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OPTIMUM DRILLED FLANGE MOMENT RESISTING CONNECTIONS FOR SEISMIC REGIONS

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ABSTRACT

Extensive damage in welded unreinforced flange (WUF) connections in previous earthquakes has led to the idea of using reduced beam section (RBS) connections to prevent brittle failure modes in welded joints. Using a similar concept, drilled flange (DF) moment resisting connections are established by a series of holes drilling on the top and the bottom flanges of the beam to create an intentional weak area to shift nonlinear deformations. DF connections are very easy-to-construct and they can also prevent the premature local buckling modes in the reduced section of RBS connections. This study aims to improve the performance of DF connections to make them viable alternatives to RBS connections for ductile steel frames in seismic regions. A wide range of experimentally validated non-linear FE models are used to investigate the effects of different design parameters such as drilled flange hole locations, hole configurations, panel zone shear strength ratio and doubler plate thickness. The results indicate that there is an optimum location and configuration for the drilled flange holes, which can reduce by up to 40% the maximum Equivalent Plastic Strain and Rupture Index of DF connections. It is shown that using strong panel zones can also improve the seismic performance of DF connections by reducing stress concentrations at the CJP groove weld lines. The results of this study are used to develop optimum design solutions for DF connections, which should prove useful in practical applications.

Keywords: Drilled Flange Connection; Reduced Beam Section Connection; Unreinforced Flange Connection; Panel Zone; Shear Strength

1. INTRODUCTION

Extensive failure in welded unreinforced flange (WUF) connections in steel moment resisting frames (MRFs) during the 1994 Northridge Earthquake highlighted the poor seismic performance of these connections [1]. Several studies on the failures observed in the pre-Northridge connections concluded that high three-axial stresses at the complete joint penetration welds at beam flanges resulted in a flange break off close to the weld lines before any significant yielding and plastic deformation could develop [2, 3]. El Tawil et al. [4] analytically studied the effects of local geometric details and yield-to-ultimate stress ratio on the inelastic behaviour of pre-Northridge connections. They highlighted the unfavourable effects of using steel material with high yield-to-ultimate stress ratio and enlarging the size of access holes that are used to facilitate welding. In a related study, Mao et al. [5] recommended using a groove welded beam web attachment with supplemental fillet welds along the edges of shear tabs and using a modified weld access hole geometry to improve the performance of pre-Northridge welded moment connections.

Stojadinovic et al. [6] conducted a series of parametric tests on pre-Northridge connections and showed that earthquake-resistant design of WUF connections should incorporate both the weld fracture and flange overstress mitigation measures, which can be achieved, for example, by changing the welding process and connection configuration. In a similar study, Ricles et al. [7-8] demonstrated that the dominant failure mode of pre-Northridge connections is brittle fracture that is developed in the elastic range of response due to flaws in the low toughness weld metal and poor geometric conditions. Han et al. [9] studied the cyclic behaviour of post-Northridge Welded Unreinforced Flange-Bolted web (WUF-B) connections. Their experimental tests showed that the WUF-B connections with a panel zone strength ratio ranging from 0.9 to 1.6 can provide a drift ratio capacity exceeding 0.02, which is suitable for satisfactory performance of the connections in Intermediate Moment Frames.

Reduced beam section (RBS) connections were developed to prevent premature brittle failure modes observed in typical WUF connections due to high stress concentrations at the connection edge [10]. RBS connections, in general, use a reduced beam flange width at a short distance from the column, and thus create a fuse in the connection to reduce stress concentrations at the column face. Reorder [11] studies showed that this type of connection is capable of providing good seismic performance with high plastic rotational capacity. However, an appropriate balance should be provided between the controlling yield mechanism and the critical failure mode. Uang et al. [12] performed six full scale beam-to-column connection tests including RBS connections with concrete slab. The results of their study indicated that using reduced beam only in the bottom flange could not prevent brittle fracture in the groove weld of the top flange, and the presence of a concrete slab or removing steel backing only slightly improved the cyclic performance of the connections.

Chen and Chao [13] studied the effect of composite action on the ductility performance of steel moment connections with reduced beam sections through a series of large size experimental studies of beam-to-column subassemblies with floor slabs. They showed that the ratio of positive moment to negative moment strength may be as high as 1.18, which is mainly from the contribution of floor slabs. Scott et al. [14] experimentally investigated the performance of eight radius cut RBS moment connections under a standard quasi-static cyclic load pattern. They concluded that inclusion of a composite slab can stabilize the beams against lateral torsional buckling without an obvious increase in the strains in the bottom beam flange. It was also shown that welding the beam web to the column flange can decrease the likelihood of weld fracture in the RBS connections. In a similar study, Lee et al. [15] have conducted eight full scale tests on RBS steel moment connections to investigate the effect of web connection type (bolted versus welded) and panel zone strength on the seismic performance of steel moment frames. The results showed that both strong and medium panel zone specimens with a welded web connection were able to provide satisfactory plastic rotation capacity for special moment resisting frames and achieve the storey drift angle of at least 0.04 radians. Moslehi Tabar and Deylami [16] performed an analytical study to investigate the effect of panel zone shear strength on the performance of RBS connections. The results of their study indicated that partial shear yielding in panel zone can improve the hysteretic response of specimens by avoiding premature instability in beams.

Pachoumis et al. [17] investigated the performance of RBS moment connections with radius cut subjected to cyclic loading and presented a theoretical model, which is validated by experimental results. In a more recent study, Ghassemieh and Kiani [18] have analytically studied the performance of RBS connections with semi rigid connections and flexible panel zone in multi-storey structures. Their study showed that overlooking the flexibility of beam-to-column joints in the seismic design of RBS connections may lead to unsatisfactory performance under strong earthquakes. Based on the concept of RBS moment connections, Chou and Wu [19] developed a new moment connection using steel reduced flange plates (RFPs), which acts as a structural fuse to eliminate weld fractures and beam buckling. Their experimental and analytical results showed that RFP connections have satisfactory performance and can reach 4% inter-storey drift without considerable strength degradation.

Although the RBS connections in general have good seismic performance and can provide high ductility levels, they usually suffer from an increased stress concentration at the beam web and a significant decrease in the moment capacity and stiffness of the connections. To address this issue, Farrokhi et al. [20] proposed a reduced plate section connection by drilling holes at cover plates to create an intentional weak point. The drilled flange (DF) connections can shift the stress concentrations from the connection face and, therefore, eliminate unfavourable local beam failure modes that are observed in conventional RBS connections. Farrokhi et al. [20] study showed that DF connections can improve the ductility capacity of the typical RBS and WUF connections. Moreover, the performance of DF connections seems to be less dependent on the weld root quality, since the major nonlinear mechanism takes place adjacent to the drilled holes. In a follow up study, Vetr et al. [21] and Vetr and Haddad [22] conducted a series of experimental tests to investigate the performance of DF beam-column connections under non-linear cyclic loading. Their test specimens consisted of eight exterior DF connections with two different hole configurations and panel zone shear capacity. The DF connections in their study, in general, demonstrated a sufficient rotational stiffness and an excellent rotational ductility.

This paper aims to optimise the performance of DF connections by identifying the best hole location and configuration, doubler plate thickness and beam to panel zone shear strength ratios. To demonstrate the efficiency of the optimum designed DF connections, their maximum Equivalent Plastic Strain (EPEQ), Triaxiality Ratio (TR) and Rupture Index (RI) are compared with typical WUF and RBS connections. The results of this study are used to provide practical design recommendations to improve the performance of DF connections as viable alternatives to RBS connections in seismic regions.

2. REFERENCE EXPERIMENTAL TESTS

In order to develop a better understanding of the cyclic behaviour of typical steel moment resisting connections, three WUF, RBS and DF test specimens are considered from previous experimental studies [21-24] as shown in Fig 1. The detailed properties of the selected test specimens are presented in Table 1.

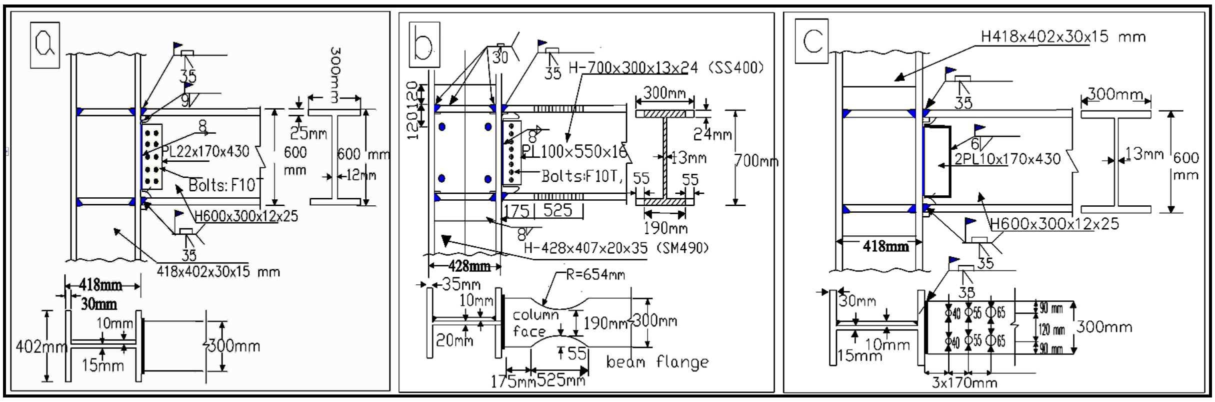

Fig 1. Schematic view of the selected test specimens: (a) WUF connection [23], (b) RBS connection [24], and (c) DF connection [21, 22]

