HYSTERETIC PERFORMANCE OF REDUCED WEB SECTION (RWS) CONNECTIONS WITH DEMOUNTABLE SLABS AND EFFECT OF COMPOSITE ACTION

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Abstract: *Little attention has been paid to reduced web section (RWS) connections with composite slabs under cyclic loading. RWS connections have been proven to act as ductile fuses with a most promising and straightforward choice that requires only one perforation within a beam web without removing the concrete slab. Thus, it could be an economic benefit in terms of both manufacture, usage, and seismic retrofit, while limiting instability, protecting non-ductile elements. This paper introduces an experimental test of demountable steel-concrete composite bolted reduced web section (RWS) connections. A single circular opening with diameter equal to 0.8 of the beam's depth, was fabricated near the beam-column joint. Two different parameters are investigated, the effect of the web opening location, and the presence or absence of bolted shear studs over the protected zone. The assessment of retrofitted connections was also examined by creating a web opening of the solid-webbed specimen after exposing it to cyclic actions representing moderate seismicity. Based on the results, employing composite RWS connections as a retrofit non-seismically designed buildings, can provide an excellent solution in terms of strength, ductility and energy dissipation. This was achieved by the formation of the Vierendeel mechanism, resulting in concentrating the high plasticity on the beam, which in turn leads to an increase in the deformability and ductility of the connections. The presence of composite action over protected zone could lead to a lower capacity of RWS connection, regardless of extra row of studs. This was attributed to high stress demands not only in the bottom flange of beam, but also in the top Tee-section of opening. All RWS connections were capable of satisfying the seismic requirements of ANSI/AISC 358-16, ANSI/AISC 341-16 and EC8. Moreover, the bolted shear studs showed an excellent seismic performance and could be easily dismantled after testing.*

Introduction

Based on the observations from previous earthquakes, the damage in steel moment resisting frames (MRFs) during strong earthquake events is generally due to the brittle failure of the weld connections subjected to excessive strain demands. Such strain demands on the bottom flange as a result of the composite slab, which might be several times larger than that with bare steel, causing a higher potential of such failure (Kim et al., 2004). Therefore, proper assessment should be made to prevent any unacceptable behaviour, such as the strong beam-weak column mechanism. This could be dominant mechanism if the contribution of composite slab is neglected in the design process (Roeder, 2002).

Different prequalified connections and requirements have been presented to address this concern (ANSI/AISC 358-16, 2016). Such connections have showed the ability to plasticize the desirable locations at beams to cap inelastic action at the welds and increase the deformability of MRFs (Roeder, 2002; Kim et al., 2004; Huang et al., 2014; Li et al., 2017; Kim and Lee, 2017; Mou et al., 2019; Di Benedetto et al., 2020). Compliance with the isolation slab and the avoidance of composite actions over the protected zones, could also help to shield non-ductile elements for inelastic engagement (Civjan et al., 2000; Civjan et al., 2001; Sumner and Murray, 2002; Jones et al., 2002; Eurocode, 2005d; Zhang and Ricles, 2006; ANSI/AISC 341-16, 2016; ANSI/AISC 358-16, 2016; Lee et al., 2016; Di Benedetto et al., 2020). Such requirements give an advantage

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in mitigating strength deterioration and strains generated in the vicinity of the bottom beam flange near the column face while no fracture was observed.

Reduced web section (RWS) connections show the capability to be employed as fuse with an satisfactory performance under several types of loading, while limiting instability and protecting the other components of the joint (Yang et al., 2009; Boushehri et al., 2019; Zhang et al., 2019; Jia et al., 2021; Tsavdaridis et al., 2021; Bi et al., 2021; Lin et al., 2021; Tabar et al., 2022). This attributes to its potential ability of exploiting the openings' location at high shear to regulate the global and local actions acting in the vicinity of the web opening to form the Vierendeel (ductile) mechanism. Also, such connections provide the simplicity of retrofitting, while facilitates the formation of strong column-weak beam mechanism.

