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## Full paper

## Evaluation of Reduced Web Section (RWS) Connections Subjected to Cyclic Loading

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## Abstract

This study investigates the performance of Reduced Web Section (RWS) connections subjected to cyclic loading, using finite element analysis (FEA) to assess their potential as a retrofit strategy for steel structures. RWS connections, which involve perforations in the beam web while preserving flange integrity, aim to enhance moment capacity and reduce out-of-plane instability. They also offer practical benefits such as retrofitting from below without floor demolition. Detailed FEA models were developed and validated against experimental data. A parametric study of 300 models (IPE270, IPE330, and IPE400 beams) evaluated key geometric parameters, including perforation diameter and its distance from the column face. Performance metrics such as yield moment ( $M_y$ ), peak moment ( $M_m$ ), yield ( $\theta_y$ ) and Ultimate ( $\theta_u$ ) rotations, and strength degradation were assessed. Results show that RWS connections can achieve drifts above 4% with less than 20% strength degradation, effectively mobilising plastic deformation away from joints into protected zones, improving seismic performance.

## Keywords

Beam-to-column joint, Reduced Web Section (RWS), cyclic loading, Seismic performance, Seismic Retrofit

## 1 Introduction

Construction of new steel structures exacerbates climate change. The World Steel Association estimates that the steel industry accounts for 7-9% of global direct CO<sub>2</sub> [1]. Retrofitting steel buildings is often more sustainable and cost-effective than demolition and new construction [2]. Numerous investigations have examined how retrofitting improves structural integrity in existing buildings, resulting in the introduction of Joint strengthening and beam weakening [3] [4].

Reduced web section connection (aka RWS connection) is a beam-weakening strategy that cuts the beam web to create a protected region where inelastic action clusters away from the column face. Also, construction is easier because access is from the floor below, avoiding slab demolition. The openings also make installing utilities and equipment easier. Compared to trimming the flanges, the cross-section has a higher moment of inertia. Hence, out-of-plane stability is less likely compared to RBS

connections [5]. Mark Amos Aschheim's 2000 patent [6] introduced RWS in seismic design by using web voids to reduce connection stress and improve seismic resilience. Comprehensive studies have shown that RWS connections can achieve high ductility and stable hysteresis cycles, making them an effective seismic retrofitting option. This is supported by numerical simulations and laboratory testing [7] [8] [9] [10] [11]. The overall geometry of a welded RWS connection with RWS geometry parameters is depicted in Figure 1.

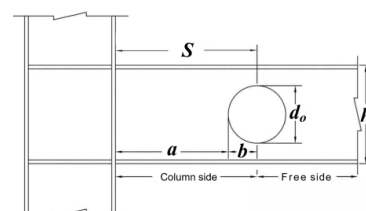
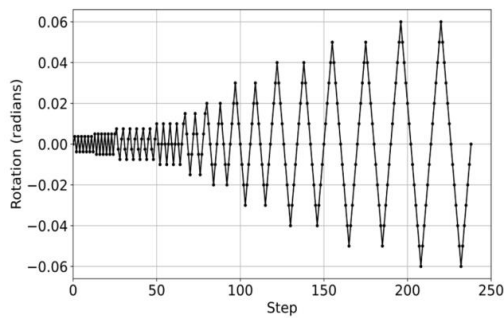


