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INFLUENCE OF 4 BOLTS-PER-ROW CONNECTION ON A STEEL FRAME BUILDING SUBJECTED TO COLUMN LOSS

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ABSTRACT

In multi-story steel frames, the beam-to-column connections should allow the transfer of the forces both in normal loading condition and also prevent progressive collapse when subjected to accidental loads. This paper studies the mechanical performance of a beam-to-column connection using different bolt configurations, under extreme loading situation, for which the connection's ductility and post-flexural behaviour should provide an adequate level of robustness to the original structure. The study presents a parametric 3D FEM analysis, developed in order to assess the performance of several geometric configurations.

Keywords: endplate connections, robustness, FEM analysis, ductility, 4-bolt-rows.

INTRODUCTION

The geometry and configuration of beam-to-column connections can have significant influence on the global behaviour of a steel moment-resisting (MR) structure, substantially affecting their strength and stiffness as well as its ductility and post-flexural behaviour. The extended end-plate connection is widely used in its classical configuration with two bolts-per-row, which behaviour is extensively studied and documented (Zoetemeijer, 1974) (Girão Coelho, Bijlaard and Simões da Silva, 2004) (Dubina *et al.*, 2011). However, new typologies of connections are possible despite the absence of accurate knowledge of their mechanical performance.

The extended end-plate configuration with four bolts-per-row solution has been approached by various researchers (Demonceau, Weynard and Müller, 2010) (Pisarek and Kozlowski, 2005), as yet to be considered in the design standards. To this contributes the discrepancy between proposed analytic descriptions for the behaviour of the equivalent t-stubs with four-bolts-per-row, where even the number of failure modes has several different proposals, from three (Demonceau, Weynard and Müller, 2010), to four (Pisarek and Kozlowski, 2005) and more recently five modes (Gang and Xuesen, 2017).

The four-bolts-per-row configuration has the potential to be a viable solution for achieving improved robustness performances for buildings in which good structural reliability represents a key criterion. According to EN1991-1-7 (CEN 2006) a structure should withstand extreme loading events, like explosions, fire or impact, which may affect its mechanical properties as far triggering progressive collapse. In these types of scenario, the connection plays a crucial role, in order to guarantee the transfer of loads from the damaged to the undamaged elements, thus allowing the creation of alternative loading paths. In order to satisfactorily accomplish this function, the connection in the affected areas should possess adequate levels of performance in terms of strength and ductility, in order to offer a good re-equilibration of internal forces.

Due to the levels of extended deformation of the structure subjected to an extreme loading event, the beams - and their adjacent connections - may also carry a significant amount of axial force, besides shear and bending moment. Such is the case in the event of loss of a column of a lower story, where the first set of beams above the damaged point is subjected to an additional axial force due to catenary action. However, most classic configurations of connections with two-bolts-per-row cannot develop significant catenary forces (Dinu, Marginean and Dubina, 2017). In this context, this study proposes new connection typologies by the addition of outer bolts in order to complement the mechanical characteristics of the connection and provide additional strength to interaction between bending moment and axial load.

REFERENCE MODEL

The studied sub-structure represents a variation of the original study on a four-bay-four span structure with six storeys (Figure 1.) and modified connection typology. The structure was previously employed in the framework of broader study on robustness (FRAMEBLAST, 2017). For the purpose of the study a two-span frame was extracted (hatched frame in Figure 1) in order to perform a column-loss experimental test, where a downwards displacement is imposed on the central column, considered lost due to an explosion event (Figure 2).

