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EUROPEAN PREQUALIFICATION OF JOINT SOLUTIONS FOR STEEL STRUCTURES SUBJECTED TO SEISMIC ACTIONS – BOLTED UNSTIFFENED EXTENDED END-PLATE BEAM-TO-COLUMN JOINTS

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Abstract: *According to EN 1998-1, the seismic design of steel structures is based on the concept of dissipative structures, in which specific zones of the structures are identified and designed to develop plastic deformations in order to dissipate the seismic energy. In steel moment resisting frames, the beam extremities are generally used as dissipative zones, and the beam-to-column joints are designed in order to resist to the internal forces associated to the development of plastic hinges at the beam extremities. However, such an approach may lead to quite expensive joint solutions, mainly due to the fact that the possible overstrength and strain hardening effects developing in the dissipative zones have to be taken into account when designing the non-dissipative zones. EN 1998-1 allows the use of partial-strength joints as dissipative zones but, in this case, the ductility and the dissipation capacity of the joints should be demonstrated by means of experimental tests, which is not realistic for practical projects. For this reason, a European RFCS project named EQUALJOINTS was developed with the objective of developing and prequalifying three types of bolted joints that are commonly used in European practice. Within the present paper, the main design issues and the relevant findings obtained for bolted unstiffened extended end-plate beam-to-column joint are described and discussed.*

Introduction

The Eurocode EN 1998-1 (CEN, 2004) dealing with the seismic design of structures allows the use of dissipative joints in case of earthquake if the ability of the joint to dissipate the energy associated to the seismic loading can be demonstrated. However, the present draft of the Eurocodes and, in particular, EN 1993-1-8 (CEN, 2005) dealing with the characterisation of steel joints is not proposing methods to predict the response of joints under cyclic loading and, in particular, to ensure a sufficient ductility of the joint under such a loading condition. Accordingly, if a designer wants to use dissipative joints, it is required to proceed to a design supported by testing which is not a practical solution in real-life projects.

It is the reason why designers are generally considering the joints as non-dissipative zones and design the joints accordingly. But designing the joints as "non-dissipative" is not an easy task as the possible overstrength and strain hardening effects which can occur at the level of the dissipative zones has to be taken into account. If reference is made to the present draft of EN 1998-1, the criterion to be respected is the following:

$$M_{Rd,j} \geq \gamma_{ov}.\gamma_{sh}.M_{Rd,\mathrm{dz}} \tag{1}$$

where $M_{Rd,j}$ is the bending resistance of the joint, γ_{ov} is the overstrength coefficient (recommended value = 1,25), γ_{sh} is the strain hardening coefficient (recommended value = 1,1) and $M_{Rd,dz}$ is the bending resistance of the dissipative zone. Accordingly, the bending resistance of the joint has to be at least equal to 1,38 times the bending resistance of the dissipative zone which is easy to say but difficult to reach in practice.

For this reason, a European RFCS project named EQUALJOINTS (Landolfo et al., 2016) was developed with the objective of developing and prequalifying three types of dissipative and/or

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non-dissipative bolted joints that are commonly used in European practice at the image of what was already done in other countries like in US with its specific standard ANSI/AISC 358-05, 2005.

The three investigated joint configurations are presented in Figure 1: unstiffened extended end-plate beam-to-column joints (a), stiffened extended end-plate beam-to-column joints (b) and haunched beam-to-column joints (c).

The design criteria developed within EQUALJOINTS project aim at defining and characterising the hierarchy requirements among the strengths of macro-components (e.g. the web panel, the connection, the beam and the column), and their sub-components (e.g. end-plate, bolts, welds, etc.), as well. According to design procedure developed within the project, the joint is considered as made of three macro-components (i.e. the column web panel, the connection zone, and the beam zone – see Figure 2) and different performance objectives are defined for each zone according to the dissipative zone(s) to be activated. In particular, the performance objectives which can be contemplated for the joint (made of the web panel + the connection) are summarized hereinafter:

- Full strength joint: all the plastic demand is concentrated in the connected beam, leaving the connection and the web panel free from the damage;
- Equal strength joint: the plastic demand is balanced between the joint and the connected beam;
- Partial strength joint: all the plastic demand is concentrated in the joint.