The presence of a composite slab is highly disruptive and dictates to modify only the beam bottom flange. While RWS connections require only perforation within a beam web without removing the composite slab which lead to straightforward retrofit solution. Thus, it represents a promising and straightforward solution that could be fully utilised in existing and new buildings. However, a computational study of Shaheen et al., 2018 has indicated that the composite action should be considered due to its effects on the seismic behaviour of the connections under cyclic loads. Also, Shaheen et al., 2018 found that small to medium web opening sizes should be considered due to detrimental impact of large opening on the cyclic behaviour of composite RWS connections. It is worth to note that Shaheen et al., 2018 studied the composite RWS connections with the presence of composite action over web opening. However, no research has been found in the literature that investigates the effect of presence of shear studs (bolted or/and welded) over the web opening on cyclic behaviour of RWS connections.

Recently, researchers and engineers have shown an increased interest in bolted shear studs due to their demountability that enables reuse. The entire beam might need to be replaced after moderate to strong earthquakes due to concentration of damage at the weakened areas (i.e., web opening). Since traditional steel–concrete composite beams with welded shear studs would be impractical to dismantle the highly damaged steel beam, demountable bolted shear studs have been introduced as practical alternatives to the traditional systems (Moynihan and Allwood, 2014; Ataei et al., 2016; Ataei et al., 2017; Liu et al., 2017; Yang et al., 2018; Girão Coelho et al., 2020; Chiniforush et al., 2021) (see Figure 1). Such bolted shear studs are easier to dismantle than welded studs without destroying the RC slab. Thus, they enable the steel beam to be removed, particularly when the bolted connections are used. The combination of structural fuses, namely RWS, bolted end-plate connections (Tartaglia et al., 2019) and bolted studs enable the ease of fabrication and assembly, and rehabilitation.

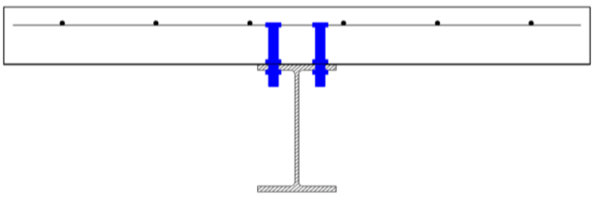


Figure 1. Demountable bolted

This paper introduces experimental tests of demountable composite RWS connections subjected to cyclic loading to investigate their hysteretic performance. Also, the effect of presence or absence of composite engagement over the plastic zone was examined. In addition, the assessment of retrofitted connections by incorporating RWS into bolted extended end-plate connection was studied for rehabilitation purposes. To achieve this, exposing the extended end-plate connection with solid-webbed beam to cyclic loads representing moderate seismicity. Afterward, a web opening was created, and then the specimen was re-tested. The above process simulates a practical rehabilitation technique and demountability of such combination of structural fuses for existing steel structures with solid-webbed beams.

Experimental program

The experimental campaign consists of four identical steel-concrete composite connections subjected to cyclic loads in accordance with the loading protocol in the AISC 341 (ANSI/AISC 341-16, 2016) (Figure 2). All specimens represented an exterior unstiffened bolted extended end-plate connection in steel structures. A specimen test matrix is presented in **Error! Reference source not found.**.