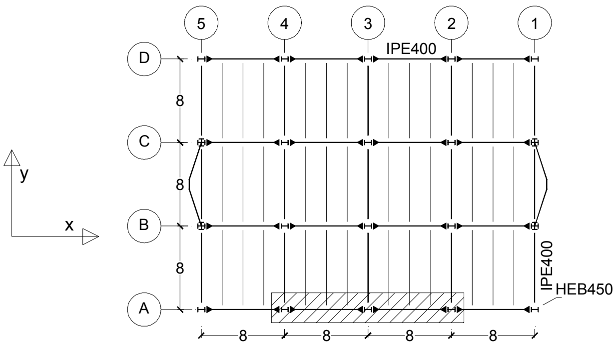


Fig. 1 - Reference building layout plan and position of retrieved specimens for testing

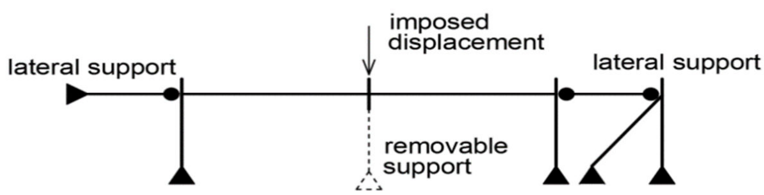


Fig. 2 - Boundary conditions and loading application for frame testing

The structure was designed as MR frames, according to usual values of dead and live loads, and seismic load corresponding to a low-seismicity zone (ag = 0.08g, TC = 0.7s) - Figure 1. The structure was scaled down due to laboratory constraints and the resulting elements were: columns - HEB260; beams - IPE220, both in steel S275J0 (Dinu, Marginean and Dubina, 2017).

The resulting original MR end-plate beam-to-column connection has five rows of M16 bolts H.R. 10.9, symmetrically distributed, and 16mm extended end-plates (see details in Figure 3). In accordance with EN 1993-1-8 classification for strength and rigidity, this connection is classified as semi-rigid and partial-strength (semi-continuous).

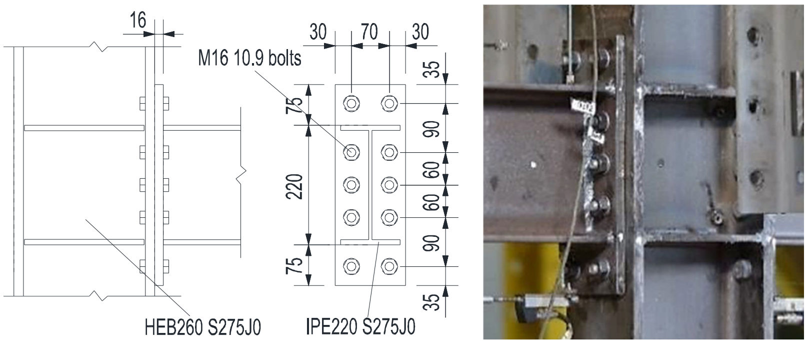
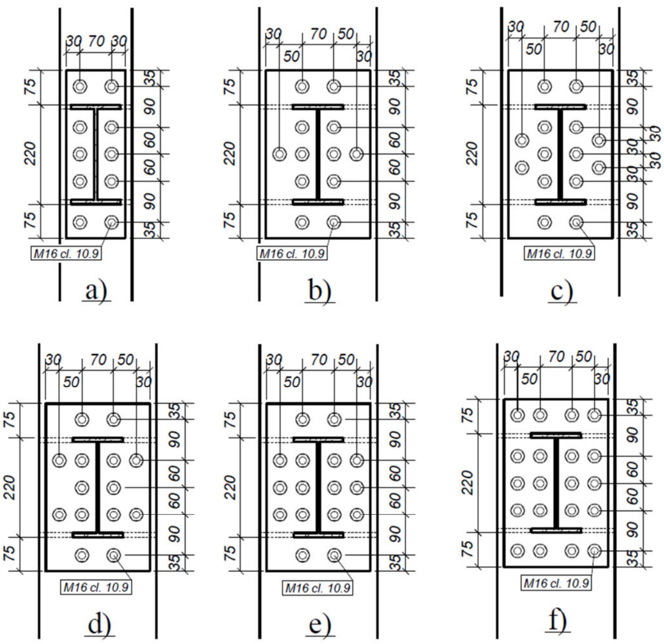


Fig. 3 - Details of the original EP specimen (dimensions in mm)

NUMERICAL MODELS

The present study is focused on the behaviour of modified typology of the original two bolts-per-row by the addition of outer bolts on the end-plate, thus forming rows with four bolts, in view of improving the mechanical behaviour of the connection and finally the overall levels of robustness of the structure. In the first configuration (**2BR_O**), a set of exterior bolts is added for the middle bolt-row, then two staggered and non-staggered set of bolts are added. The three four-bolt configuration corresponds to doubling the inner rows, while at last, 5 set of additional bolts are added, forming a complete 4-bolts per-row connection (Figure 4).