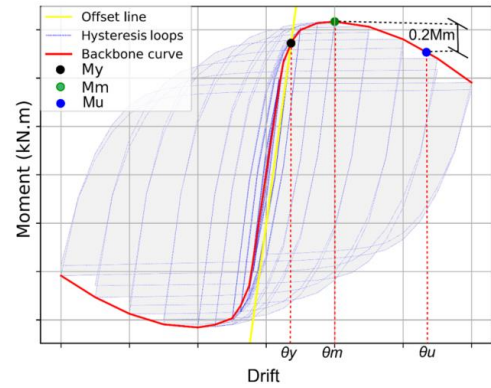
Figure 1 RWS connection geometry

Despite ample design guidelines for beams with web openings to enable stable inelastic action [12] [13], there are no specifications for designing RWS connections for seismic performance. For instance, FEMA 350, the Recommended Seismic Design Criteria for New Steel Moment Frame Buildings (SAC Joint Venture), emphasises prequalified connections that modify beam flanges or strengthen beam-column connections [3]. FEMA 350 notes that “an understanding of the utility of this system for enduring seismic actions is developing”, and RWS connections have not yet been prequalified up to this date [3]. This study contributes to addressing this knowledge gap by performing a parametric assessment of the behaviour of welded RWS connections, considering different geometric configurations. Although numerical simulations and experimental assessments have shown that RWS connections can withstand cyclic earthquake demands, these assessments only examined limited specimens and did not address overstrength or geometric constraints to enforce capacity design principles. Consequently, this study aims to perform a parametric analysis of RWS connections for low-rise buildings. They will be based on detailed finite element (FE) simulations of 300 specimens using IPE270, IPE 330, and IPE 400 beams with span-to-depth ratio of 8 subjected to prequalification testing protocols. The most widely used protocol is AISC’s [14], illustrated in Figure 2.



**Figure 2** AISC 341 loading protocol [14]

In this study, attention is drawn to the following characteristics. (I) Effective yielding occurs when structural damage causes non-reversible deformation. In the AISC guideline [14], it is the ordinate of the moment-rotation backbone hysteresis curve where the specified yielding moment ( $M_y$ ) of the connection is reached. (II) The maximum moment capacity ( $M_m$ ), the largest moment recorded during the imposed cyclic drift demands up to 6%. (III) The ultimate moment ( $M_u$ ), namely the moment and drift ordinates for which vertical load carrying capacity is compromised, or moment decreases by 20% from the maximum moment capacity, illustrated in Figure 3. Also, an outline of the backbone curve, i.e., the envelope of the typical hysteretic response of a moment connection, is shown in Figure 3.



**Figure 3** Force deformation to determine Yield and Peak points, and a 20% drop in strength method used to determine the Ultimate points along the cyclic backbone, following AISC 341

## 2 Methodology

The component method, widely used in research and specified in Eurocode 3 (EC3), can evaluate beam-to-column joint components [15]. The component method separately analyses the beam, column, end plate, bolts, and welds and their interactions for the entire connection. Design guidelines can be simplified by breaking down connections into their parts to understand their complex behaviour under different loading conditions [15]. Following the objectives of this study, beams with fixed ends are modelled, assuming a strong panel zone, to eliminate the flexibility of the column and panel zone and isolate the impact of reduced-web-section (RWS) geometry on cyclic response. This produces a conservative upper bound on plastic rotations and drift demands by eliminating any beneficial effect the panel zone could have on drift capacity. Researchers like Mojtabaei et al. [16], Ye et al. [17], and Horton et al. [18] [19] employed this approach. This ensures that changes in response are due to alterations in RWS geometry, providing clear insights into its behaviour under seismic cyclic loading. Later, where frame-level estimates are required, the additional rotation of the joint components can be superimposed using the EC3 component method [15], ensuring that the conclusions of the parametric study remain applicable once realistic joint flexibility is reintroduced. Moreover, the equivalent plastic strain index (PEEQ) is one of the most effective parameters for determining the likelihood of the connection assembly being brittle or ductile and has been used by researchers as a criterion of plastic strain demand [20] [21]. The PEEQ index values are used to determine the location of the plastic hinge under cyclic loading conditions [22]. An increase in the PEEQ index for a specific location indicates a higher possibility of tearing, damage, or deformation occurring in that region [22]. In addition, another study showed that high PEEQ values indicate a high likelihood of fracture due to tearing at a relatively low drift angle [23]. Complimentarily, FEMA 350 has highlighted that when the plastic hinge forms in the beam at the face of the column, this can result in large inelastic strain demands on the weld metal and surrounding heat-affected zones. These conditions can lead to brittle failure [3]. Hence, the PEEQ distributions are also considered to evaluate whether inelasticity initiates into the beam web within the designated protected zone away from the column face; therefore, protecting welds and fragile components from inelastic action. It is

expected that providing appropriate combinations of  $S$  and  $d_o$  should lead to low inelastic action at the beam-column interface, hence attaining low PEEQ values there.