Table 1. Detailed properties of the reference experimental tests

<table>
<tr><th rowspan="3">Test Specimen</th><th rowspan="3">Beam and Column Sections</th><th rowspan="3">Doublers Plate Thickness</th><th colspan="2">Material properties (Mpa) F_y</th><th rowspan="2">Weld Property</th><th rowspan="3" colspan="3">Connection Type and Specification (see Fig 1)</th></tr>
<tr><th>Beam</th><th>Column</th></tr>
<tr><th>Flange/ Web</th><th>Flange/ Web</th><th>d/e/f</th></tr>
<tr><td rowspan="2">WUF
(S6 [23])</td><td>H600×300×12×25</td><td rowspan="2">10mm</td><td rowspan="2">304/
455</td><td rowspan="2">343/
512</td><td rowspan="2">E7018
35/20</td><td rowspan="2" colspan="3">---------------</td></tr>
<tr><td>H418×402×30×15</td></tr>
<tr><td rowspan="2">RBS
(DB700-SW [24])</td><td>H700×300×13×24</td><td rowspan="2">10mm</td><td rowspan="2">400/
450</td><td rowspan="2">345/
450</td><td rowspan="2">E70T7
35/25</td><td>a</td><td>b</td><td>c</td></tr>
<tr><td>H428×407×20×35</td><td>175</td><td>525</td><td>55</td></tr>
<tr><td rowspan="2">DF
(RDH1 [22])</td><td>H600×300×12×25</td><td rowspan="2">10mm</td><td rowspan="2">310/
420</td><td rowspan="2">367/
537</td><td rowspan="2">E7018
35/20</td><td>d1</td><td>d2</td><td>d3</td></tr>
<tr><td>H418×402×30×15</td><td>40</td><td>55</td><td>65</td></tr>
<tr><td colspan="9">a, b, c refer to the dimensions of the reduced beam section in the RBS connection (see Fig1-b)
d1, d2, d3 are hole diameters in the DF connection (see Fig 1-c)
d/e/f are electrode type, bevel angle (degree) and weld root diameter (mm), respectively.</td></tr>
</table>

The selected WUF connection (Fig 1-a) is the test specimen S6 in Chen et al. study [23]. Their experimental results showed that the fracture of this connection initiated from the intersection between the weld access hole and the complete penetration weld at storey drift angle of 4%. This fracture line was then propagated towards the flange edges. The test was terminated due to beam fracture close to CJP groove welding in the heat affected zone area of the beam. The cyclic response of this connection is shown in Fig 2-a. Based on the results, no strength degradation is observed in the cyclic behaviour of this connection until failure point.

Fig 1- b shows the schematic view of the RBS connection in Lee et al. [24] experimental study (DB700-SW specimen), which is designed to have a strong panel zone according to AISC Seismic Provision [25]. The cyclic behaviour of this test specimen is shown in Fig 2-c under ATC 24 loading protocol [26]. The experimental results indicate that this RBS connection exhibited good rotation capacity up to 5% storey drift with no significant strength degradation. The failure of this connection was due to the web and flange local buckling in the reduced flange area.

The seismic performance of DF connections has been experimentally investigated by Vetr et al. [21] and Vetr and Haddad [22]. Table 2 shows the summary of their experimental results. It is shown that the sub assemblages with medium and strong panel zones exhibited maximum storey drift angles more than 0.045 radians. The dominant failure mode of these types of DF connections was always due to ductile rupture at holes edge on the beam top or bottom flange.

Table 2. Summary of the experimental tests on DF connections [21, 22]

Test Specimens	Hole Configuration	Column Size	Beam Size	Panel Zone Shear Strength Ratio	Panel Zone	Max Drift Ratio	Rupture Mode
RBS-DHA1	19-25-30	IPE 220	IPE 270	0.76	Strong	0.050	Ductile rupture at holes edge on beam top flange
RBS-DHA2	19-25-30	IPE 220	IPE 270	0.88	Medium	0.050	Ductile rupture at holes edge on beam top flange
RBS-DHA3	30-25-19	IPE 220	IPE 270	0.61	Strong	0.045	Ductile rupture at holes edge on beam bottom flange
RBS-DHA4	30-25-19	IPE 220	IPE 270	0.98	Weak	0.040	Beam to column weld connection fracture
RDH1	40-55-65	H 418	H 600	0.65	Strong	0.050	Ductile rupture at holes edge on beam bottom flange
RDH2	40-55-65	H 418	H 600	0.89	Medium	0.050	Ductile rupture at holes edge on beam bottom flange
RDH3	40-55-65	H 418	H 600	0.38	Strong	0.050	Ductile rupture at holes edge on beam bottom flange
RDH4	40-55-65	H 418	H 600	1.15	Weak	0.040	Beam to column weld connection fracture

The drilled hole configuration of the test specimen RDH1 (the reference DF connection) consisted of three rows of twin holes as shown in Fig 1-c. The diameter of the holes varied from 40 to 65 mm by increasing the distance between the centre of the holes and the column face. It is shown in Fig 2-e that this connection exhibited a stable hysteretic behaviour up to 5% storey drift. Based on Vetr and Haddad [22] experimental observations, yielding around the drilled holes on the beam flange started at storey drift ratio of 0.015 rad. This was followed by the local yielding of the web at storey drift ratio of 0.025 rad. A ductile rupture started at storey drift ratio of 4%, located on the edge of one of the drilled holes in the bottom beam flange. The crack was then extended to the beam bottom flange edge at storey drift ratio of 5%, and the experiment was terminated at this stage. Fig. 3 shows the failure mode of this connection, which is the typical failure mode of DF connections with medium and strong panel zone in Vetr and Haddad [22] experimental tests (see Table 2). The flaking off the white washed area in this figure can represent the pattern of the yielding lines. Although some local buckling is observed around the drilled holes in the beam top flange, it is evident in Fig 3 that the dominant failure mode of this connection was due to the rupture of the beam bottom flange at the edge of the drilled holes. The hysteretic behaviour shown in Fig 2-e also indicates that this test specimen exhibited around 18% strength degradation at storey drift angle of 4% and, therefore, can be qualified according to AISC seismic provisions [25].

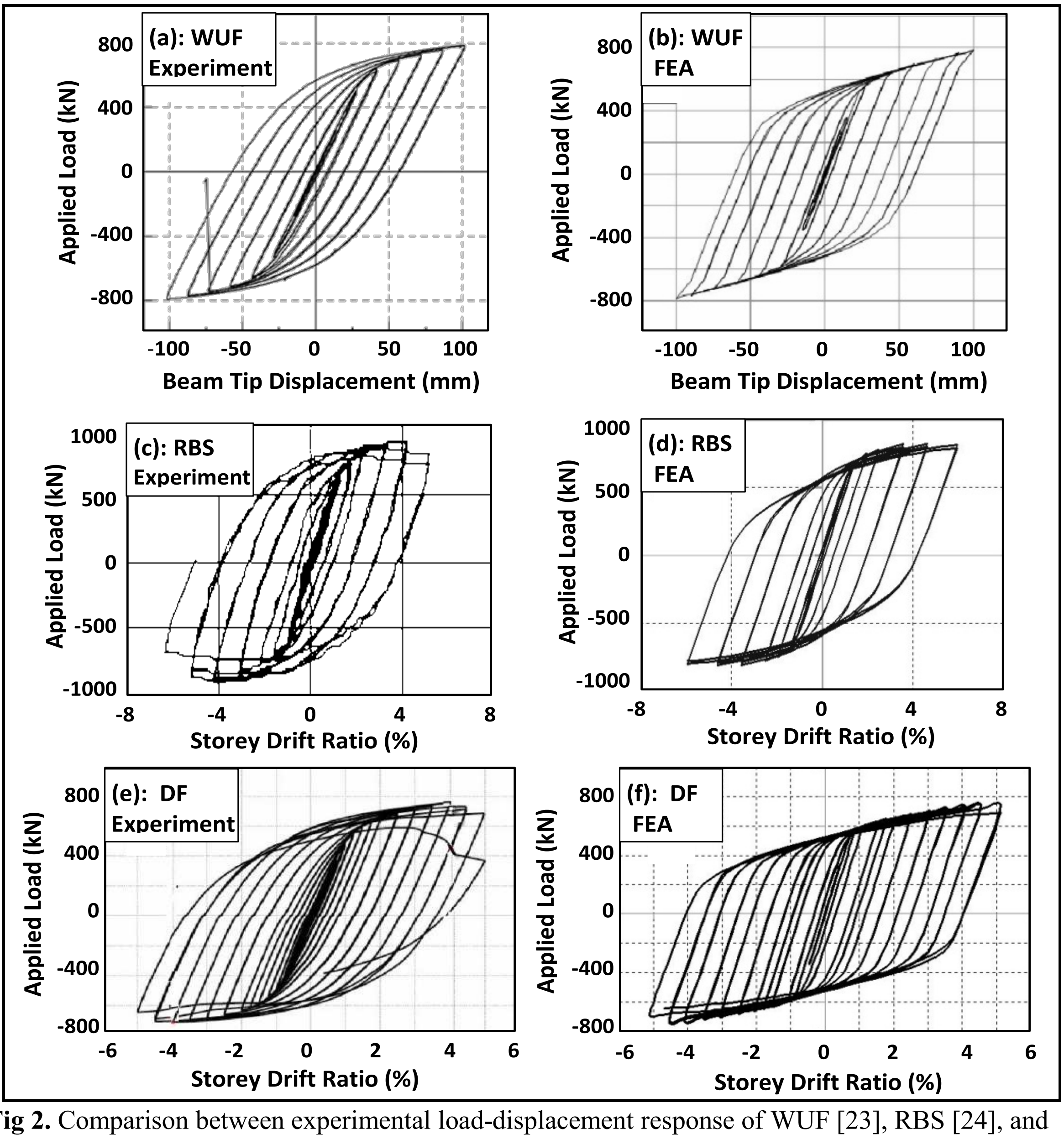

Fig 2. Comparison between experimental load-displacement response of WUF [23], RBS [24], and DF [22] test specimens (left) with the results of the nonlinear FEA simulations in this study (right)