Model name	Suppl. rows	Alignment
2BR_O	0	Non-staggered
2BR_1	1	Non-staggered
2BR_2V	2	Staggered
2BR_2	2	Non-staggered
2BR_3	3	Non-staggered
4BR	5	Non-staggered

a) 2BR_O b) 2BR_1 c) 2BR_2V
d) 2BR_2 e) 2BR_3 f) 4BR

Fig. 4 - Connection configurations (dimensions in mm) and description of the models considered

All the supplementary bolts were identical in diameter and steel class to the original inner bolts (M16 H.R. 10.9)

The FE models were developed in the 3D FEA software package ABAQUS (Dassault Systèmes, 2016). The models comprise two consecutive beams, connected to three columns. The columns were restrained laterally (both in-plane and out-of-plane) and their support was modelled as a pinned connection. The models were monitored in displacement-controlled steps. They considered a vertical downward imposed displacement of the central column (Figure 2). The elements were modelled using solid finite elements (C3D8R with reduced integration). Between steel elements in contact a general contact law was applied.

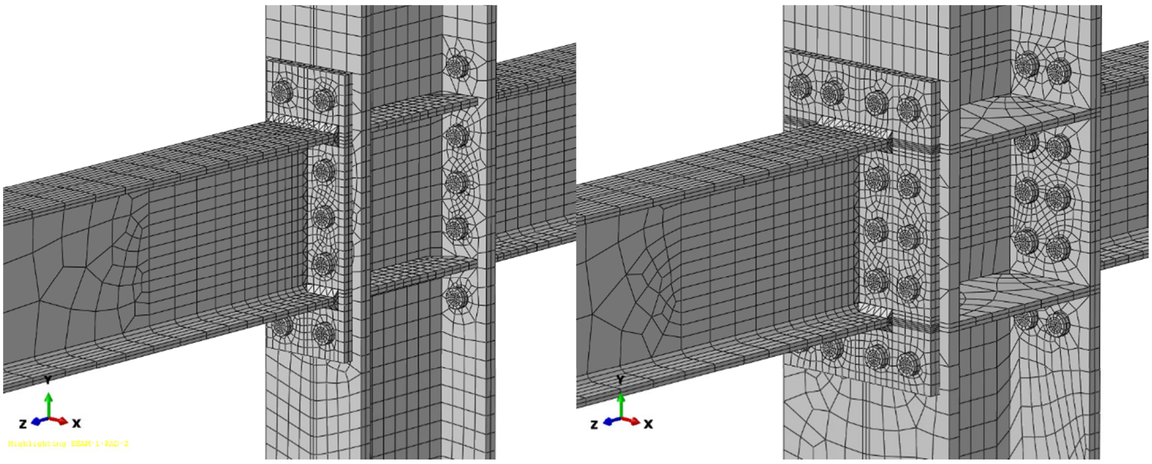


Fig. 5 - Finite element models for both the original and the 4BR configuration

Figure 5 shows the mesh applied to the different elements of the connection: a denser mesh was applied to the end-plates and bolts as the plastic deformations are expected in these components. Also, it could be noted that both the original and modified connections use an HEB 260 profile, but in case of the original configuration the flanges were reduced to a width of 160mm in order to realise the final cruciform cross-section made by two such profiles.