### 3 Validation of FE models

Five full-scale sub-assemblies previously investigated experimentally were used to validate the proposed framework of this research using ABAQUS finite element models. Nia et al. [24] conducted three experiments representing small, medium, and large connections, abbreviated as DC-S, DC-M, and DC-L, respectively [24]. Two identical specimens of each connection were tested; for example, the DCS model was evaluated using two subassembly specimens, DC-S-1 and DC-S-2. These connections are sub-assembly connections with Welded Unreinforced Flange-Welded Web (WUF-W) configurations, representing short, intermediate, and tall steel frame buildings with seven, twelve, and twenty storeys, respectively. Details of these connections, along with test set-up for the experimental program are illustrated in Figure 4. Similarly, Davarpanah et al. studied the cyclic behaviour of fully welded RWS connections with elliptical web openings. Two identical specimens of each connection were tested, abbreviated as HE-RWS-1 [5] and HE-RWS-2 [23]. However, HE-RWS-1 is the subassembly tested under the standard AISC 341 loading protocol, whereas HE-RWS-2 is the subassembly tested under an extended AISC 341 protocol with two additional cycles at 7 % drift. The connection detail of HE-RWS is illustrated in Figure 5.

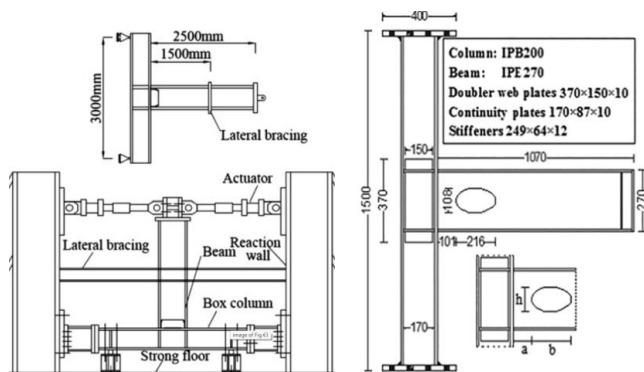


Figure 4 (Left) DC-M test set-up experiments [24]

Figure 5 (Right) HE-RWS test set-up experiments [5] [23]

All experiments followed the AISC/ANSI 341-16 loading protocol, except for HE-RWS-2 as mentioned earlier. The material used in all assemblies is ST37 steel (comparable to A36 steel). Further information on HE-RWS-1 and DC-S, DC-M and DC-L connections from experimental tests available in [5] [23] and [24].

Material properties of the HE-RWS-2 assembly differ from those of HE-RWS-1. These properties, shown in Figure 6, will be used in the parametric study of this research after validation.

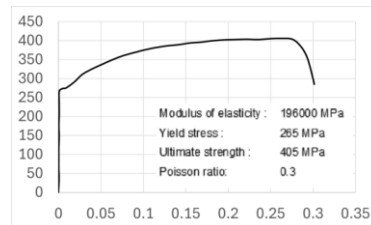


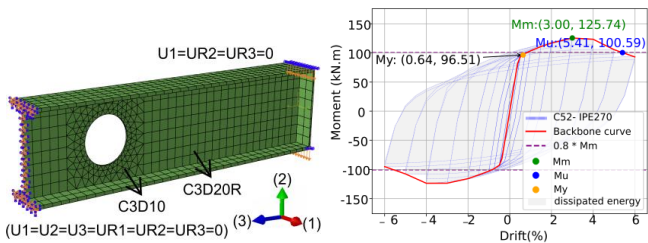
Figure 6 The stress-strain curve of the steel for HE-RWS-2