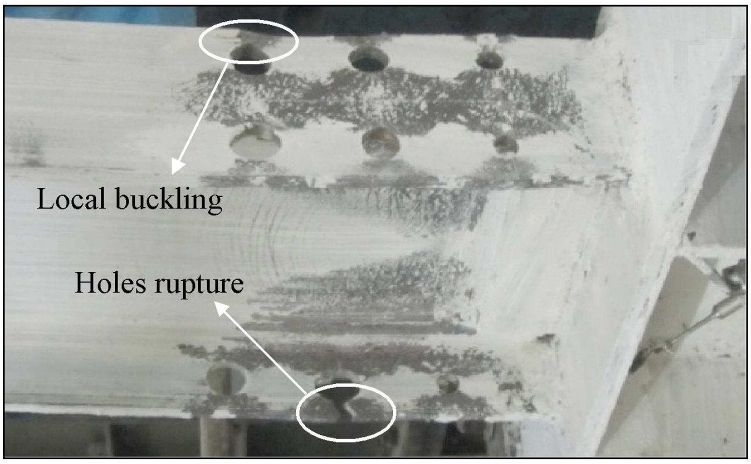

Fig 3. Typical failure mode of DF connections with medium and strong panel zone in Vetr and Haddad's [22] experimental tests (RDH1 specimen)

3. FINITE ELEMENT MODELLING APPROACH

To study the seismic behaviour of the selected beam-to-column moment connections, nonlinear finite element (FE) analyses were carried out using ANSYS software [27]. Fig. 4 shows the FE model of the WUF, RBS and DF test specimens used in this study.

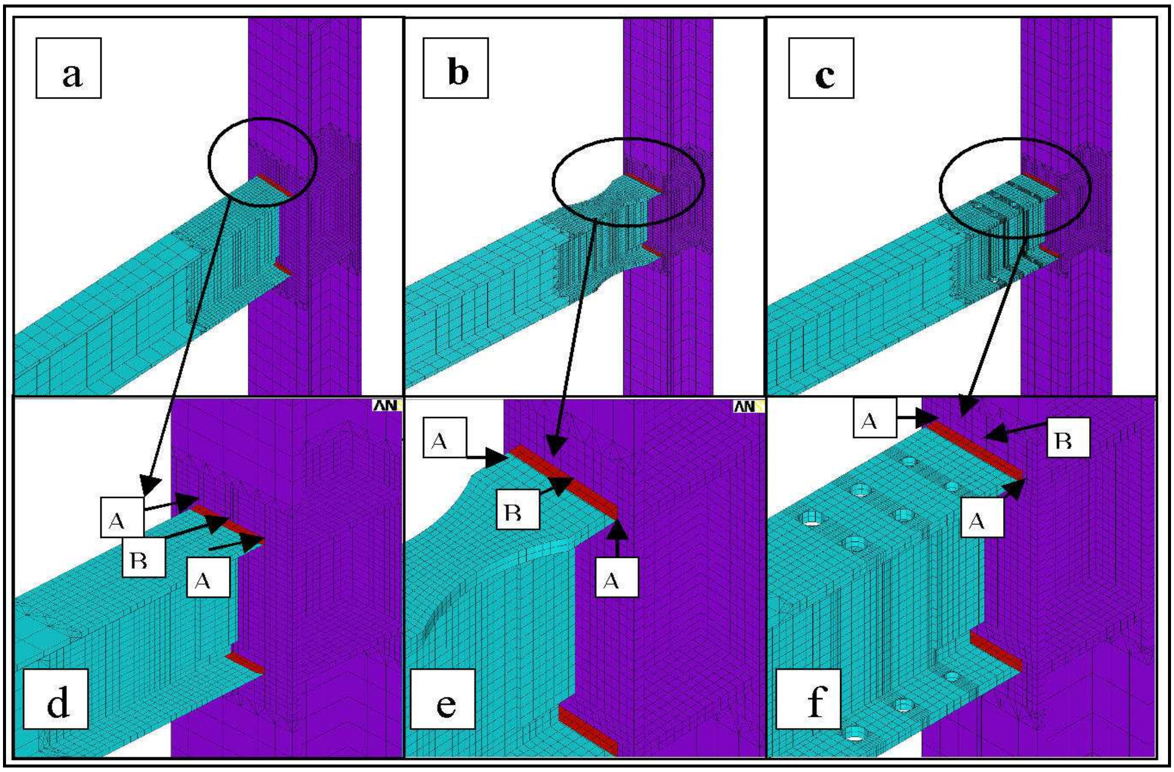

Fig 4. Analytical models of the reference connections and their critical points with maximum Rupture Index (a): WUF connection, (b): RBS connection, and (c): DF connection

Steel elements and fillet welds were modelled using a 3D solid element (SOLID45), which is suitable for nonlinear large displacement problems [27]. The material properties used in the analyses were based on the measured stress–strain relationships obtained from the reference experimental tests. FE models were generated using non-uniform meshes with local refinement in the regions with high stress concentration and holes. Bolts and shear tabs were not modelled in the FE model of the WUF and RBS test specimens, since no slippage was observed between the shear tab and the beam web in these connections [23-24]. The Von-Mises yielding criterion and multi-linear kinematic hardening plastic model [27] were used to model the plasticity and cyclic inelastic behaviour of steel material, respectively. The beam flange and web nonlinear buckling behavior as well as local kinking of the column flanges were taken into account in the analysis by applying initial imperfections consistent with the first buckling mode shape of the test specimens. It is shown in the following sections that the detailed FE models developed in this study could accurately simulate the nonlinear cyclic behaviour of the WUF, RBS and DF test specimens.

4. PERFORMANCE PARAMETERS

To evaluate the fracture potential of the connections, the following Rupture Index (RI) is adopted in this study:

$$RI = \frac{\varepsilon_{eqv}^{pl} / \varepsilon_y}{\exp\left(-1.5.\dfrac{\sigma_m}{\sigma_{eff}}\right)} \tag{1}$$

where ε_{eqv}^{pl}, ε_y, σ_m and σ_{eff} are the equivalent plastic strain, yield strain, hydrostatic stress, and the equivalent stress (also known as Von-Mises stress), respectively. Since the loading protocol used for the analytical studies was cyclic, the larger value of RI in compression and tension was considered as the rupture index for each load cycle (or storey drift angle). In general, locations with higher values of RI have a greater potential for fracture and failure [8, 28, 29]. In the presence of a crack or defect, a large tensile hydrostatic stress can also produce large stress intensity factors at the tip of the crack and increase the likelihood of brittle fracture [29].

The ratio of the hydrostatic stress to the von-Mises stress (i.e. σ_m / σ_{eff}), which appears in the denominator of Equation (1), is called triaxiality ratio (TR). It has been reported by El-

Tawil et al. [30] that TR values less than −1.5 can cause brittle fracture, whereas values between −0.75 and −1.5 usually result in a large reduction in the rupture strain of the metal.

Studies by El-Tawil et al. [30] and Ferreira et al. [31] suggested that crack initiation could be predicted with reasonable accuracy by defining a threshold value for the Equivalent Plastic Strains (EPEQ). The EPEQ for a given stress–strain state can be calculated using the following equation:

$$\text{EPEQ}= \varepsilon_{eqv}^{pl} = \frac{1}{\sqrt{2}(1+\upsilon^{/})}\left[\left(\varepsilon_x^{pl}-\varepsilon_y^{pl}\right)^2+\left(\varepsilon_y^{pl}-\varepsilon_z^{pl}\right)^2+\left(\varepsilon_z^{pl}-\varepsilon_z^{pl}\right)^2+\frac{2}{3}\left(\gamma_{xy}^{pl^2}+\gamma_{yz}^{pl^2}+\gamma_{zx}^{pl^2}\right)\right]^{\frac{1}{2}} \quad (2)$$

where ε_x^{pl}, ε_y^{pl} and ε_z^{pl} are plastic strains, γ_{xy}^{pl}, γ_{yz}^{pl} and γ_{zx}^{pl} are plastic shear strains, and $\upsilon^{/}$ is the effective Poisson’s ratio. The EPEQ is a measure of the local inelastic strain demands, which can be useful in evaluating and comparing the performance of different connection configurations.

5. EXPERIMENTAL VALIDATION OF THE ANALYTICAL MODELS

The experimental hysteretic response of the three selected WUF, RBS and DF connections (see Table 1) and the test observations are used to validate the accuracy of the analytical models described in Section 3. It is shown in Fig. 2 that the FE results, in general, compare well with the experimental load-displacement response of the WUF, RBS, and DF test specimens. Especially the results indicate that the maximum strength determined from the inelastic FE analysis correlated very well with the experimental readings for all test specimens. The FE models could also simulate the strength degradations in the connections. The only exception was the strain degradation observed in the last cycle of the DF experimental test (at 5% storey drift), which was mainly due to effects of the wide cracks occurred in the bottom beam flange as explained in Section 2.

Based on the results of the FE analyses, the Equivalent Plastic Strain EPEQ distribution in the selected connections was calculated. Fig. 5 compares the EPEQ distribution and flaking off the white washed area on RBS and DF connections at storey drift angle of 0.05 radians. It is shown that the EPEQ contours and yield distribution areas are in very good agreement with the experimental observations. However, it should be mentioned that the surface stress distribution may vary within the thickness of the steel. In this study, the comparison has been made based on the surface strains to demonstrate the highest magnitude of the local stress and strain fields. It is also shown in Fig. 5 that the maximum equivalent plastic strains around the drilled holes in DF connections were around 0.11, while the corresponding values for RBS connections reached 0.08 (i.e. 35% less). The effects of drilled holes on the stress and strain distributions will be discussed in more detail in the following sections.