Table 1 - Steel mechanical properties for different elements

	f_y	f_u
	[Mpa]	[Mpa]
Bolts M16	965	1080
Beam flange IPE220	351	498
Beam web IPE220	370	497
Column flange HEB260	393	589
Column web HEB260	402	583
End-plate 16mm	305	417

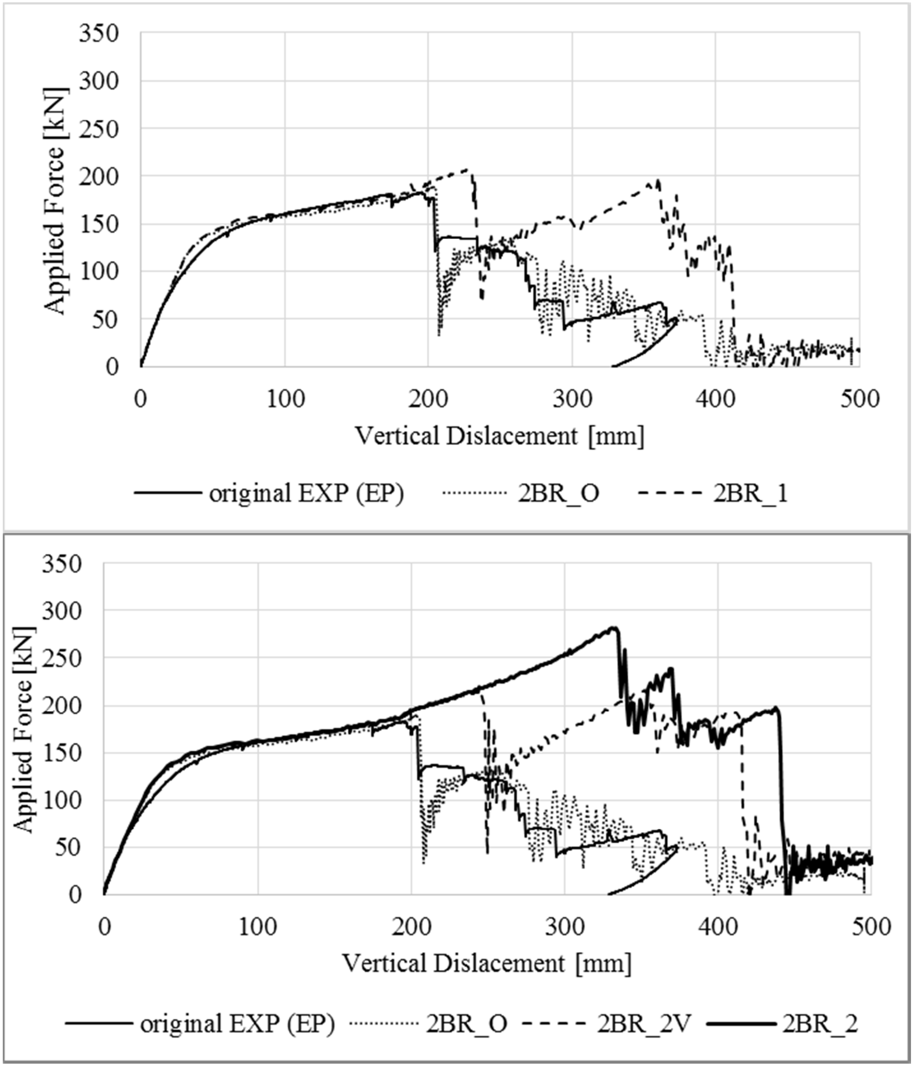
The material properties used in the definition of the elements were taken from the results given by the tensile tests performed on the original configuration (**Table**). The stress-strain curves used in modelling considered modified true stress-strain behaviour curves. The failure behaviour of the material was also modelled, using ductile damage parameters (fracture strain and stress triaxiality) together with displacement-based damage evolution.

NUMERIC RESULTS

For each configuration, the force-displacement curve was registered and the failure pattern observed. In Table 2, the results for strength and rotation at maximum load are shown. The bending moment was evaluated at the point of connection between end-plate and column. Also, the force-displacement behaviour curves resulted from numerical analyses are represented in Figure 6, in comparison with the experimental test curve.