It should be stated that both EN 1993-1-8 and EN 1998-1 do not consider the case of equal strength joint, which is proposed within the EQUALJOINTS project as an intermediate performance level. According to the current EN 1998-1 classification, an equal strength joint falls on the category of partial strength.

Moreover, in function of the resistance of the connection and column web panel for both equal and partial strength joint, additional performance objectives can be introduced:

- Strong web panel: all the plastic demand is concentrated in the connection (partial strength joint) or in the connection and in the beam (equal strength joint);
- Balance web panel: the plastic demand is balance between the connection and the column web panel (partial strength joint) or in the connection, in the web panel and in the beam (equal strength joint);
- Weak web panel: all the plastic demand is concentrated in the column web panel (partial strength joint) or in the web panel and in the beam (equal strength joint).

The investigated joint configurations have been prequalified for seismic loading situation through experimental, numerical and analytical investigations considering different possible performance objectives according to the considered joint configurations and the structural typologies where the considered joint is met (i.e. moment resisting frames, dual concentrically braced frames and dual eccentrically braced frames).

Within the present paper, a summary of the investigations conducted on the unstiffened extended end-plate beam-to-column joint configuration is proposed. In a first section, the investigated joint configuration is described in details. Then, the conducted experimental tests are detailed, providing the main outcomes from the test results. Finally, comparisons to existing analytical models and, in particular, to the analytical approach as proposed in the Eurocodes are performed.

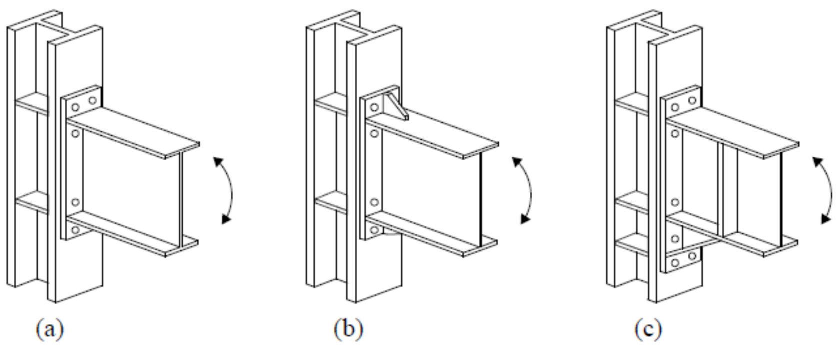


Figure 1. Bolted beam-to-column joint configurations investigated in the framework of the EQUALJOINTS RFCS project

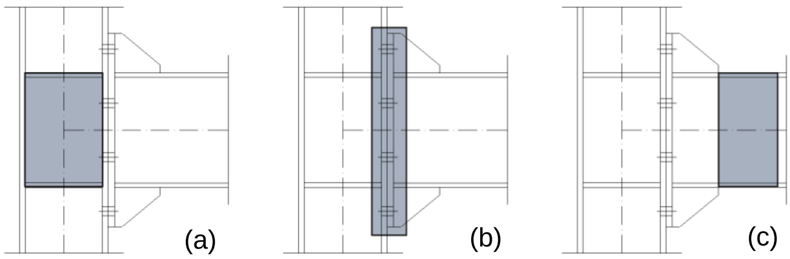
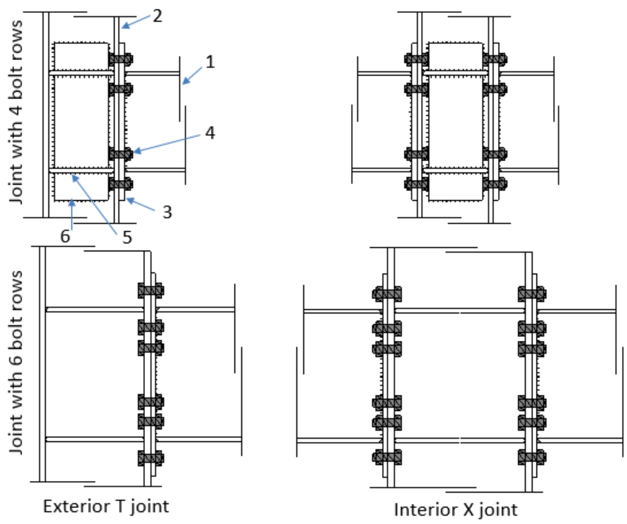