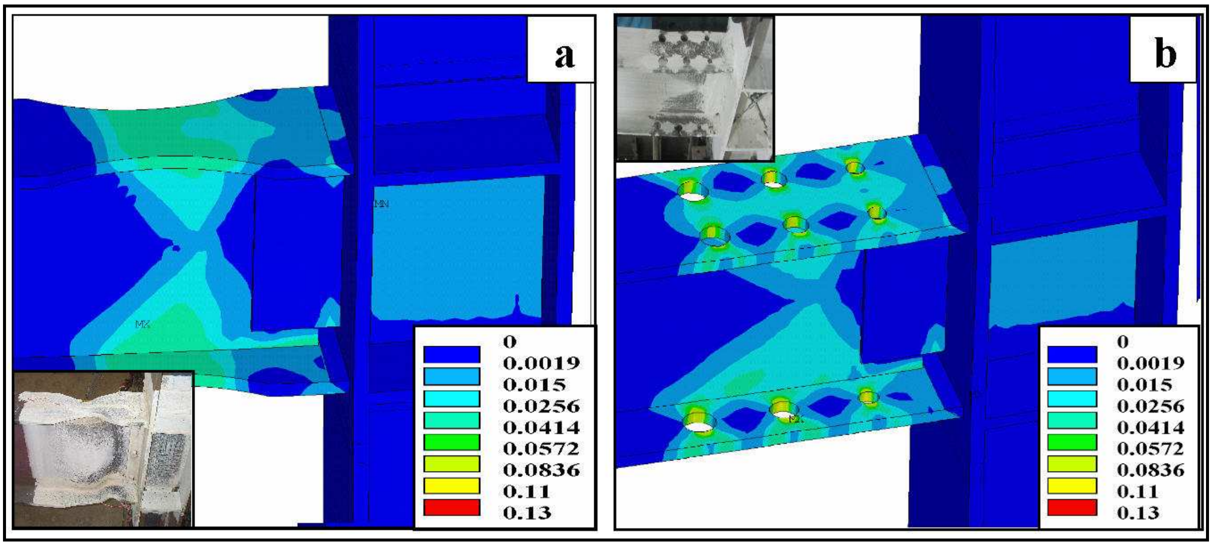

Fig 5. Comparison between Equivalent Plastic Strain distribution and flaking off the white washed area on (a) RBS and (b) DF connections at storey drift angle of 0.05 radians

The FE results and experimental observations presented in Fig. 5 show that the failure of the RBS connection was due to local buckling of the beam web and flanges in the reduced region of the beam, while using the DF connection could delay this premature failure mode. The maximum equivalent plastic strains in the RBS and DF connections at storey drift angle of 0.05 radians were 0.042 and 0.13, respectively, while the corresponding values at the CJP groove weld region reached 0.028 and 0.043. The maximum plastic strains in the DF specimen occurred in the drilled flange area of the connection and in the vicinity of the drilled holes (see Fig. 5 (b)), which is in complete agreement with the failure mode of this specimen as described in Section 2.

These results in general demonstrate that the detailed FE models could adequately simulate the non-linear behaviour and the failure mechanism of the selected moment resisting connections.

6- MORE EFFICIENT DESIGN OF DF CONNECTIONS

The previously validated FE analysis techniques are used to investigate the effects of panel zone shear strength ratio, doubler plate thickness and drilled hole location and configuration on the seismic performance of DF connections compared to their WUF and RBS counterparts and obtain the best design solutions.

6-1- Panel Zone Shear Strength

According to FEMA-355D [32], the required doubler plate thickness in the panel zone is determined based on a balance condition between the flexural yield strength of the beam and the panel zone shear strength. For a connection where flexural yielding develops at the face of the column, the shear demand generated by the flexural yield of the beam can be defined by the following expression [32]:

$$V_{PZMy} = \frac{\sum M_y}{d_b}\left(\frac{L}{L-d_c-2l_p}\right)\left(\frac{h-d_b}{h}\right) \tag{3}$$

where V_{PZMy} is the panel zone shear force associated with the initiation of flexural yielding, L is the beam span length, h is the column total height, d_b is the depth of the beam section, d_c is the height of the column section, M_y is the yield moment capacity of the beam and l_p is the cover plate length. The panel zone shear yield force can be calculated using the following equation [32]:

$$V_y = 0.6F_{yc}.d_c.t_{wc} \tag{4}$$

where F_{yc} is the column web yield stress and t_{wc} is the column web thickness (including the doubler plate thickness). Therefore, for a given connection, the panel zone shear yield force V_y can be easily controlled by the thickness of the doubler plate.

6-2- Developed FE Models

Four series of DF connections with different drilled hole configurations (Dh1, Dh2, Dh3 and Dh4) as well as two series of WUF and RBS connections, mainly for comparison purposes, are developed as shown in Fig. 6. Each series contains five connections with similar geometry but different panel zone shear strength ratios and doubler plate thicknesses (30 FE models in total). The specifications of these connections are summarised in Table 3.

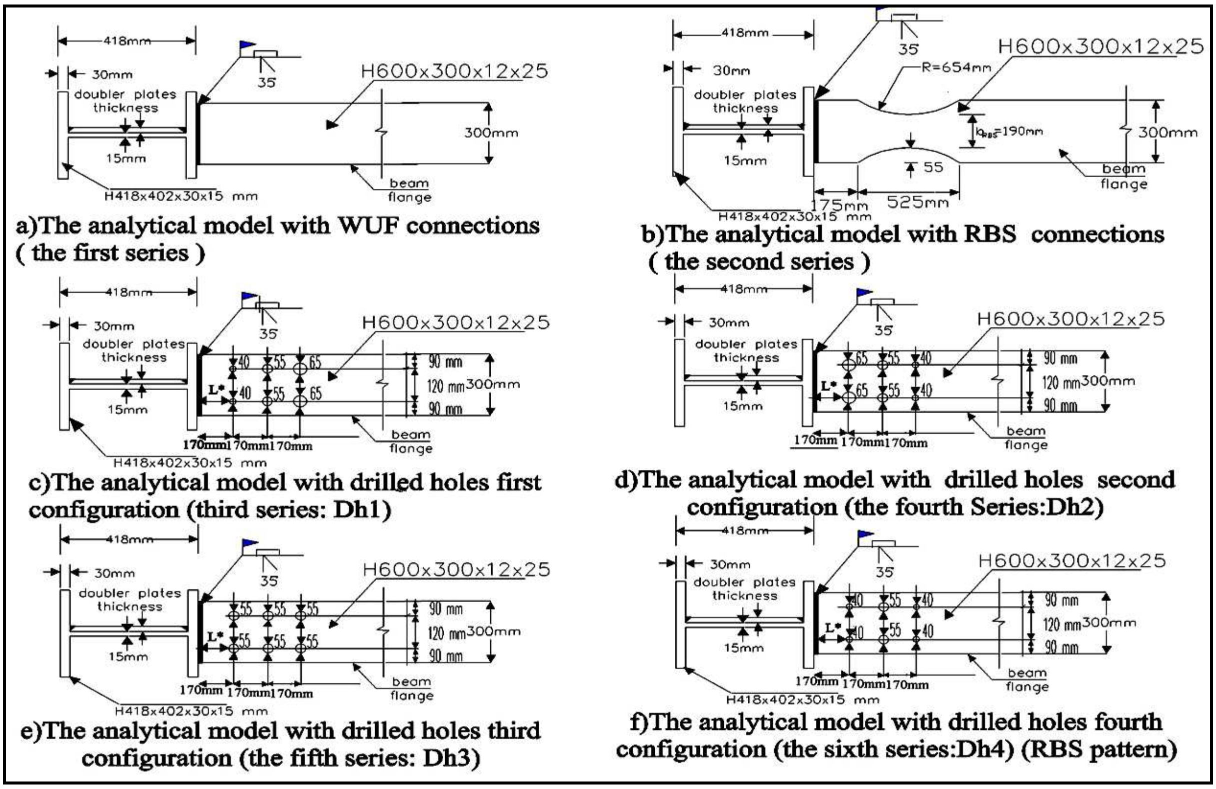

Fig 6. Schematic view of the six series of selected beam-to-column moment resisting connections

Three different diameters 65, 55 and 40 mm were considered for the flange drilled holes in Dh1 to Dh4 series, which were equal to 22% 18% and 17% of the flange width. The centre-to-centre distance of the holes was 170 mm similar to the experimental test RDH1 [22] as explained in Section 2. As shown in Fig. 6, the distribution of the drilled hole diameters from the column face were (40mm, 55mm, 65mm), (65mm, 55mm, 40mm), (55mm, 55mm, 55mm) and (40mm, 55mm, 40mm) in Dh1 to Dh4 models, respectively.

Table 3. Specification of the selected moment resisting connection series

Series	**Type of connection**		**Panel zone shear strength ratio (V_{PZMy}/V_y)**				
			0.7	**0.8**	**0.9**	**1**	**1.1**
1	WUF	Naming conventions	WUF-0.7	WUF-0.8	WUF-0.9	WUF-1	WUF-1.1
		Doubler plate thickness	2.5	2	1.6	1.2	1
2	RBS	Naming conventions	RBS-0.7	RBS-0.8	RBS-0.9	RBS-1	RBS-1.1
		Doubler plate thickness	1.05	0.75	0.5	0.25	0.15
3	Dh1	Naming conventions	Dh1-0.7	Dh1-0.8	Dh1-0.9	Dh1-1	Dh1-1.1
		Doubler plate thickness	2.2	1.8	1.4	1	0.8
4	Dh2	Naming conventions	Dh2-0.7	Dh2-0.8	Dh-0.9	Dh-1	Dh-1.1
		Doubler plates thickness	2.2	1.8	1.4	1	0.8
5	Dh3	Naming conventions	Dh3-0.7	Dh3-0.8	Dh-0.9	Dh-1	Dh-1.1
		Doubler plate thickness	2.35	1.9	1.5	1.1	0.9
6	Dh4	Naming conventions	Dh4-0.7	Dh4-0.8	Dh4-0.9	Dh4-1	Dh4-1.1
		Doubler plate thickness	2.25	1.8	1.4	1.1	0.9

Based on FEMA-355D [32], the panel zone shear strength ratio V_{PZMy}/V_y between 0.6 and 0.9 provides a safe margin to exhibit adequate panel zone energy dissipation capacity and prevent excessive deformation and stress concentrations in moment-resisting connections. The doubler plate thickness used in the reference DF experimental work (described in Section 2) leads to $V_{PZMy}/V_y \cong 0.7$, which is close to the lower threshold. In this study, different doubler plate thicknesses are used in the non-linear FE models to obtain V_{PZMy}/V_y ratios of 0.7, 0.8, 0.9, 1.0 and 1.1 as summarised in Table 3. The name of the connections consists of two parts, which indicates the connection type and the design panel zone shear strength ratio (V_{PZMy}/V_y). For instance, RBS-0.8 model is a reduced beam section connection with $V_{PZMy}/V_y = 0.8$. In the following sections, the seismic performance of the developed FE models is evaluated under the AISC [25] cyclic loading shown in Fig. 7.