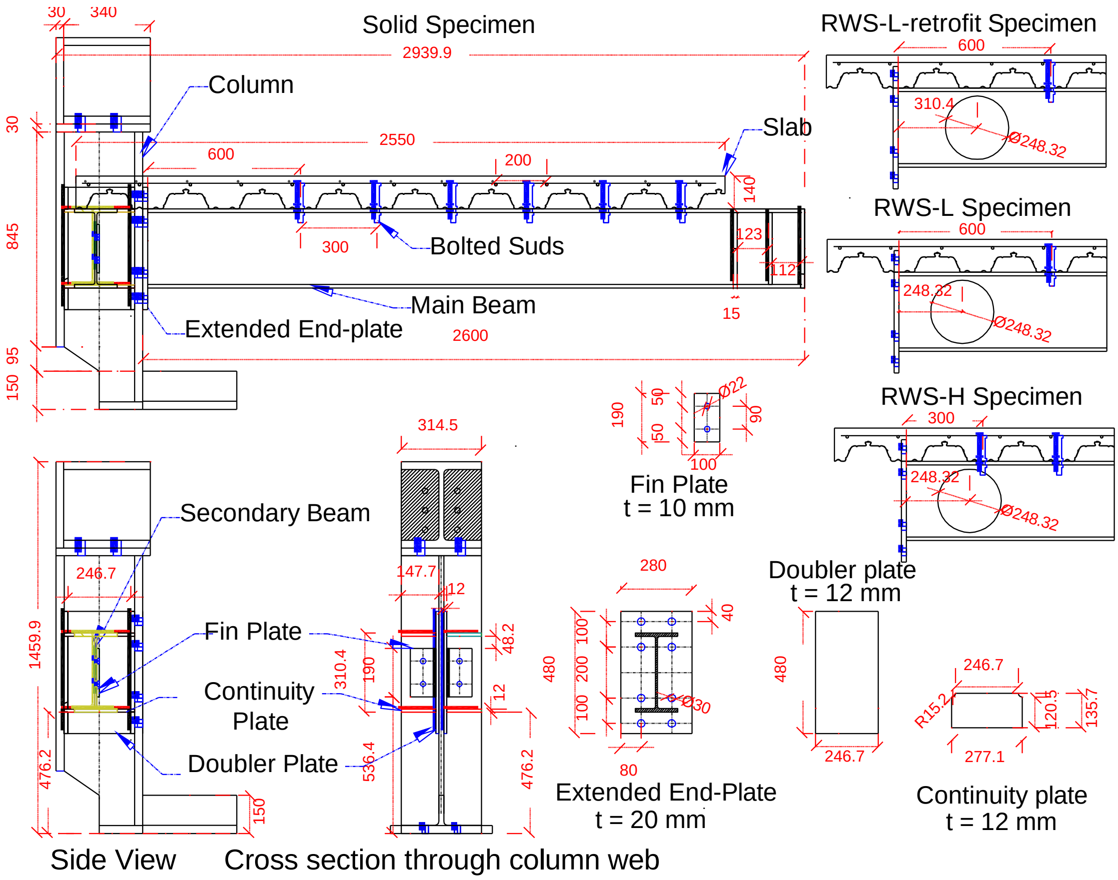


Figure 2. Dimensions of test specimens (mm)

Specimen ID		Solid	RWS-L-retrofit	RWS-L	RWS-H
Web opening	Diameter d	-	$0.8h$	$0.8h$	$0.8h$
	End-distance S		$1h$	$0.8h$	$0.8h$
$M_{pl,Rd}$ or $M_{pl,Rd,RWS}$ (kNm)		300.2	257.1		
$M_{j,Rd}/M_{pl,Rd}$		1	1.17		
Joint Category		Partial strength	Full strength		
Primary and secondary beams		305x165 UB 54			
Column		305x305 UC 198			
Bolts		M27 Gr. 10.9 with preloading force of 321 kN			
Two rows of bolted studs		M20x160 mm - Gr. 8.8 with preloading force of 40 kN			
Note: h = height of the beam; 80d = means the diameter of the web opening is equal to 80% of h; 80S = means the **e**nd-distance is equal to 80% of h. $M_{j,Rd}$ = joint capacity. $M_{pl,Rd,RWS}$ = the nominal plastic bending capacity for the steel section with a web opening = $F_y\ (W_{pl,y} - \frac{d_o{}^2.t_w}{4})$.					

Table 1. Specimen test matrix

The first specimen was a partial-strength composite connection with solid-webbed beam (refers as the solid specimen). It was designed based on the nominal plastic moment resistance of the connected bare steel solid-webbed beam $M_{pl,Rd}$, without including the contribution of composite slab, and compliant with EC3-1 and -8, EC4-1 EC8-1, and SCI P428 (Eurocode, 2005a; Eurocode, 2005b; Eurocode, 2005c; Eurocode, 2005d; Girão Coelho et al., 2020). The solid specimen was tested under cyclic loading to achieve around 70% of its sagging capacity to mimic effects of moderate seismicity for rehabilitation purposes. Beam was perforated with single web opening that equal to 80% of the beam height, and the specimen was re-tested. Noteworthy, the web opening was created off-site, thus the composite slab was disassembled and new M27 bolts for connection were used.