The true stress-strain relationship used in the finite element analyses of this study was derived by converting the experimental engineering stress-strain data. Equation (1) provides the conversion expressions from engineering stress-strain ( $\sigma_e$ - $\epsilon_e$ ) to true stress-strain ( $\sigma_t$ - $\epsilon_t$ ).

$$\sigma_t = \sigma_e(1 + \epsilon_e) \quad (1.1)$$

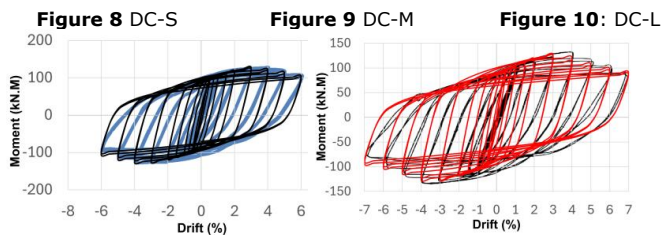
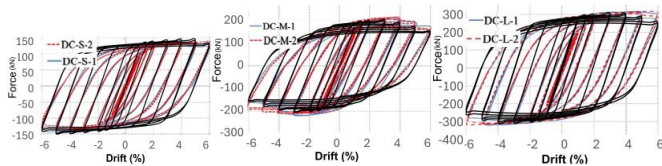
$$\epsilon_t = \ln(1 + \epsilon_e) \quad (1.2)$$

Additionally, the steel behaviour of the DCS, DCM, DCL, and HE-RWS-1 assemblies is modelled using a trilinear stress-strain relationship, while the HE-RWS-2 assembly is modelled using a multilinear stress-strain relationship. Combined isotropic-kinematic hardening was applied using Abaqus' COMBINED hardening model with two back stresses. The isotropic component was defined by the true stress-strain curve (Figure 6), and the kinematic component was automatically calibrated by Abaqus from the first unloading-reloading cycle (default DATA TYPE=HALF CYCLE), following Abaqus' standard combined hardening implementation (Abaqus Version 6.6 documentation, Section 18.2.2). Parameters of the material model were based on tensile testing of coupons for basic properties (yield stress and strain, strain hardening slope and ultimate strain), supplemented by values found in the literature to describe other behavioural phenomena like Bauschinger effects. Twenty-node reduced-integration brick elements (C3D20R) are used globally, whereas ten-node tetrahedral elements (C3D10), with finer mesh size (10mm), are selected around the web opening to enhance computational efficiency and accurately capture the local buckling behaviour of the beam web and flanges. Out-of-plane displacements are restricted by providing lateral support at boundaries. Sensitivity studies of the configuration and the density of the FE mesh size led to an optimal maximum global element size of 20mm. Tie constraints were used to model the welds, as the capacity design of welds in this study will ensure their elastic response [5]. Effects of manufacturing imperfections and residual stresses is included by imposing a deformation pattern in the cross-section resembling the first buckling mode of the web. It was found through eigenvalue analyses and was further scaled by  $t_w/200$  (where  $t_w$  is the web thickness), following practices adopted in related research [25] [26] [27]. Then models are subjected to the full 64 load-steps in accordance with AISC 341 [14], earlier illustrated in Figure 2, with incrementing  $\theta$  in steps of 0.01 radians recommended by FEMA 350 [3]. Also, full 3D nonlinear static analyses is effectuated using the Newton-Raphson algorithm as the numerical solver within ABAQUS. Figure 7 shows the general finite-element model used in this numerical study (left) and an example cyclic displacement diagram (right).



**Figure 7** (left): Typical FE model considered in this study. (Right): Example of cyclic displacement diagram

Figures 8–12 compare the hysteretic curves predicted by the finite-element model for the configuration shown in Figure 7 with experimental results for DC-S, DC-M, DC-L, HE-RWS-1, and HE-RWS-2. In the DC validations, the black curves, in HE-RWS-1 the blue curve, and HE-RWS-2 the red curve represent the current FEM results. Implemented as a single Python script in ABAQUS, the model successfully captures the moment–rotation hysteretic curves of five experiments covering a range of properties and geometries.