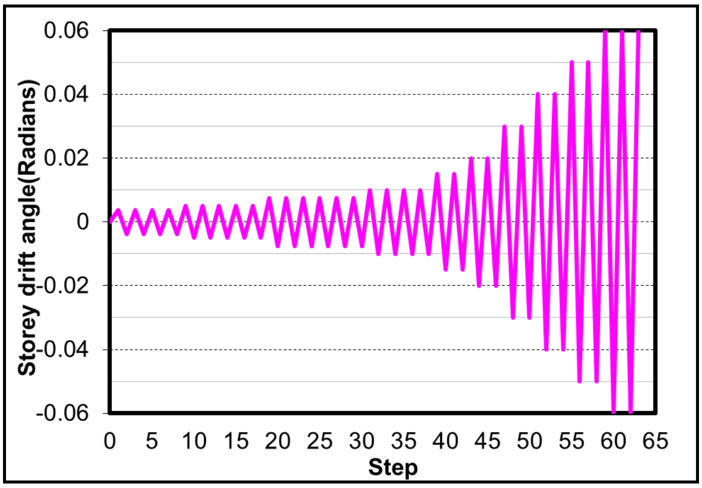

Fig 7. AISC [25] cyclic loading pattern

6-3- Dominant Failure Mode

While the fracture of the CJP groove welds in DF connections is a an unfavourable brittle failure mode, the typical failure in DF connections with adequate panel zone shear strength is the fracture of the drilled flange in proximity of the holes due to the excessive plastic strain accumulation in the drilled flange area. The experimental results by Vetr and Haddad [22] showed that this type of failure is ductile and usually occurs at very large storey drifts.

The focus of the current study is to provide design recommendations to prevent brittle failure modes at groove weld lines and reduce stress/strain concentrations in the drilled flange area of the DF connections. To study the dominant failure modes in DF connections, Figs. 8 and 9 compare the EPEQ distributions of DF connections with different hole configurations

(Dh1 to Dh4 connections) and panel zone shear strength ratios (V_{PZMy}/V_y =0.7 and 1.1) at storey drift angle of 0.04 radians. Overall, the results indicate that Dh1 and Dh4 connections exhibited the lowest and the highest equivalent plastic strains in the drilled flange area of the connections, respectively.

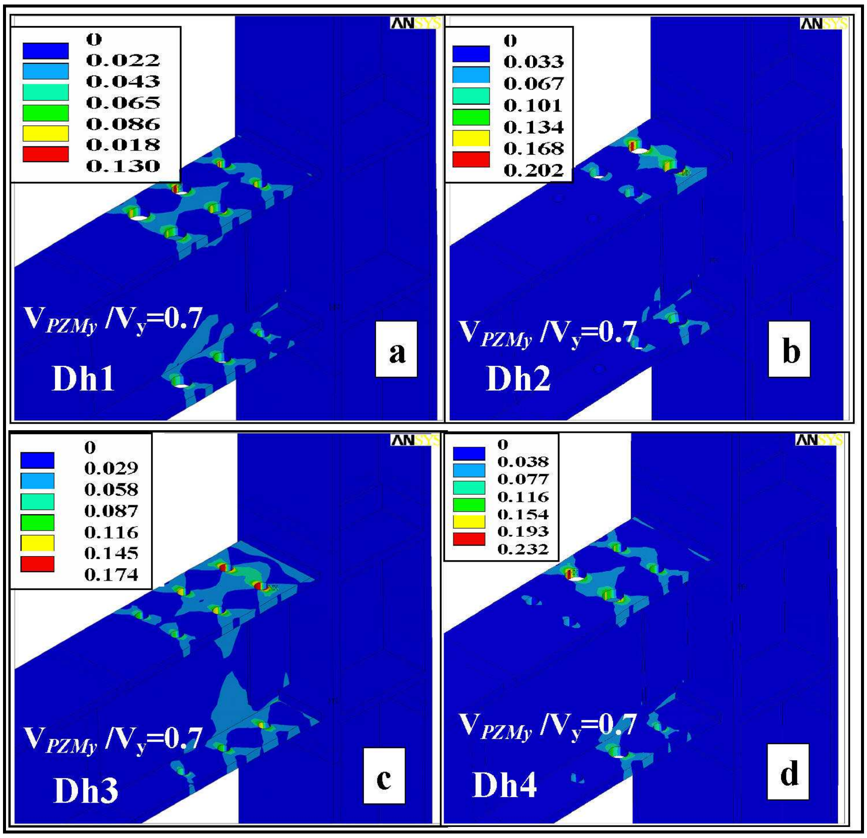

Fig 8. Equivalent plastic strain distributions of Dh1, Dh2, Dh3 and Dh4 connections with $V_{PZMy}/V_y = 0.7$ at storey drift angle of 0.04 radians

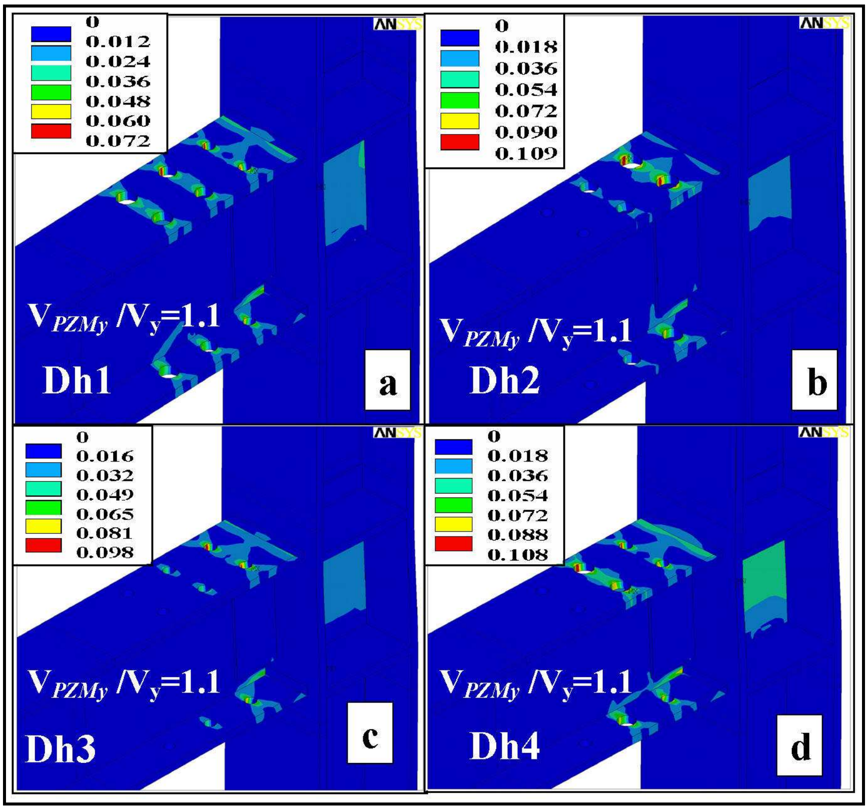

Fig 9. Equivalent plastic strain distributions of Dh1, Dh2, Dh3 and Dh4 connections with $V_{PZMy}/V_y = 1.1$ at storey drift angle of 0.04 radians

For better comparison, Table 4 presents maximum Equivalent Plastic Strain (EPEQ) at beam to column groove weld lines and holes' edge locations in different DF connections. It is shown in Table 4 that the maximum EPEQ in Dh1 connections around drilled holes was 15% to 44% lower than the maximum strains in DF connections with other drilled hole configurations. On the other hand, Dh1 connections also exhibited lower equivalent plastic strains (up to 40%) in the CJP groove weld lines. This implies that using Dh1 configuration (see Table 3) can reduce the chance of undesirable brittle failure mode in the CJP groove weld lines and also increase the flexural strength of the DF connections by reducing the maximum Equivalent Plastic Strains in the vicinity of the drilled holes.

Table 4. Comparison of maximum Equivalent Plastic Strain at groove weld lines (point B) and holes' edge locations in different DF connections

DF Connection	$V_{PZMy}/V_y = 0.7$		$V_{PZMy}/V_y = 0.9$		$V_{PZMy}/V_y = 1.1$	
	Critical point B	**Around holes**	**Critical point B**	**Around holes**	**Critical point B**	**Around holes**
Dh1	1.93E-02	1.30E-01	4.96E-02	1.04E-01	6.71E-02	7.20E-02
Dh2	2.20E-02	2.02E-01	5.20E-02	1.43E-01	6.86E-02	1.09E-01
Dh3	2.42E-02	1.74E-01	5.90E-02	1.23E-01	6.90E-02	9.80E-02
Dh4	3.20E-02	2.32E-01	5.90E-02	1.22E-01	7.00E-02	1.08E-01

6-4- Effects of Drilled Hole Locations

In this section, six different connections with panel zone shear strength ratio of 0.7 are considered (WUF-0.7, RBS-0.7, Dh1-0.7, Dh2-0.7, Dh3-0.7, and Dh4-0.7 in Table 3). To investigate the effects of drilled hole locations on the seismic performance of DF connections, a set of 11 new FE models are developed for each DF connection configuration by varying the distance of the first drilled hole row on the beam flanges from the column face (edge distance L* in Fig. 6) from 0 to 10 times of the diameter of the drilled holes (D).