Figure 2. Macro-components at the joint level according to EQUALJOINTS design procedure: a) web panel, b) connection and c) beam.

Investigated joint configuration

The investigated unstiffened extended end-plate joint configuration is described in Figure 3. The connection is symmetrical according to the beam axis and, depending on the beam depth, 4 or 6 bolt rows can be adopted. The use of the additional plates to reinforce the column web is an option while the use of the continuity plates (transverse column stiffeners) is recommended for all cases.



1: beam	3: end-plate	5: continuity plates
2: column	4: bolts	6: additional plates

Figure 3. Description of unstiffened extended end-plate joints

The unstiffened end-plate beam-to-column joints have been prequalified for different beam depth, considering different performance objectives covering a wide range of possible applications.

In particular, it has been demonstrated that the use of extended unstiffened end-plate bolted joints as prequalified in the EQUALJOINTS project can be used for structural systems where the demand in terms of ductility remains limited. Accordingly, the use of these joints is particularly recommended in the “moment resisting spans” of dual concentrically or eccentrically braced structures. Also, with such a joint configuration, only equal-strength or partial-strength joints and balance or weak column web panel can be contemplated.

Table 1 summarises the limit values for the prequalified data. These values are closely linked to the limits met in the test campaign conducted in the EQUALJOINT project and summarised in the next section. However, it can be observed within Table 1 that a wide range of structural solutions met in practice can be covered respecting these limit values.

The full design procedure describing how to fix the dimensions of the different joint component and how to characterise the joint properties is provided in (Landolfo et al., 2018b).

Elements	**Parameters**	**Application range**
Beam		
	Depth	Maximum = 600mm
	Span-to-depth ration	Maximum = 23, Minimum = 10
	Flange thickness	Maximum = 19mm
	Material	From S235 to S355
Column		
	Depth	Maximum = 550mm
	Flange thickness	Maximum = 31mm
	Material	From S235 to S355
Beam/column depth		
End-plate		
	Thickness	18-25mm
	Material	From S235 to S355
Continuity plates		
	Thickness	Equal or larger than the thickness of the connected beam flange
	Material	From S235 to S355
Additional plates		
	Thickness	Only required if strong web panel is targeted – thickness defined according to EN 1993-1-8
	Material	From S235 to S355
Bolts		HV or HR
	Size	from M27 to M36
	Grade	10.9
	Number of bolt rows	4 for beam depths up to 450mm – 6 for higher beam depths
Welds		
	End-plate to beam flanges	Reinforced groove full penetration (Figure 4)
	Continuity plates to column flanges	Groove full penetration (Figure 4)
	Additional plates to column flanges	Groove full penetration (Figure 4)
	Other welds	Fillet welds: throat thickness is greater than 0.55 times the thickness of the connected plates.