### 3.1 Parametric analysis with fixed beam support

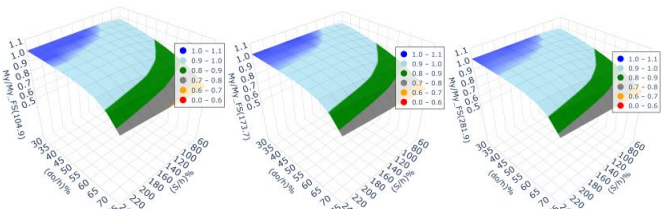
After FEM validations, the material properties of HE-RWS-2 mentioned earlier is adopted for the parametric study. In total, 300 models are analysed—100 with IPE270 beams, 100 with IPE330 beams and 100 with IPE400 beams—each incorporating isolated web-opening geometry parameters, earlier depicted in Figure 1,  $d_o$  ranging from  $0.30h$  to  $0.75h$ , along with ten  $S$  variations spanning  $0.60h$  to  $2.4h$  for each beam was carried out. span-to-depth ratio of all models in this parametric study is equivalent to 8. Table 1 lists all models along with their geometric parameters and designated names.

**Table 1** All combinations of ( $S$  and  $d_o$ ) for each IPE beam

| $(S/h) \%$ | $(d_o/h) \%$ |     |     |     |     |     |     |     |     |     |
|------------|--------------|-----|-----|-----|-----|-----|-----|-----|-----|-----|
|            | 30           | 35  | 40  | 45  | 50  | 55  | 60  | 65  | 70  | 75  |
| 60         | C00          | C10 | C20 | C30 | C40 | C50 | C60 | C70 | C80 | C90 |
| 80         | C01          | C11 | C21 | C31 | C41 | C51 | C61 | C71 | C81 | C91 |
| 100        | C02          | C12 | C22 | C32 | C42 | C52 | C62 | C72 | C82 | C92 |
| 120        | C03          | C13 | C23 | C33 | C43 | C53 | C63 | C73 | C83 | C93 |
| 140        | C04          | C14 | C24 | C34 | C44 | C54 | C64 | C74 | C84 | C94 |
| 160        | C05          | C15 | C25 | C35 | C45 | C55 | C65 | C75 | C85 | C95 |
| 180        | C06          | C16 | C26 | C36 | C46 | C56 | C66 | C76 | C86 | C96 |
| 200        | C07          | C17 | C27 | C37 | C47 | C57 | C67 | C77 | C87 | C97 |
| 220        | C08          | C18 | C28 | C38 | C48 | C58 | C68 | C78 | C88 | C98 |
| 240        | C09          | C19 | C29 | C39 | C49 | C59 | C69 | C79 | C89 | C99 |

### 3.2 Yield moment

Figure 13 shows the normalised yield moment ( $M_y/M_{y\_SolidBeam}$ ) for 100 combinations of  $S/h$  and  $d_o/h$  across IPE270, IPE330, and IPE400 sections. In all cases, the diameter-to-height ratio ( $d_o/h$ ) significantly affects the yield moment, while the effect of spacing ( $S/d_o$ ) is minimal. The yield moment drops by over 40% for the largest perforations (75% of the beam height), with similar trends observed across all three beam sizes. These results highlight  $d_o$  as the critical parameter influencing strength reduction.

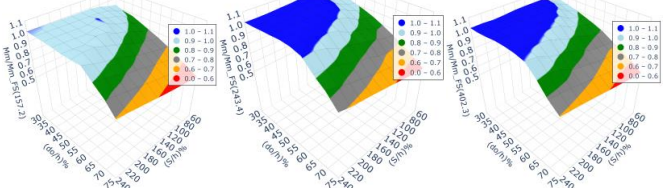


**Figure 13** Normalised  $M_y$ , left: IPE270, middle: IPE330, right: IPE400

### 3.3 Maximum moment

Figure 14 illustrates the normalised peak moment ( $M_m/M_{m\_SolidBeam}$ ) across varying  $S/h$  and  $d_o/h$  ratios for IPE270, IPE330, and IPE400 sections. The graphs show a consistent decline in peak moment as the perforation diameter increases, with reductions of up to 50% observed when  $d_o/h$  reaches 75%. In contrast, the influence of spacing ( $S/d_o$ ) remains relatively minor throughout the parametric range.