According to AISC 341-10 seismic provision [25], beam-to-column connections in steel moment frames shall be capable of sustaining an inter-storey drift angle of at least 0.04 radians, while the measured flexural resistance of the connection (determined at the column face) is at least 80% of the plastic moment capacity of the connected beam. Therefore, in this study, performance parameters such as Equivalent Plastic Strain (EPEQ), Triaxiality Ratio (TR) and Rupture Index (RI) are presented and compared at storey drift angle of 0.04 radians.

Based on the previous experimental tests, points “A” and “B” in Fig. 3 are considered as the critical points on CJP groove weld lines with maximum stress demands. Fig. 10 compares RI at the critical points A and B for the six selected connections as a function of “edge distance” (or clear hole distance) to “hole diameter” ratio (L*/D). The results in general indicate that the drilled hole locations can significantly affect the maximum RI of DF connections. While RBS connections always exhibited the minimum RI in the CJP groove weld lines, using DF connections with optimum drilled hole locations can lead to almost similar results. It is shown in Fig. 10 that the optimum range for L*/D ratio in DF connections is between 3 to 5, which results in lower RI at the critical points of the CJP groove weld lines, and thus less fracture potential. Based on this conclusion, DF connections Dh1 to Dh4 are designed with the optimum L*/D ratio of 4 (see Fig. 6).

It is also shown in Fig. 10 that the maximum RI in DF connections with very small L*/D ratio (i.e. less than 1.5) is considerably higher than similar RBS and WUF connections. This implies that the clear distance between the first row of the drilled flange holes and the column face in DF connections should be at least 1.5 times the diameter of the holes. Otherwise, the drilled holes will reduce the performance of the connections by increasing the fracture potential of the CJP groove weld lines. Similarly, the results indicate that the performance of DF connections with very high L*/D ratios (i.e. greater than 10) is not better than conventional WUF connections. It means the drilled holes in this case cannot practically reduce the stress concentrations at the CJP groove weld lines.

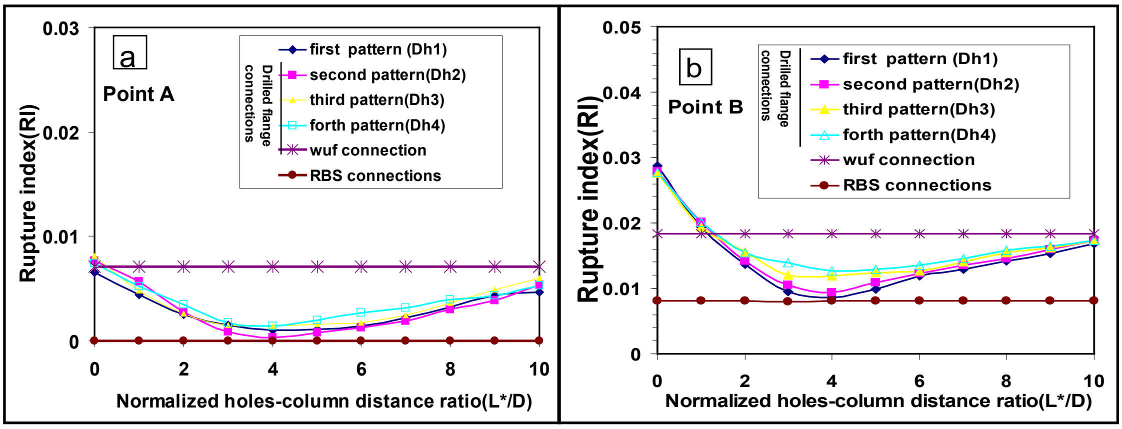

Fig 10. Rupture Index (RI) at the critical points A and B versus clear hole distance to diameter ratio (L^*/D), storey drift angle of 0.04 radians and $V_{PZMy}/V_y = 0.7$

6-5- Effects of Drilled Hole Configuration

The results of the analytical models corresponding to the four series of DF moment resisting connections Dh1 to Dh4 (see Table 3) are used to investigate the effects of using different drilled hole configurations. Fig. 11 compares the Equivalent Plastic Strain (EPEQ), Triaxiality Ratio (TR) and Rupture Index (RI) along CJP groove weld lines in WUF, RBS and DF connections with V_{PZMy}/V_y of 0.7 and 0.9. It is shown that EPEQ and RI in RBS and DF connections are significantly higher at the centre of the connection joint (point B in Fig. 3) compared to values at the two edges of CJP groove weld lines (point A in Fig. 3), which is in agreement with the previous experimental test observations [22, 24]. For WUF connections with high panel zone shear strength ratio, EPEQ and RI at the corners (point B) tend to the maximum values at the centre of the connection (point A). For better comparison, Table 5 compares the maximum EPEQ, TR and RI at CJP groove weld lines in DF, WUF and RBS connections. As it was expected, for the same storey drift angel, WUF connections exhibited significantly EPEQ and RI compared to similar RBS and DH1 connections. This behaviour can explain the poor seismic performance of WUF connections in previous earthquakes [1-3].

The results of this study show that, in general, the configuration of the drilled holes can play an important role in the performance of DF connections. Table 5 and Fig. 11 show that using drilled hole diameters “40mm, 55mm, 40mm” (Dh4 in Fig. 6) resulted in a higher EPEQ and RI and lower TR in the CJP groove weld lines compared to the other configurations. This implies that this type of hole configuration (i.e. using large holes in the middle row) leads to higher fracture potential, and hence lower cyclic performance. In contrast, DF connections with “40mm, 55mm, 60mm” hole diameters (Dh1 in Fig. 6) provided the best design solution with up to 40% less EPEQ and 25% less RI compared to the other DF connections.

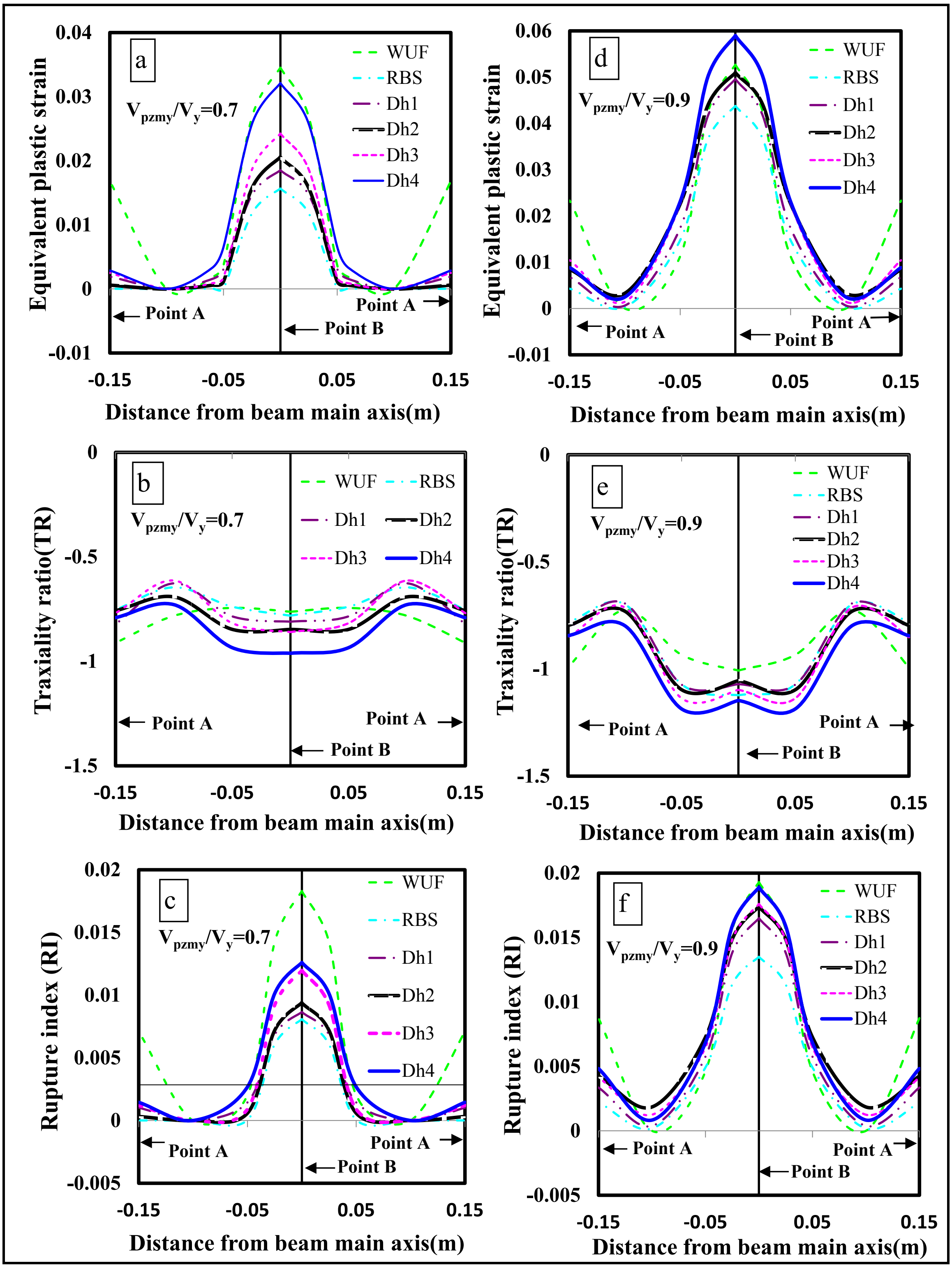

Fig 11. Equivalent Plastic Strain (EPEQ), Triaxiality Ratio (TR) and Rupture Index (RI) along CJP groove weld lines in WUF, RBS and DF (Dh1 to Dh4) connections with panel zone shear strength ratio (V_{PZMy}/V_y) of 0.7 and 0.9, storey drift angle of 0.04 radians