Table 1. Limit values for prequalified data

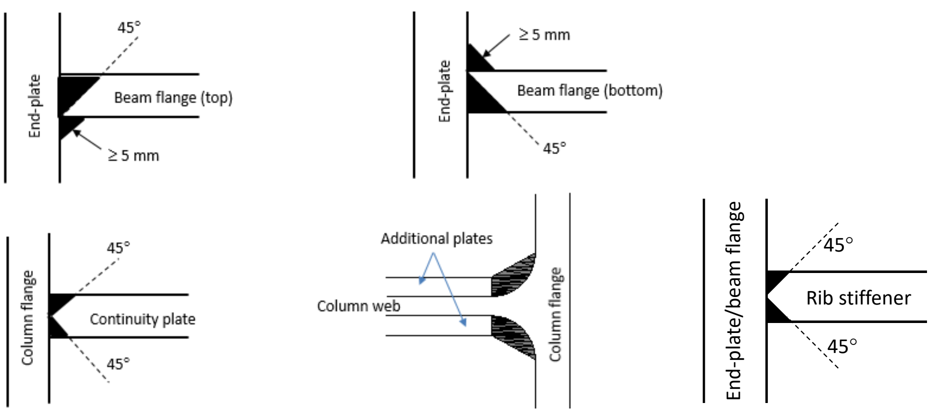


Figure 4. Details of the groove full penetration welds

Experimental tests

In total, 24 experimental tests including three different beam depths (IPE360, IPE 450 and IPE600) have been performed on unstiffened extended end-plate beam-to-column joints as indicated in Table 2:

- 18 tests on single-sided unstiffened extended end-plate joint configurations and;
- 6 tests on double-sided unstiffened extended end-plate joint configurations.

The definition of the parameters in the test nomenclature reported in Table 2 is as follows:

- E1, E2 and E3 are joint configurations with respectively IPE360, IPE450 and IPE600 beam(s) – see Figure 5;
- TB and XW are respectively single-sided and double-sided joint configurations;
- E & P reflect Equal strength joint and Partial strength joints respectively;
- PP are partial strength joints with shot peening for the welds;
- M = Monotonic loading protocol;
- Ci = Cyclic AISC loading protocol – Test "i";
- CE = Cyclic "EQUALJOINTS" loading protocol.

The objectives of these tests were (i) to demonstrate the ability of the proposed joint configuration to exhibit appropriate properties in case of seismic loading in terms of resistance and ductility and (ii) to investigate the effects of the loading protocol use for the tests and of the use of welding post treatments on the global response of the joints.

The main geometrical properties of the tested joints are reported in Figure 5. All joints are made of S355 steel grade elements. M27, M30 and M36 10.9 prestressed bolts are respectively used E1, E2 and E3 joint configurations.

There are at least 6 cyclic tests using the AISC loading protocol for each tested joint configuration as indicated in Table 2. For E1 and E2 joint configurations, monotonic tests are performed in order to evaluate the influence of cyclic loading on the joint response. For E3 joint configuration, there is one cyclic test with an alternative loading protocol developed in the framework of the EQUALJOINTS project. Finally, the effect of shot peening (Psp) treatment applied to the welds on the ductility of the three investigated joint configurations is investigated.

In addition, different tests on the base materials met within the tested specimens have also been performed:

- Coupon tensile tests;
- Charpy tests (except for the bolt material);
- Cyclic tests (except for the bolt material) and;
- Tightening tests for the bolts to determine the friction coefficient k.

Also, for all the tested specimens, the actual geometrical properties have been measured, including the dimensions of the welds, of the profiles, of the end-plates and of the stiffeners.

The "E1" joint configurations with the smallest beam were tested at the University of Naples Federico II while the "E2 and E3" joint configurations with the biggest beams were tested at the University of Liège. Within this section, only a summary of the obtained results are reported; all the details are made available in (Landolfo et al., 2018a).