The second composite specimen to be tested under cyclic loads, was the re-tested specimen, (hereinafter referred as RWS-L-retrofit). This led to examine the effects of residual stresses due to previous moderate earthquake events. The other specimens were two identical composite RWS connections that designed based on the capacity of solid specimen, following the requirements of SCI P355 (Lawson and Hicks, 2011) and SCI P428 (Girão Coelho et al., 2020) as shown in Table 1. The only difference between these two RWS specimens, was the presence or absence of composite action (i.e., bolted studs) above the web opening. RWS-L specimen was the one that had no bolted studs from the connection face to the ends of the web opening. While RWS-H specimen was the one that had bolted studs along the connected beam including the web opening.

Placing bolted studs along the connected beam including the web opening represents the high composite action. This allows to examine the effect of incorporating the web openings on such connections. The results could be employed for performance comparison on highly coupled slab-beam as many of the existing buildings have studs all along the entire beams.

While a low composite action can be achieved by preventing the bolted studs from the connection face to the ends of the web opening in the case of composite RWS connections according to EC8-1 clause 7.7.5 (Eurocode, 2005d). Regarding the composite solid specimen, the Prequalified Connections guidance (ANSI/AISC 358-16, 2016) recommends no composite engagement in the area of the column face to 1.5 of the depth of the connecting beam. All four specimens had a 25 mm gap between the components of connection and concrete slab, to avoid the crushing and cracking of the concrete according to the ANSI/AISC 358-16 (ANSI/AISC 358-16, 2016) and EC8-1 clause 7.7.5(2) (Eurocode, 2005d).

A strong stocky column was used to ensure that the development of a plastic hinge mechanism initiates in the beam during cyclic loading to avoid any contribution from the column. The beam section was selected based on the slenderness and span-to-depth ratios, taking into account the test rig size (Figure 3). The beam section was a highly ductile section, according to US seismic code (ANSI/AISC 341-16, 2016) and EC3 (Eurocode, 2005a).

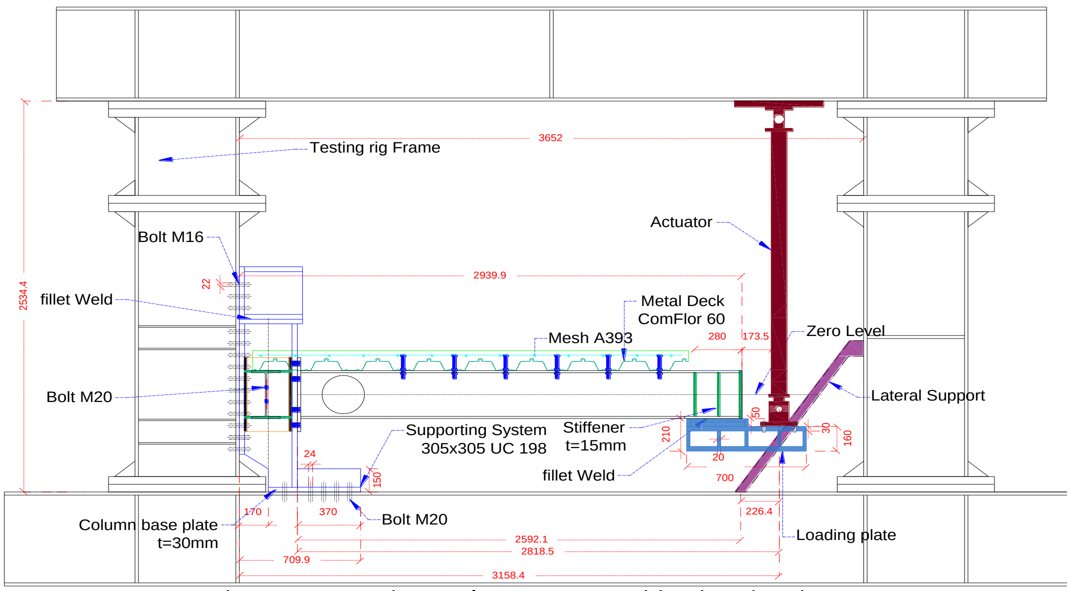


Figure 3. Experimental test setup – side view (mm)

Experimental results

The incorporation of the web opening (reduced web section) into the solid-webbed beam decreased the strength capacity of the connected beam. Therefore, it altered the category of the connection from partial to full strength as the capacity ratio $M_{j,Rd}/M_{pl,Rd}$ of the joint-to-beam was increased by 17%. This means that it could keep the connection and column away from plastic engagements and any undesirable behaviour. It also indicates that the reduced web section entails the strong connection-weak beam concept to be dominant mechanism. However, the high strength degradation after 4% rotation in all RWS specimens would be a concern for their application in high seismic zones.