Table 5: Maximum Equivalent Plastic Strain (EPEQ), Triaxiality Ratio (TR) and Rupture Index (RI) at CJP groove weld lines

Connection	V_{PZMy}/V_y	EPEQ	TR	RI
Dh1	0.7	1.93E-02	-0.61	9.40E-03
	0.9	4.96E-02	-0.69	1.65E-02
	1.1	6.71E-02	-0.86	1.80E-02
Dh2	0.7	2.20E-02	-0.69	1.10E-02
	0.9	5.20E-02	-0.73	1.74E-02
	1.1	6.86E-02	-0.92	1.82E-02
Dh3	0.7	2.42E-02	-0.62	1.20E-02
	0.9	5.90E-02	-0.71	1.76E-02
	1.1	6.90E-02	-0.92	1.85E-02
Dh4	0.7	3.20E-02	-0.72	1.26E-02
	0.9	5.90E-02	-0.79	1.90E-02
	1.1	7.00E-02	-0.94	2.00E-02
WUF	0.7	3.45E-02	-0.74	1.83E-02
	0.9	5.27E-02	-0.72	1.88E-02
	1.1	7.2E-02	-0.73	1.99E-02
RBS	0.7	1.57E-02	-0.64	8.08E-03
	0.9	4.38E-02	-0.69	1.35E-02
	1.1	6.21E-02	-0.87	1.51E-02

6-6- Effects of Panel Zone Shear Strength Ratio

The shear strength of the connection panel zone (V_{PZMy}/V_y) is another parameter that can affect the seismic performance of the moment resisting connections. In practical applications, V_{PZMy}/V_y ratio can be easily controlled by changing the thickness of doubler plates in the connection (see Equations 3 and 4). Comparison between Figs. 11 (b) and (e) shows that, in general, Triaxiality Ratios (TR) at the critical points on the CJP groove weld lines increases by an increase in the shear strength ratio (V_{PZMy}/V_y). Therefore, connections with higher shear strength ratio are expected to be more prone to the premature fracture in the CJP grove weld lines. Similarly, by comparison between Figs. 8 and 9, it can be concluded that using a strong panel zone will considerably reduce the maximum plastic strains in the drilled flange area of the DF connections. For example, it is shown that the maximum strains are, on average, two times higher in the DF connections with V_{PZMy}/V_y =0.7 compared to the similar connections with V_{PZMy}/V_y =1.1. These results confirm that decreasing the shear strength ratio in DF connections (e.g. by increasing the shear strength of the panel zone) can help transferring plastic strains from column face to the drilled flange area of the beam.

To study the effects of panel zone shear strength ratio in more details, Fig. 12 compares the Von-Mises stress distributions in the WUF, RBS and Dh1 connections with V_{PZMy}/V_y of 0.7, 0.9 and 1.1, at storey drift angle of 0.04 radians. It should be mentioned that V_{PZMy}/V_y ratios equal to 0.7, 0.9 and 1.1 can represent connections with strong, medium and weak panel zones, respectively. For better comparison, the maximum Von-Misses stresses at CJP groove weld lines and connection panel zones are also compared in Table 6. It should be mentioned that, based on the ultimate strength of the weld material (welding electrode E7018), maximum allowable Von-Mises stress at CJP groove weld lines is around 4900 Kg/cm^2. The results in Table 6 indicate that changing the panel zone shear strength will change the stress distribution in the connections; however its effect is in general more significant in the panel zone area rather than CJP groove weld lines.

Comparison between Figs. 12 (a) and (c) indicates that the drilled holes connections could shift the stress concentration from the CJP weld lines of WUF connections to the drilled area of the flange (i.e. intentional weak area of the flange). However, the capability of DF connections in transferring plastic stress accumulation from column face to the drilled flange area decreases by increasing the panel zone shear strength ratio. The results show that the DF connection with weak panel zone (i.e. V_{PZMy}/V_y =1.1) exhibited up to 10% higher Von-Mises stress at the CJP groove weld lines compared to the similar connection with strong panel zone (i.e. V_{PZMy}/V_y =0.7). It is also shown that using weak panel zone increased the maximum Von-Mises stress in the connection panel zone by almost 6%. This implies that decreasing the panel zone shear strength ratio (e.g. by increasing the thickness of doubler plates) can reduce the risk of premature fracture in the CJP groove weld lines as well as failure in the panel zone. This enhancement in the performance can be attributed to the higher contribution of strong panel zones in shifting the plastic strains from the beam flange to the column face.

Table 6. Maximum Von-Misses stress at CJP groove weld lines and connection panel zone

Von-Misses Stress (Kg/cm^2)		**Strong Panel Zone** $V_{PZMy}/V_y = 0.7$	**Medium Panel Zone** $V_{PZMy}/V_y = 0.9$	**Weak Panel Zone** $V_{PZMy}/V_y = 1.1$
Maximum stress at CJP groove weld lines	WUF	4333	4310	4155
	RBS	4058	4030	4084
	Dh1	4103	4456	4514
Max stress in the connection panel zone	WUF	3854	3774	4175
	RBS	4058	4030	4662
	Dh1	4092	3862	4514

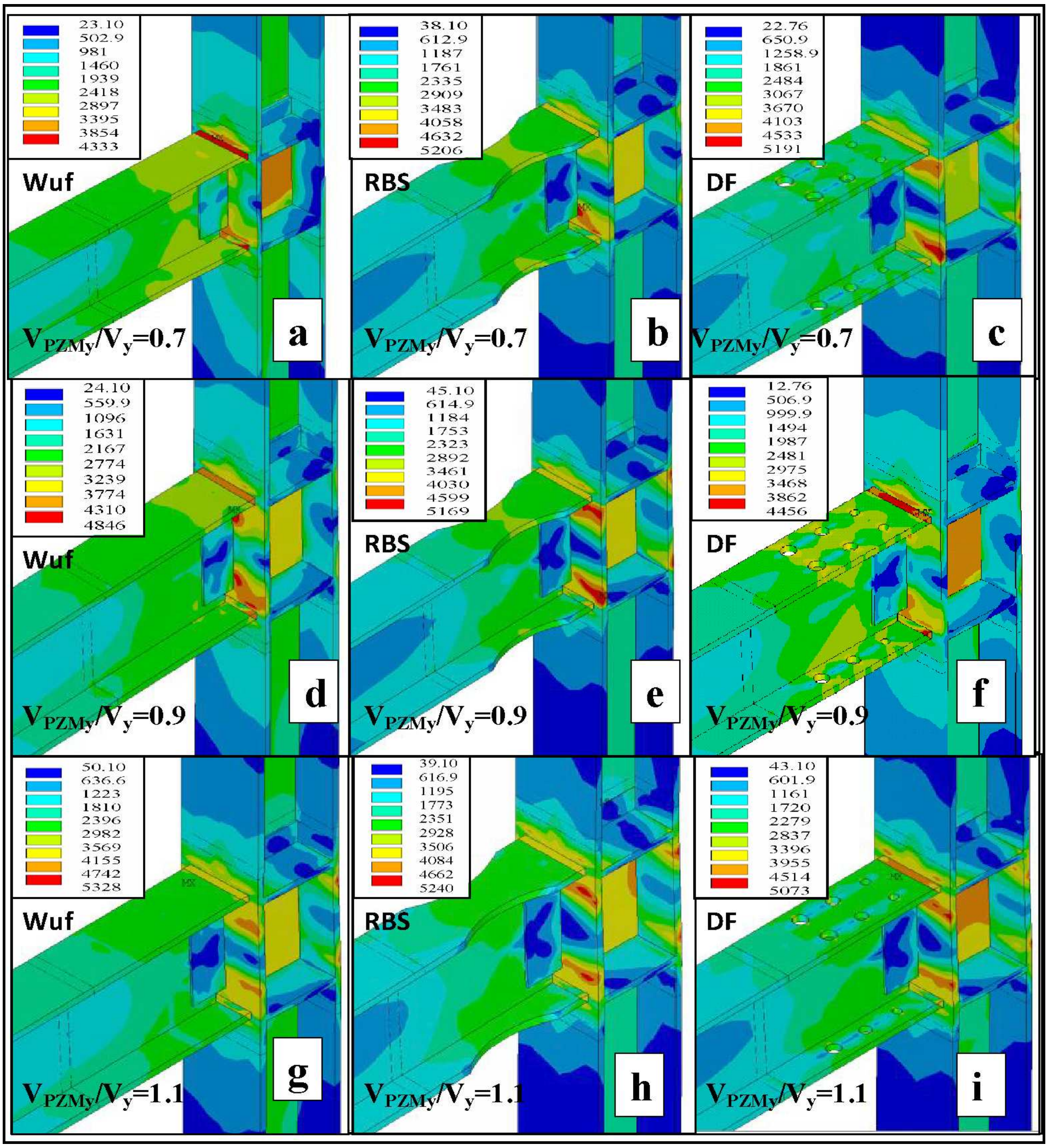

Fig 12. Von-Misses stress distribution in WUF, RBS and DF (Dh1) connections with panel zone shear strength ratios (V_{PZMy}/V_y) of 0.7, 0.9 and 1.1 at storey drift angle of 0.04 radians

Fig. 13 shows the maximum Equivalent Plastic Strain (EPEQ) and Rupture Index (RI) at the critical points of CJP groove weld lines (points A and B in Fig. 3) in WUF, RBS and Dh1 to Dh4 connections as a function of panel shear strength ratio (V_{PZMy}/V_y). The results indicate that there is a general trend of increasing equivalent plastic strains and rupture indices by increasing the shear strength ratio, with an exception for some of the corresponding values in WUF connections. For example, it is shown that DF connections with shear strength ratio V_{PZMy}/V_y =1.1 (i.e. weak panel zone) experience more than 3 times higher equivalent plastic strains and up to 90% higher RI compared to those with shear strength ratio V_{PZMy}/V_y =0.7 (i.e. strong panel zone). This implies that using strong panel zone can significantly improve the performance of the DF connections, which is in agreement with Von-Misses stress distributions presented in Fig 12. While RBS connections always exhibited lower EPEQ and RI compared to similar WFS and DF connections, the results in Fig. 13 show that the cyclic performance of the optimum designed DF connections (e.g. Dh1 with V_{PZMy}/V_y =0.7) can be as good as well-designed RBS connections.