Tested unstiffened extended end-plate beam-to-column joints		
E1-TB-E-M	E2-TB-E-M	E3-TB-E-CE
E1-TB-E-C1	E2-TB-E-C1	E3-TB-E-C1
E1-TB-E-C2	E2-TB-E-C2	E3-TB-E-C2
E1-TB-P-C1	E2-TB-P-C1	E3-TB-P-C1
E1-TB-P-C2	E2-TB-P-C2	E3-TB-P-C2
E1-TB-PP-C	E2-TB-PP-C	E3-TB-PP-C
E1-XW-P-C1	E2-XW-P-C1	E3-XW-P-C1
E1-XW-P-C2	E2-XW-P-C2	E3-XW-P-C2

Table 2. Unstiffened extended end-plate beam-to-column joint specimens

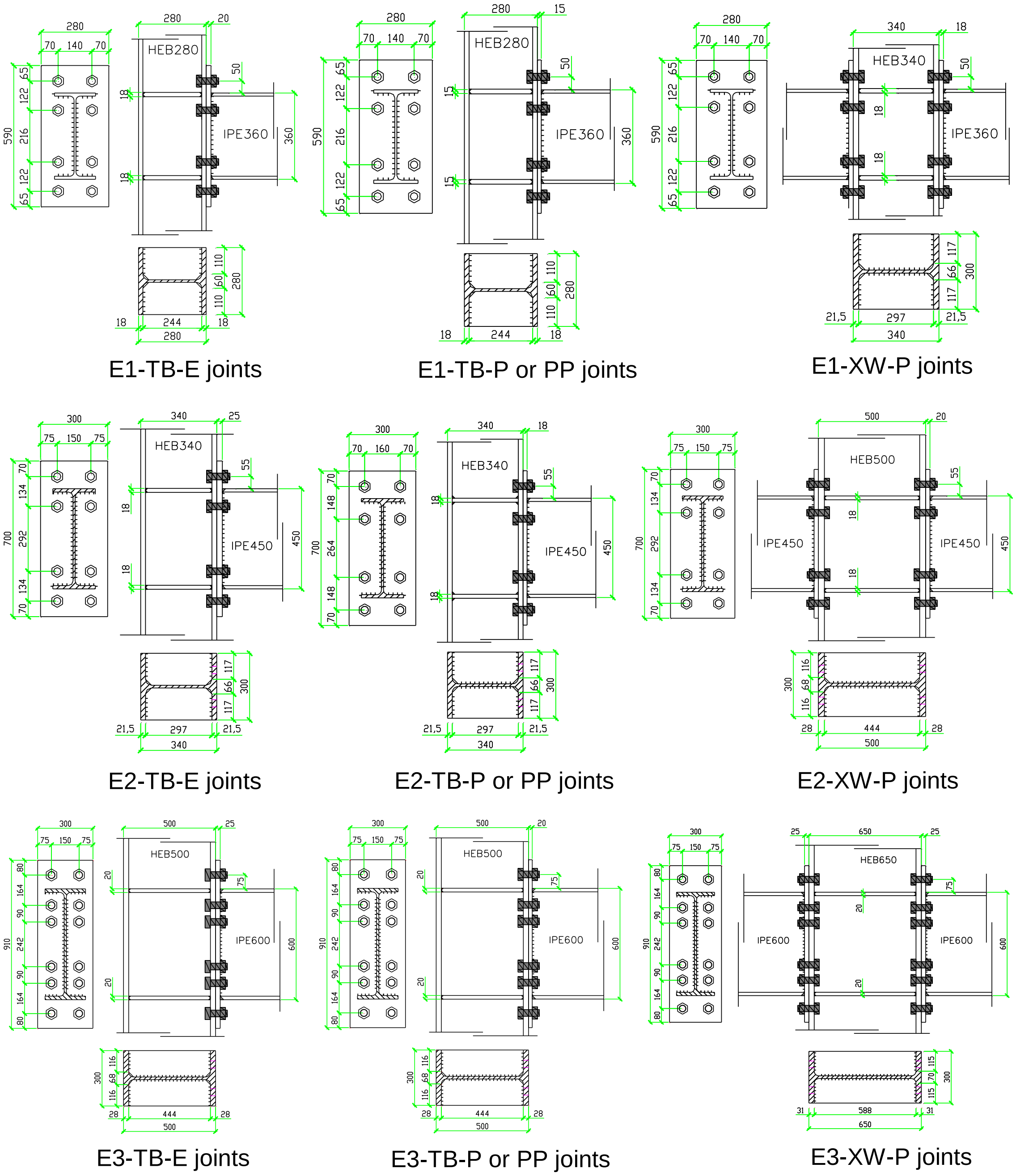


Figure 5. Main properties of the unstiffened extended end-plate beam-to-column joint configurations.