It should be highlighted that the strength capacities of all RWS connections after 4% rotation were higher than 80% of the steel solid-webbed beam full plastic moment ($\boldsymbol{M_{pl,a,Rd}}$) see Table 2. Also, they were higher than 80% of the moment resistance of steel section at opening ($\boldsymbol{M_{o,Rd,steel}}$) (Table 2). In details, the applied sagging moments at 5% rotation were 307.5kNm, 305.8kNm, and 290.9kNm for RWS-L-retrofit, RWS-L and RWS-H, respectively. While the applied hogging moments were -267.8kNm, -250.5 kNm, and -243.3kNm for RWS-L-retrofit, RWS-L and RWS-H, respectively.

In all these cases, all applied moments under both directions were higher then $0.8\boldsymbol{M_{pl,a,Rd}}$ = 240kNm and $0.8\boldsymbol{M_{o,Rd,steel}}$ = 205.7kNm. Thus, the seismic codes requirements of ANSI/AISC 358-16 ANSI/AISC 341-16 and EC8 were met in all three tested RWS connections (Eurocode, 2005d; ANSI/AISC 341-16, 2016; ANSI/AISC 358-16, 2016). Also, Shaheen et al. (2018) stated that, for composite RWS connections, a smaller diameter than 0.75*h* and short end distance of a web opening is more effective in mitigating the plastic engagements around the connection and leads to more ductile performance.

For seismic design applications, in general, it is important to provide a stable hysteretic response with high energy dissipation capacity at plastic hinge locations to control the key performance response parameters (Bernuzzi et al., 1996). In this paper, RWS connections performed as expected in terms of enforcing the ductile failure, alleviating the slabs' cracks, and achieving higher moment capacities than the steel beam full plastic moment (M_{pl}) at 4% rotation in positive and negative directions (see Figure 4). The stability of the hysteretic behaviour of the beam-column connections is a key role in the capability of the entire MRF system to well dissipate energy. All RWS specimens showed stable energy dissipation capabilities without pinching phenomena and a good ductility. Even RWS-L-retrofit specimen that was cyclically loaded and then had retrofit improvement by creating a web opening, behaved as well as the other RWS connections, apart from concrete cone failure. These stable hysteretic performances were attributed to the force redistribution in the vicinity of the web opening due to an early development of plastic hinges. Such redistribution of the global forces led to the domination of the Vierendeel mechanism in the beams rather than failure in the connections. Such mechanism limited the shear forces that could be transferred to the components of the connections which is favourable in seismic design.

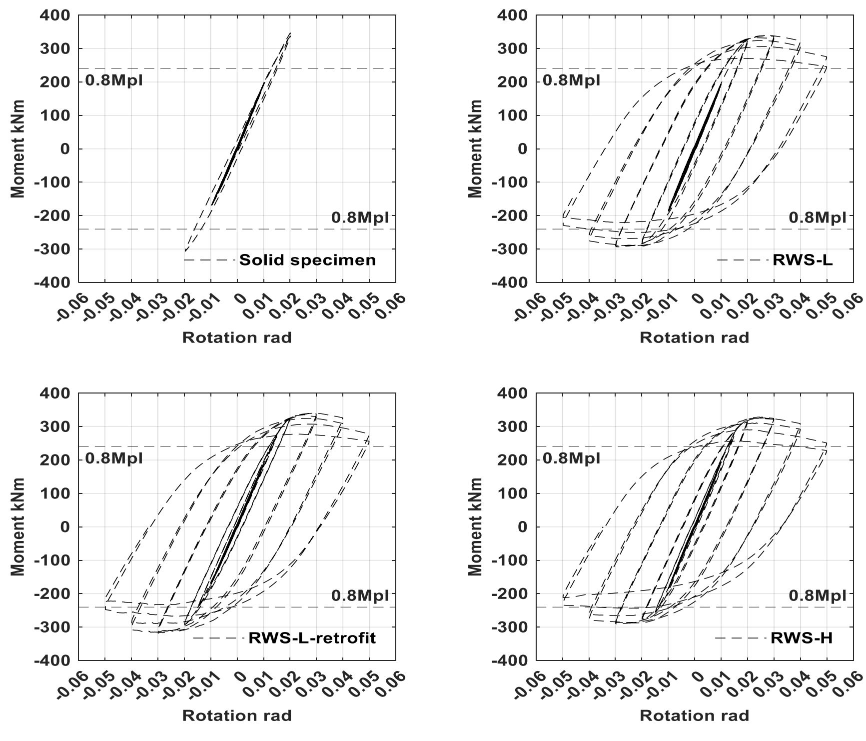


Figure 4. Hysteretic curve of the specimens.