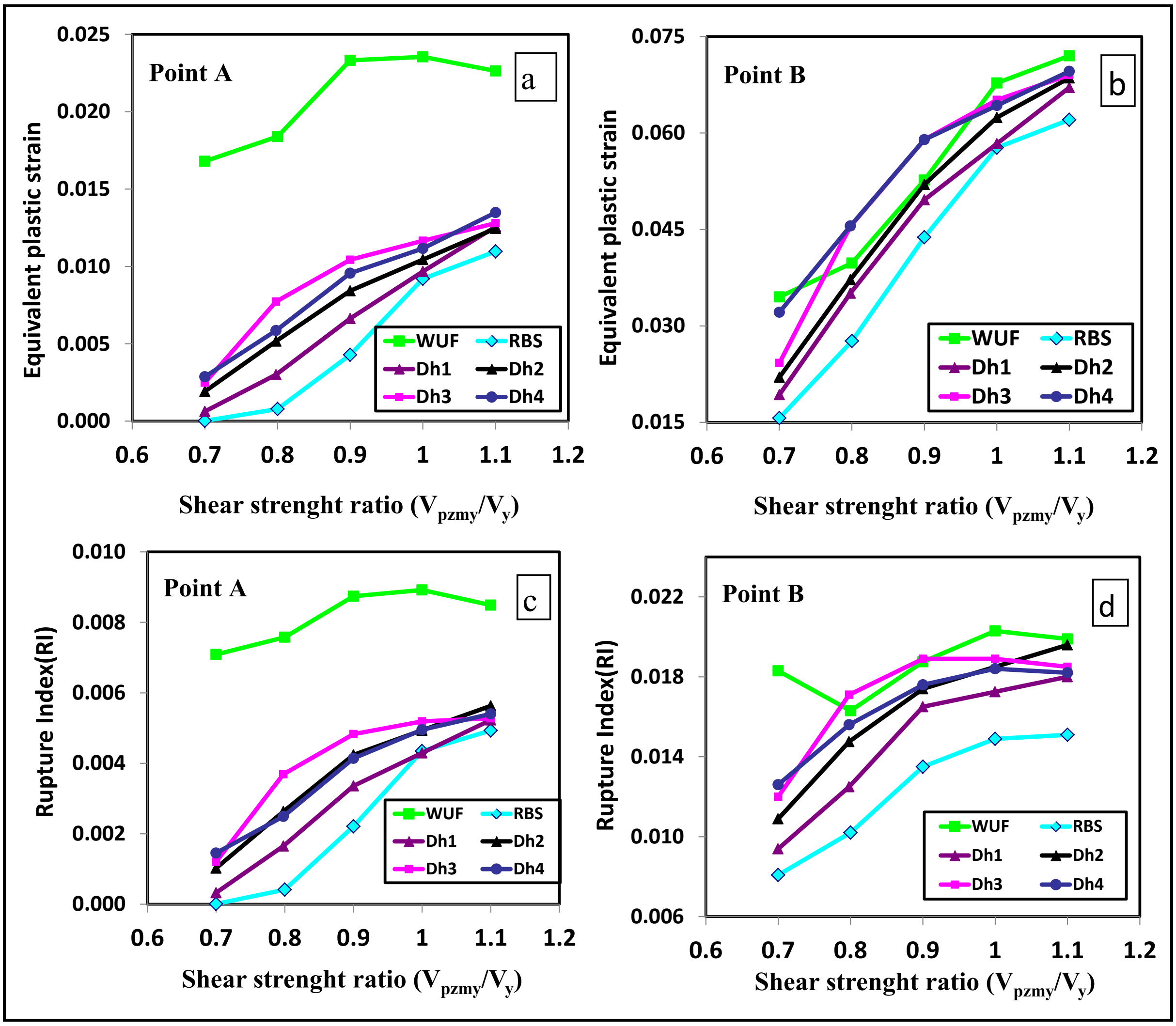

Fig 13. Equivalent Plastic Strain (EPEQ) and Rupture Index (RI) versus panel zone shear strength ratio (V_{PZMy}/V_y) for critical points A and B on CJP groove weld lines at storey drift angle of 0.04

To study the potential failure in the panel zone, Fig. 14 compares the panel zone shear strain at storey drift angle of 0.04 in WUF, RBS and DF connections with shear strength ratios (V_{PZMy}/V_y) of 0.7 to 1.1. As it was expected, increasing the panel zone shear strength ratio was always accompanied by an increase in the panel zone shear strain at the failure point.

The results indicate that, for similar V_{PZMy}/V_y , Dh4 connections experienced higher panel zone shear strains compared to other DF connections. This is especially evident in the connections with a strong panel zone (i.e. V_{PZMy}/V_y =0.7). It is also shown in Fig. 14 that WUF connections always exhibited the lowest panel zone shear strains, which is consistent with the stress distributions shown in Fig. 12.

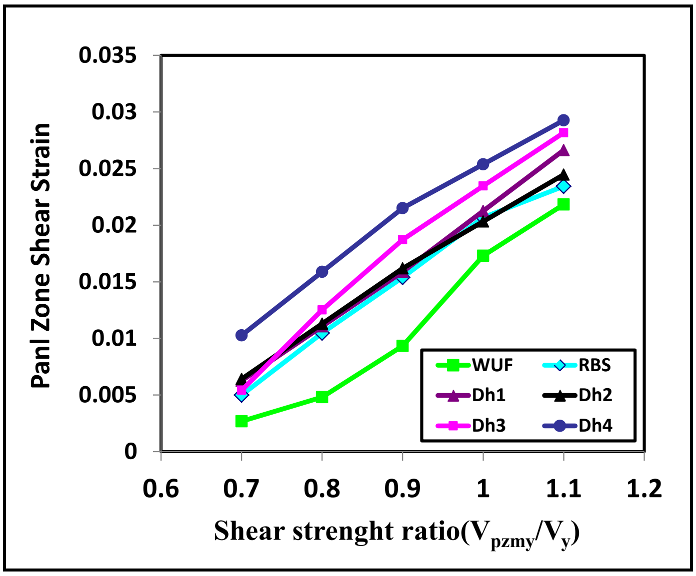

Fig 14. Comparison of panel zone shear strain at storey drift angle of 0.04 in WUF, RBS and DF connections with different panel zone shear strength ratios (V_{PZMy}/V_y)

The results of this study show that drilled flange (DF) moment connections can efficiently shift the stress concentrations from column face to the drilled flange area of the beam (i.e. intentional weak area) and, therefore, provide a viable alternative to the relatively complex reduced beam section (RBS) connections. However, DF connections should be designed carefully to prevent premature rupture of CJP groove weld lines and failure in the drilled flange area and panel zone. This can be achieved by using an appropriate panel zone shear strength ratio and drilled hole location and configuration as it was discussed in the paper.

7. CONCLUSIONS

A comprehensive analytical study was carried out to investigate and improve the seismic performance of DF moment resisting connections as an efficient and easy-to-construct alternative to more complex RBS connections for ductile frames in seismic regions. More than 70 non-linear FE models were used to investigate the effects of drilled flange hole locations, panel zone shear strength ratio and hole configuration on the seismic performance of DF connections, and find the optimum design parameters. Based on the presented results, the following conclusions can be drawn:

1- Detailed FE models can adequately simulate the non-linear behaviour and the failure mechanism of WUF, RBS and DF connections used in previous experimental tests.
2- The drilled flange holes in DF connections could efficiently shift the stress concentrations and plastic strain accumulation from the CJP groove weld lines to the intentional weak area of the flange (i.e. drilled flange area) and, therefore, prevent the premature brittle fracture of the welded joints. It was shown that, for similar storey drift angels, DF connections exhibited more than two times lower Equivalent Plastic Strain (EPEQ) and Rupture Index (RI) at the critical points of the CJP groove weld lines compared to similar WUF connections.
3- While drilled flange holes with a large “edge distance” will not be efficient, using a very small edge distance will significantly increase the RI at the CJP groove weld lines. Based on the results of this study, the optimum range for the "edge distance" to the “hole diameter” ratio in DF connections was found to be between 3 to 5.
4- Drilled hole configuration plays an important role in controlling the non-linear performance of DF connections. Increasing hole diameters from the column face (e.g. 40mm, 55mm, 60mm) could reduce the maximum EPEQ at drilled flange locations and CJP groove weld lines by up to 44% and 40%, respectively, compared to DF connections with other drilled hole configurations. This hole configuration can also reduce the RI of the CJP groove weld lines by up to 25%.
5- Decreasing the panel zone shear strength ratio V_{PZMy}/V_y in DF connections (i.e. using a strong panel zone) can considerably reduce the maximum EPEQ and RI at CJP groove weld lines and maximum shear strains in the connection panel zone. Using a strong panel zone, however, will increase the maximum plastic strains at the drilled flange area of the connections.
6- It is shown that using V_{PZMy}/V_y =0.7 with optimum drilled hole location and configuration can significantly reduce the chance of undesirable brittle failure mode in CJP groove weld lines of DF connections and also increase their flexural strength by reducing the stress/strain concentrations in the vicinity of the drilled holes.

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