Table 1 - Comparison of results between the experimental test and the FE models

	Maximum vertical force	Maximum bending moment	Beam rotation at maximum load
	F_{max}	M_{max}	ϕ_{max}
	[kN]	[kNm]	[mrad]
EXP	182.14	124.77	64.609
2BR_O	188.79	129.32	70.717
2BR_1	207.12	141.88	81.194
2BR_2V	216.77	148.49	86.893
2BR_2	281.40	192.76	115.11
2BR_3	276.00	189.06	113.96
4BR	348.43	238.67	171.8



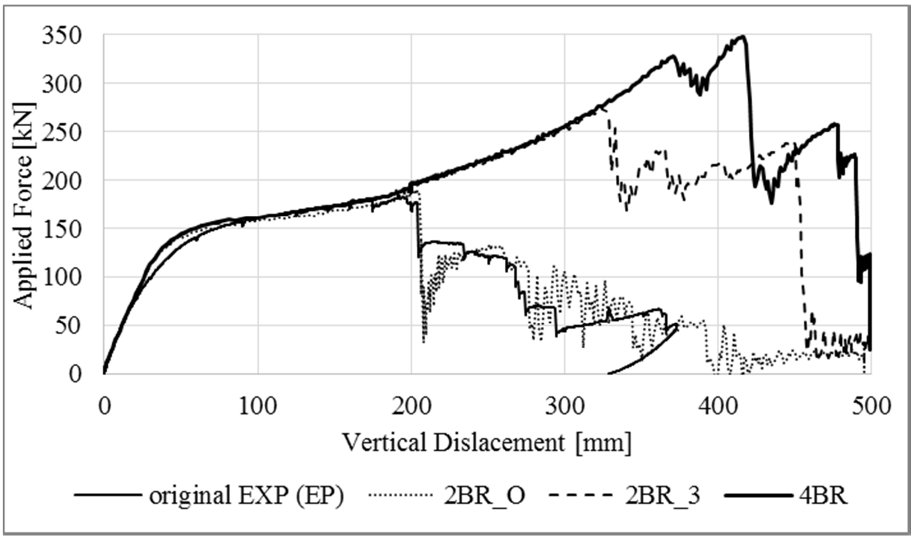


Fig. 6 - Comparison between force-displacement curves

As the results show, both the addition of extra bolts and the widening of the end-plate have a direct influence on the ultimate strength of the connection, by increasing the original resistance, and also on its ductility, allowing for higher rotations at the point of maximum strength. The widening of the end-plate by itself increases the strength of the connection by 8%.

The numerical models confirm a significant difference in the post-flexural behaviour, as the configurations with extra outer bolts are generally able to regain a significant percentage of their maximum strength after their initial failure (post-flexural rebound rate).