Among all RWS specimens, RWS-H specimen had slightly less strength capacities in both directions as shown in Table 2. This was not expected in comparison with the identical specimen RWS-L, regardless of the extra row of the shear studs. This extra row of studs should had given a superior to RWS-H specimen over the other RWS specimens in terms of the strength, especially in sagging. However, the extra row of studs led to higher strain demands in the top Tee-section (Figure 5). This caused the earlier development of plastic hinges in the top Tee-section at Lower moment side (LMS) because of their location above the LMS. This behaviour was reasonable since the web opening consists of top and bottom Tee-sections with similar local behaviour under the same global action. Each Tee-section also consists of top and bottom parts that will go into compression and tension under the same global action, as shown in Figure 5. The location of the extra row of studs above LMS led the bottom part of the top Tee-section to experience a higher strain demand. Subsequently, earlier plastic hinges initiate, which in turn led to an earlier crack at the LMS. This behaviour was believed to cause RWS-H specimen to experience lower capacity.

		RWS-L-retrofit Connection	RWS-L Connection	RWS-H Connection
M at column face $\boldsymbol{M_f}$ (kNm)	+ ve	340.2	339.4	328.7
	- ve	- 318.5	- 293.4	- 290.3
M at opening centreline $\boldsymbol{M_o}$ (kNm)	+ ve	300.3	307.1	290.1
	- ve	- 281.2	- 265.4	- 256.2
$M_f / M_{pl,a,Rd}$	+ ve	1.13	1.13	1.09
	- ve	- 1.06	- 0.98	- 0.97
$M_f / M_{pl,Rd}$	+ ve	0.68	0.68	0.66
	- ve	- 0.83	- 0.76	- 0.75
$M_o / M_{o,Rd,steel}$	+ ve	1.17	1.19	1.13
	- ve	- 1.09	- 1.03	- 1.00
$M_o / M_{o,Rd,comp}$	+ ve	0.97	0.99	0.91
	- ve	- 1.09	- 1.03	- 1.00
θ_u (rad)	+ ve	0.0499	0.0499	0.0499
	- ve	- 0.0499	- 0.0498	- 0.0499
θ_y (rad)	+ ve	0.0170	0.0163	0.0164
	- ve	- 0.0151	- 0.0131	- 0.0131
M_y (kNm)	+ ve	302.9	289.9	296.7
	- ve	- 252.0	- 247.6	- 244.9

Note: M_f =max moment at face of the column. M_o = max moment at opening centreline. $M_{pl,a,Rd} = F_y\, W_{pl,y}$ Nominal bending moment resistance of steel solid webbed beam. $M_{pl,Rd}$ = Bending moment resistance of composite solid webbed beam using stress block method. $M_{o,Rd,steel} = F_y\,(W_{pl,y} - \frac{{d_o}^2.t_w}{4})$ Nominal bending moment resistance of steel section at opening. $M_{o,Rd,comp}$ = Bending moment resistance of composite section at opening according to SCI P355.

Table 2. Results Summary

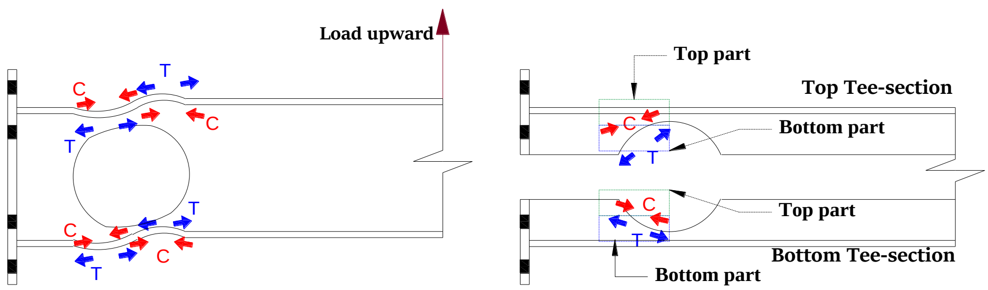


Figure 5.Illustration of the behaviour of Tee-sections.

