyclic loading test, it did not have any signs of strength degradation. It shows continuous strain hardening of the structural elements during the test, indicating that plasticity of the beam specimen was not fully developed. In addition, the entire beam section had very little to no deformations. As suggest in [Tsavdaridis and Papadopoulos, 2016], the 1st web-post restricts the movement of the stress throughout the span, therefore more plastic stresses will be concentrated in the vicinity of the extended end-plate connection.

Figure 8 Lateral displacement of RWS_1

Figure 9 Lateral displacement of RWS_2

Figure 10 Lateral displacement of RWS_3

Figure 11 Lateral displacement of RWS_4

In terms of energy dissipation, RWS_1 shows significant signs of pinching in comparison with the RWS_2, RWS_3, and RWS_4. This indicates that the energy was mainly dissipated through a limited range of structural components and hence the plasticity was fully developed at an early stage where energy was not able to be dissipated through yielding. RWS specimens with multiple web openings (i.e., RWS_2, RWS_3, RWS_4) dissipated energy more efficiently which suggests that the energy was uniformly dissipated through various structural components. All RWS specimens had little lateral displacement throughout the experimental test as shown in Figure 8, Figure 9, Figure 10 and Figure 11.

It is worth to note that all RWS specimens had similar and low elastic column deformation, demonstrating that the RWS connection is efficient to achieve the “weak beam - strong column” mechanism even when used in the design (or rehabilitation) of UK (non-seismically designed) connections with weaker columns than the deep columns that would have been used in the design of seismic resistant frames.

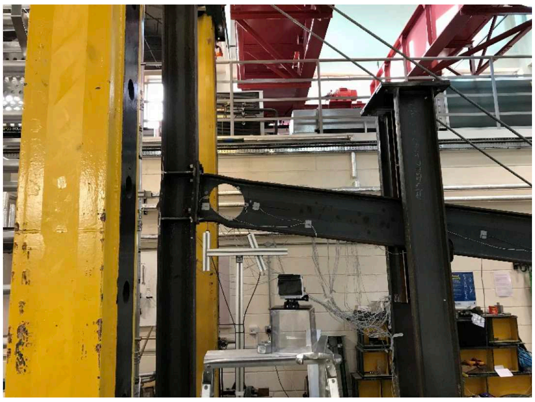

Figure 12 Ultimate failure of RWS_1

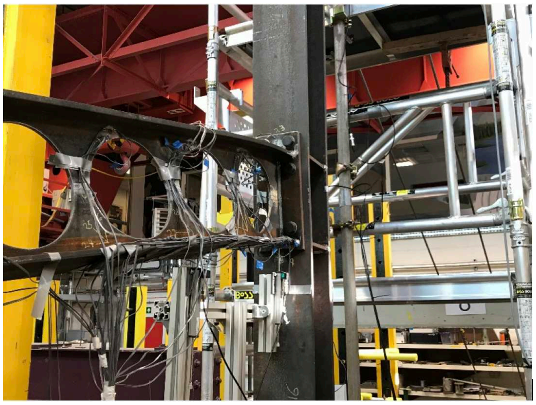

Figure 13 Ultimate failure of RWS_2

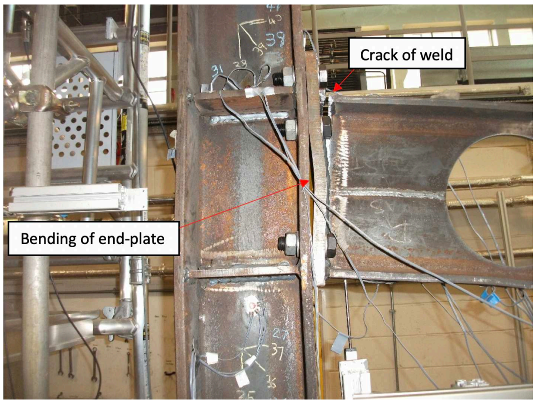

Figure 14 Ultimate failure of RWS_4

Conclusion and Contributions

This paper presents a series of comprehensive experimental test results for RWS connections with different geometrical properties to understand their behaviour under cyclic loading and the differences in performance when comparing with the well-known RBS connections in a complimentary paper.

From the experimental results, it was found that RWS connection can be considered as a seismic connection which can dissipate energy efficiently with minimal pinching and buckling. Energy was dissipated through yielding of the web openings while the specimens generally achieved a minimum of 0.45 Rad after the loading test. The test specimens also achieved high moment capacity. The reduction of stiffness through the web openings appeared to provide a higher moment capacity and lesser chances of web buckling similar to RBS connections due to the trimmed flanges.

The results of this paper confirm the FE results previous presented by [Tsavdaridis and Papadopoulos, 2016] as well as further concludes that: (a) the yield moment capacity of RWS connection mainly depends on the 1st web opening regardless of the number of web openings along the length of the beam, and (b) the extra wide 1st web-post of RWS_4 will restrict the stress mobilisation and stiffen the end-plate connection, ultimately causing the cracking of the weld. It was also found that the stress concentration is heavily dependent on the stiffness ratio between the end-plate connection and the RWS connection.

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