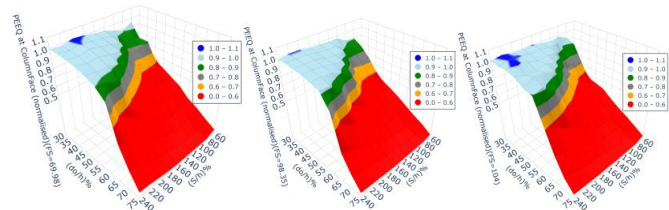
All three beams exhibit similar trends, with the surface sloping primarily along the  $d_o/h$  axis and slight curvature along the  $S/h$  direction. This consistency reaffirms that the perforation size is the dominant factor affecting moment capacity, while spacing has a secondary effect.



**Figure 14** Normalised  $M_m$ , left: IPE270, middle: IPE330, right: IPE400

### 3.4 PEEQ index at column face

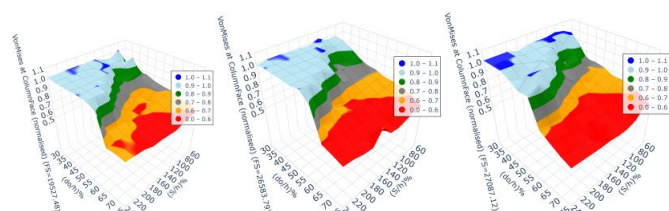
Figure 15 shows that the normalised PEEQ at the column face decreases with increasing  $d_o/h$ , indicating a significant reduction in plastic strain demand at the column face, more than 90%. Also, although the influence of  $S/h$  is less pronounced, the plots reveal an interaction between  $d_o$  and  $S$ , suggesting that neither parameter should be considered in isolation. This trend is consistently observed across all IPE beam sizes, indicating that to effectively minimise strain concentration at the column face,  $d_o$  and  $S$  should be selected simultaneously, balancing the opening size and spacing to optimise performance.



**Figure 15** Normalised PEEQ at column face, left: IPE270, middle: IPE330, right: IPE400

### 3.5 Von Mises Stress distribution at the column face

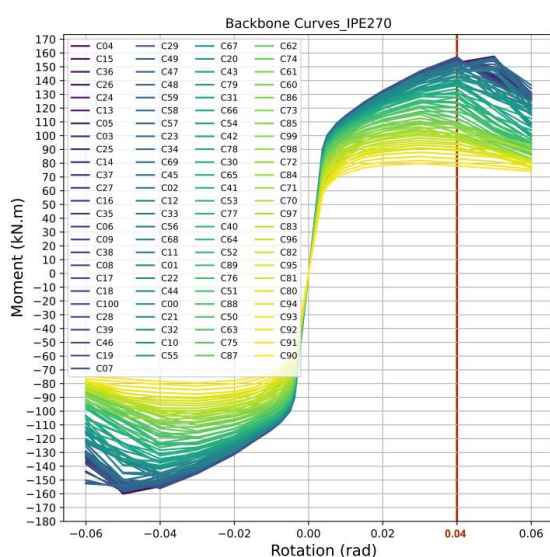
Figure 16 illustrates that von Mises stress at the column face decreases by more than 40% with increasing  $d_o/h$ , particularly for medium and large web opening sizes. Although the effect of  $S/h$  is modest, a clear interaction is observed, especially for medium-sized openings. This trend, consistent across all IPE beams, suggests that optimising both parameters together is essential to minimise stress concentration at the column face.