Table 3 - Post-flexural behaviour comparison

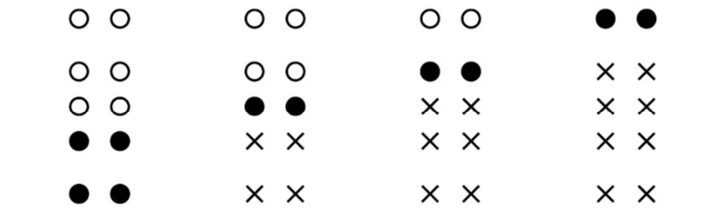
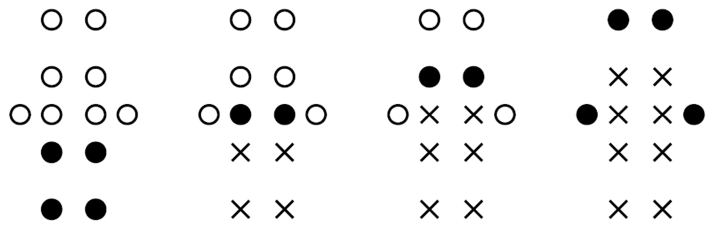
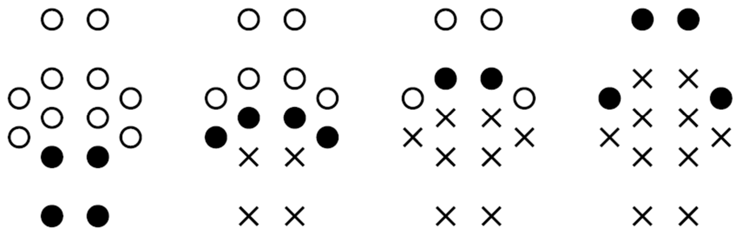
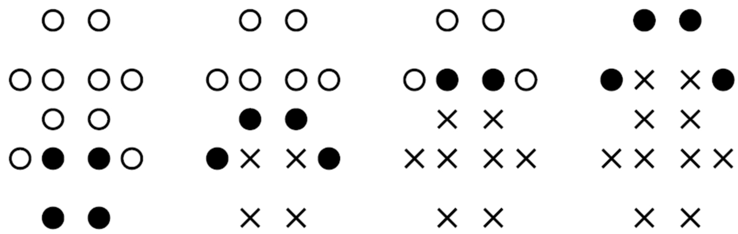
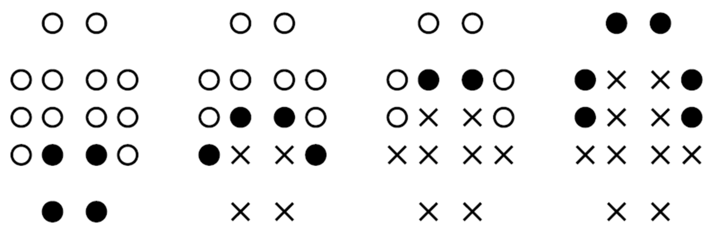
	F_{max}	F_{pc}	F_{pc}/F_{max}	d_v	d_{pc}	$\Delta d/d_v$
	[kN]	[kN]	[%]	[mm]	[mm]	[%]
EXP	182.14		no rebound	194.10		
2BR_O	188.79	132.88	70.4%	202.43	253.61	25.3%
2BR_1	207.57	199.97	96.3%	228.05	360.15	57.9%
2BR_2V	216.78	215.77	99.5%	243.96	352.44	44.5%
2BR_2	281.40	238.79	84.9%	330.92	369.85	11.8%
2BR_3	276.00	241.01	87.3%	323.54	448.26	38.5%
4BR	328.34	348.43	106.1%	370.56	416.93	12.5%

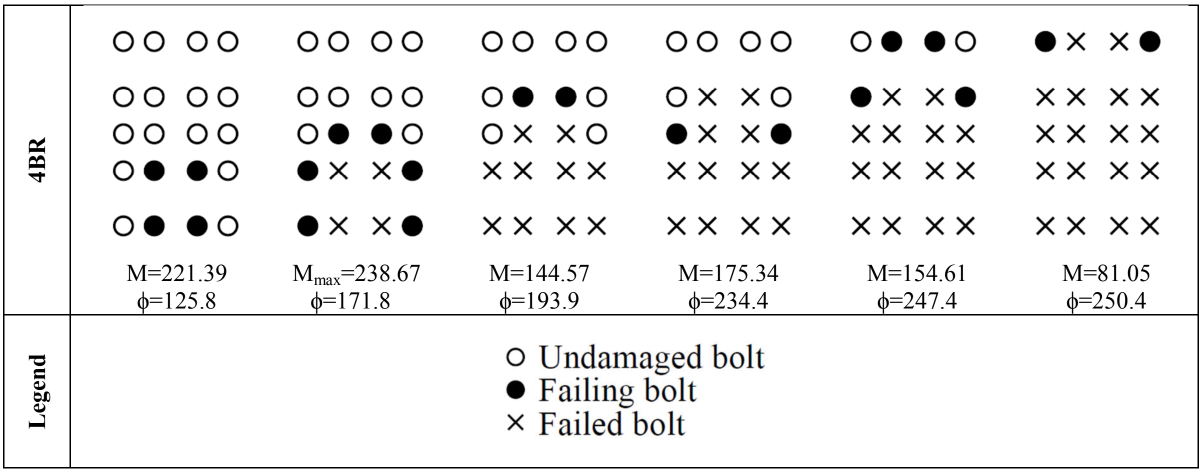
$F_{i,max}$ -vertical force at initial failure; F_{pcr} - maximum force after initial failure; d_v vertical displacement at initial failure; d_{pc} vertical displacement at maximum force after initial failure

FAILURE PATTERN AND INTERPRETATION

In the classic configuration of 2-bolts-per-row, the failure mechanism develops as an initial failure of the bolts adjacent to the flange in tension, followed by a sequential failure of each bolt-row, without significant post-flexural rebound between bolt failures. In Table 4, the failure sequence under sagging bending moment is shown (left to right), together with the corresponding values for bending moment and rotation, recorded for a connection on the central column.