The design procedure described in the note of Table 2 shows that $\boldsymbol{M_{pl,a,Rd}}$ could be enough to estimate the applied moment at the column face with overstrength factors of 9% and 11%. It can also be seen that the applied moment at web opening could be calculated by using the equation $\boldsymbol{M_{o,Rd,steel}}$ where the reduced section due to the opening was considered. Table 2 also indicates that the capacity of composite section at column face was overestimated when using the stress block method. However, the capacities of steel and composite sections at the centreline of the web opening could be predicted by using SCI P355 (Lawson and Hicks, 2011).

Figure 6 shows the stiffness degradation of all specimens. RWS specimens had slightly similar pattern in comparison to the solid specimen up to 0.2rad in both directions. It worth to note that the solid specimen was tested up to 0.2rad. However, it was predicted that the solid specimen would be experiencing less stiffness degradation among all specimens if the test was completed until the end of protocol. The stiffness degradation in positive and negative directions was less than 20% in the solid specimen. While in RWS connections, the degradation in stiffness was ranged between 72% to 79% in both directions in comparison to the initial stiffness values.

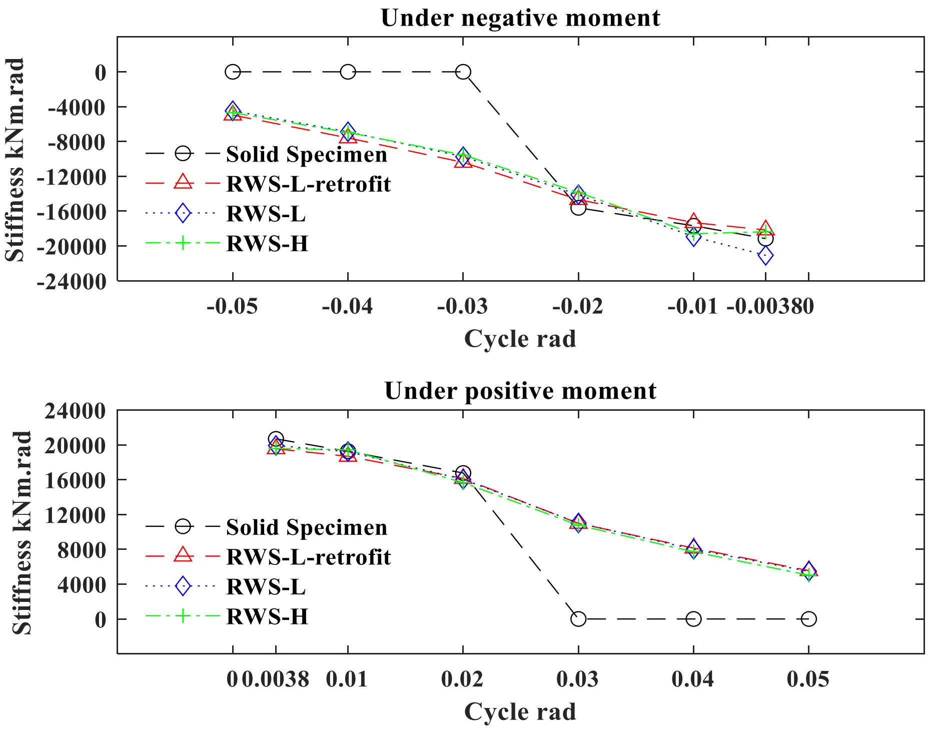
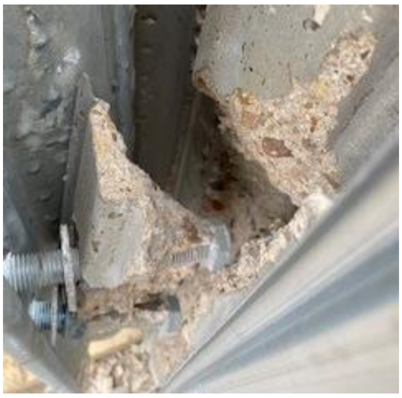
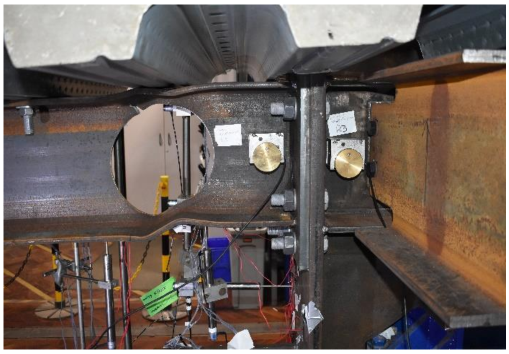