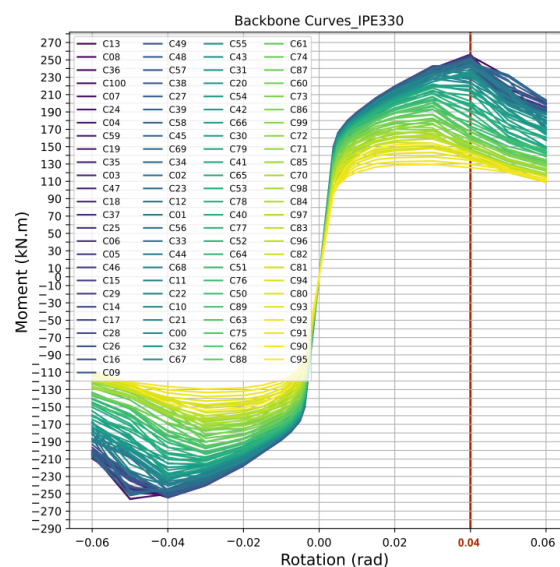
**Figure 16** Normalised von Mises distribution at column face, left: IPE270, middle: IPE330, right: IPE400

### 3.6 Backbone Curves

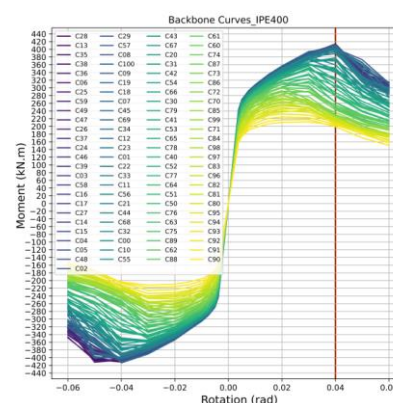
Figures 17–19 present the backbone curves of all 300 models. Most models exhibit the expected moment–rotation response: an initial elastic region, followed by yielding and post-peak degradation due to local buckling. In most cases, strength degradation begins after 4% rotation, indicating that these connections meet the AISC 341 drift demand criteria for prequalified connections with the span-to-depth ratio of 8 used in this study.



**Figure 17** Backbone Curves of 100 models (IPE270)



**Figure 18** Backbone Curves of 100 models (IPE330)



**Figure 19** Backbone Curves of 100 models (IPE400)

## 4 Concluding remarks

This study presents 300 FE of fixed-end beam models—100 with IPE270 beams, 100 with IPE330 beams, and 100 with IPE400 beams - each having a span-to-depth ratio of 7.92. The material used in both is ST37 steel (comparable to A36 steel). To investigate the effect of geometry parameters  $S$  and  $d_o$  on the cyclic performance of the RWS connections  $M_y$ ,  $M_m$ ,  $M_u$ , local buckling PEEQ index and von Mises distribution at the column face. The summary of the results has been presented below:

The parametric investigation confirms that the web perforation diameter ( $d_o/h$ ) is the primary factor influencing both strength degradation and localised plasticity in RWS connections, while spacing ( $S/h$ ) plays a secondary role. However, there is a clear interaction between  $d_o$  and  $S$ ; although the effect of spacing is generally modest, it can influence the structural response, particularly for medium-sized web openings. Moreover, the results demonstrate that, with the appropriate selection of  $d_o/h$  and  $S/h$ , it is possible to reduce stress and strain concentrations at the column face without significantly compromising load-bearing capacity. This is accomplished while still activating the RWS mechanism, which is capable of moving high stresses and strains away from the joints and into protected zones that are located in the vicinity of web openings. Moreover, most

configurations satisfy the AISC 341 drift criteria for beams where yielding is inhibited outside the protected zone. Also, the consistent behaviour observed across IPE270, IPE330, and IPE400 sections suggests that RWS connections exhibit predictable behaviours; however, further studies are essential to validate this trend across a broader range of configurations.

Overall, this study demonstrates that appropriately selected geometric parameters of RWS connections offer a promising solution for seismic retrofitting of structures. Furthermore, the findings support the potential for RWS connections to achieve prequalification status, provided that the geometric parameters are carefully optimised in tandem, due to their interactive effects.

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