An illustration of the results obtained for all the performed tests is provided in Figure 8.

The tests have been achieved on « single and double sided joint configurations » involving one or two connection(s). The response of this connection(s) may be represented in the form of a moment-rotation curve ($M_{b,Ed}$-φ_c curve); the moment is the one at the interface between the column flange and the endplates while the rotation (in fact a relative rotation) is the difference between the rotation of the beam axis and that of the column axis at the connection level.

Besides that, another main source of joint deformability is associated to the shear of the column web panel, which may be represented by a $V_{wp,Ed}$- γ curve where $V_{wp,Ed}$ is the shear force in the panel and γ its shear deformation.

In Figure 8, the shear one has been expressed in terms of an $M_{b,Ed}$-γ curve, $M_{b,Ed}$ being the bending moment in the connection (the one used in the connection $M_{b,Ed}$-φ_c curves) for an easier comparison. This enables a direct estimation of the relative importance of both the φ_c and γ

rotations. The joint response ($M_{b,Ed}$-φ_j curve), according to the definition provided in Eurocode 3 Part 1-8, is drawn; it is obtained by adding φ_c and γ rotations. Finally, the "assembly response" characterising the tested specimens are reported in the form of a $M_{b,Ed}$-θ curve in which θ designates the interstorey drift ratio (also called "chord rotation") obtained by dividing the deflection under the applied load at beam end by the physical length of the beam.

Globally, the test results show that the specimens' behaviour corresponds to the one expected through their design, i.e.:

- a significant rotation capacity is reached for all the specimens - the maximum rotation is about 50-70 mrad;
- the energy is mainly dissipated through three zones: the web panel, the end-plate and the beam;
- the ductility of the end-plate with a smaller thickness (allowing activating a mode 1 failure mode) is sufficient if reference is made to the Eurocode (i.e. it is possible to reach a rotation capacity of 35mrad);
- the contribution of the web panel to the global deformation of the specimen can be significant (from 30 to 60%) and excessive if comparison is made with EN 1998-1 recommendations.

Some more detailed aspects can anyways be highlighted:

About the joint classification

The overstrength effects strongly affect the global behaviour of the specimens and, in particular, the components which are activated at yielding as all the components are not made of the same material. This leads to the fact that the identification of the weak zone of the joint using the nominal properties could not be in line with the actual behaviour of the joint as illustrated in Figure 6. This could lead to difficulties in predicting the ductility of the joint and of the contribution of the column web panel to the global deformation of the joint (key parameter when considering a seismic design). This confirms the need of mastering this overstrength effect.

About the ductility of the end-plate

The ductility of "thin" end-plate (associated to a failure mode 1, see EN 1993-1-8) looks to be appropriate; at least, the minimum rotation capacity of 35 mrad required according to EN 1998-1 can be reached. On the other hand, for "intermediate" end-plates (associated to a failure mode 2), the end-plate was not fully activated during the test due to the apparition of cracks between the beam flange and the end-plate (cracks 2 and 3 in Figure 7).

About the crack locations

The apparition of cracks 2 and/or 3 (Figure 7) considerably limit the ductility of the connection. The apparition of crack type 2 before the yielding of the beam section could be explained by a concentration of stresses at the free sides of the beam flanges. For crack type 3, it is assumed that the full strength welds (with a filler material respecting the nominal properties of the beam) becomes "partial strength" ones due to the overstrength of the beam material. Therefore, the weld fails before the yielding of the beam, but for a bending moment greater than the nominal plastic capacity of the beam.