Figure 6. Degradation in the stiffness for all tested specimens

The failure mode of all 3 RWS specimens was the Vierendeel mechanism (Figure 7). All specimens experienced local buckling of beam flanges above and below the web opening and then straightening out by changing the load direction. This pattern indicates that Vierendeel mechanism was developed. The introduction of web opening alleviated the cracks of reinforced concrete (RC) slab in RWS-H specimen. This can be seen in the similar crack patterns in RWS-L specimen and its identical specimen RWS-H. Where the only difference between the two specimens was the extra row of studs (composite action) that placed over the web opening in RWS-H. Regarding RWS-L-retrofit (re-tested specimen), only one more line of crack was seen, plus the two micro-crack lines that were observed in the solid specimen. These two micro-crack lines became visible, however, did not propagate deeply inside the RC slab. Nevertheless, concrete cone failure with concrete crushing was observed in RWS-L-retrofit after the removal of the metal deck. No fracture was found in the bolted studs. No visible damage observed in all other steel members in all RWS specimens. Only in RWS-H, the washer of bolted studs was failed over the web opening by bending, but the shanks of studs did not bend.



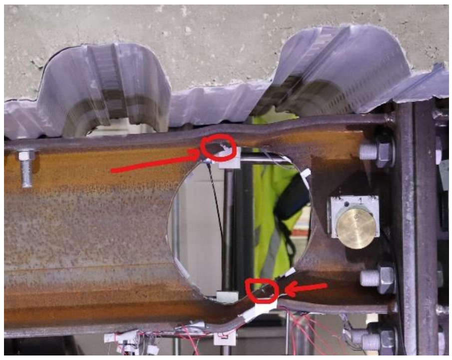
Concrete cone failure.



Top crack at the end of first cycle of 0.05 rad (hogging).

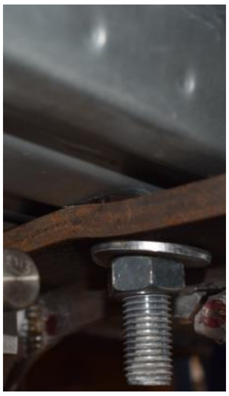
a) Failure modes of RWS-L-retrofit specimen

Vierendeel Mechanism.

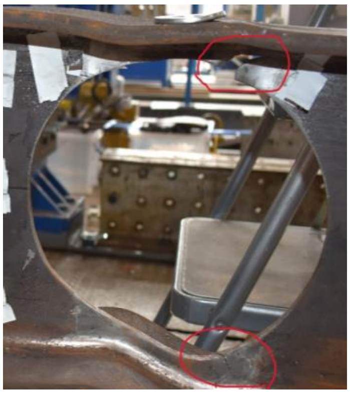


Second and third cracks in RWS-L connection.

b) The failure modes of RWS-L connection



Bolted studs' failure.



Cracks.

c) The failure modes of RWS-H connection

Figure 7. Failure modes of RWS specimens.

Concluding remarks

Four steel-concrete composite bolted reduced web section (RWS) connections were experimentally tested to investigate the effect of engagements of composite action over the protected zone. This allows to augment the test database for further experimental and FE investigations. Based on the obtained results the following conclusive remarks can be pointed out:

- The capability of the RWS connections to exploit perforation's location in high shear zone to trigger plasticity in Tee-sections, resulting in ductile (Vierendeel) failure in connected beam.
- RWS connections could be employed as ductile fuse in existing and new buildings.
- The domination of Vierendeel mechanism on the behaviour of the connection proves the need to include the effect of web opening in the component method approach used in EC3 for a joint design.

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