Table 2 - Bolt failure sequence of central connection (under sagging moment) with corresponding bending moments [kNm] and rotations [mrad]

EXP & 2BR_O		M_{max}=129.32 ϕ=70.7	M=91.03 ϕ=101.8	M=32.29 ϕ=126.7	M=32.42 ϕ=142.7
2BR_1		M_{max}=141.88 ϕ=81.2	M=107.44 ϕ=143.4	M=83.80 ϕ=209.5	M=61.21 ϕ=218.1
2BR_2V		M_{max}=148.49 ϕ=86.9	M=141.86 ϕ=174.2	M=116.80 ϕ=212.1	M=132.63 ϕ=231.8
2BR_2		M_{max}=192.76 ϕ=115.1	M=157.28 ϕ=156.8	M=123.43 ϕ=179.3	M=131.80 ϕ=215.7
2BR_3		M_{max}=189.06 ϕ=114.0	M=159.54 ϕ=155.5	M=147.22 ϕ=191.22	M=163.17 ϕ=228.8



In the case of the alternative configurations featuring 4-bolts-per-row, the initial failure of the inner bolts from the 2 upper rows occurred in a similar way as in the classic 2-bolts-per-row configuration, each developing a slightly different failure sequence depending on the particular geometry.

In a detailed analysis, there are some particularities which should be noted. Although the configurations 2BR_2V and 2BR_2 are very similar, the difference in geometric position of the outer bolts highly affect the strength of the connection, as the contribution of the outer bolts in the first failure decreases as its distance to the centroid of the active group of bolts increases. This confirms some previous studies which demonstrated that, the inner bolt rows located close to the centroid of the connection noticeably increase the resistance and the rotation capacity under column loss. These bolt rows are very important to redistribute the internal forces developing into the connection, allowing to mobilize the catenary action under column loss, (Cassiano, D'Aniello and Rebelo, 2017).

On another note, the differences in performance between configurations 2BR_2 and 2BR_3 are very small, minimising the advantages of the middle extra bolt in first loading steps but improving it in later stages by slightly delaying the collapse.

By analysing the failure sequences two main conclusions can be drawn: (i) whilst analysing a single row with four bolts, the inner row always fail before the outer bolts; (ii) however, if considering all the rows which participate in each individual step of the failure, the outer bolts of a row tends to fail at the same time with the inner bolts of the next row(s) if they are adjacent to a flange. The results show that if no flange is present, the bolts tend to fail pair-by-pair (inner bolts and outer bolts separately).

CONCLUSIONS

While the connection with a classic configuration of two-bolts-per-row shows a simple and linear failure pattern, the alternative configurations featuring four bolts per row do not prove a linear failure row-by-row, but a non-uniform pattern. When adjacent to a flange, the bolts tend to form a resisting diagonal coupling between the first outer bolts of row with the inner bolts of a second row. This diagonal coupling can span through two rows, in the case of failure almost exclusively in bending, where the presence of catenary force is insignificant, or it can span through three rows in later stages, where catenary action is significant and has the effect of equalizing the tension of the remaining bolts, regardless of their position.

The clear difference of behaviour between the configurations with two extra bolts (2BR_2V and 2BR_2) at the levels of both strength and ductility show a direct correlation between the performance of the connection and the relative position of the outer bolts, and consequently, the potential to form diagonal couples. However, this failure patterns also challenges the suitability of the classic analytic method of equivalent T-stub calculation for studying the post-yielding behaviour of a bolted end-plate connection with four bolts per row.

Finally, all the configurations with additional outer bolts have recorded higher post-flexural rebound rates than the original model, thus providing good levels of strength and ductility after the first failure. This indicates that the addition of and outer set of bolts is a viable solution in situation of extreme loading, where the additional bolts can delay the failure of the connection at significant levels of strength.

ACKNOWLEDGMENTS

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