About the weld peening

The test results on partial strength extended unstiffened joints fabricated using shot peening (i.e. those identified with the subscript "pp") for the welds of the connection clearly show that this treatment does not positively influence the response of the joints as expected.

About the loading protocol

The influence of the loading protocol on the joint response is seen to be rather negligible as same levels of resistance and ductility were obtained.

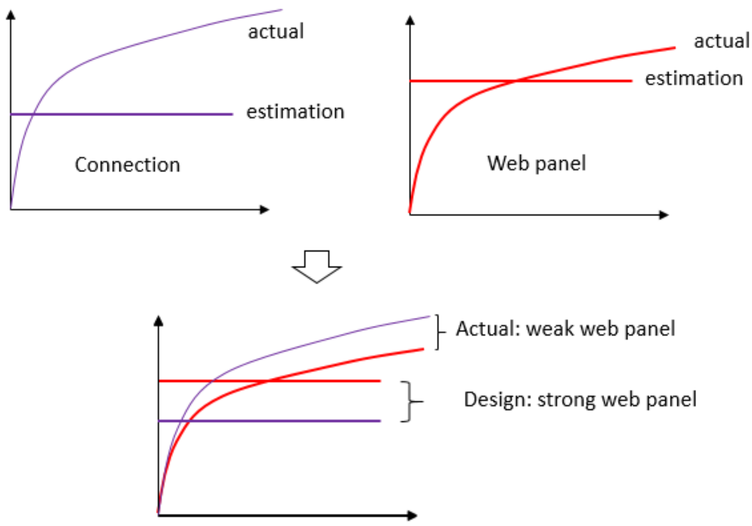


Figure 6. Change of the joint classification

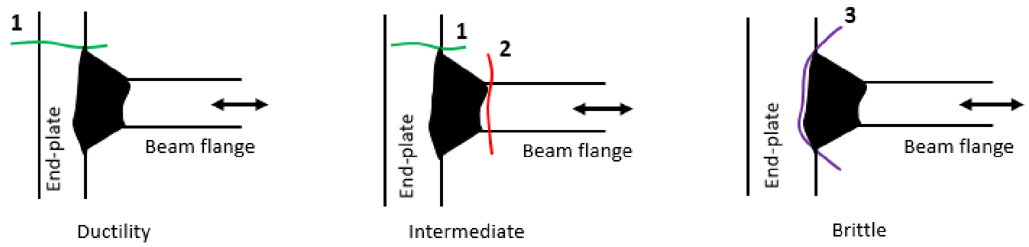


Figure 7. Crack locations

Comparisons to existing analytical models

The EN 1993-1-8 rules dealing with the characterisation of the steel joints properties (CEN, 2005; Jaspart and Weynand, 2016) have been used to predict the tested joint characteristics considering the actual material properties of the different joint components and the so-obtained results have been compared to the experimental ones (see Figure 8).

In the $M_{b,Ed}$- γ diagrams, two curves are reported, by assuming successively that the web panel is sheared (i) along a height equal to the distance between the centres of gravity of the beam flanges (according to EN 1993-1-8 - Figure 6.15) and (ii) along a “maximum shear” height resulting from the application of EN 1993-1-8 assembly procedure (section 6.2.7.2).

Globally, it can be observed that the analytical predictions obtained by EN 1993-1-8 in terms of resistance and stiffness agree quite well with the experimental results, for all the connections and for the joints. A similar conclusion is drawn for the column web panels as far as the panels are assumed to have depth equal to the “maximum shear” height resulting from the application of the Part 1-8 assembly procedure (section 6.2.7.2). On the contrary, an unsafe estimation of the web panel resistance is obtained when the height of the panels is taken as equal to the distance between the centres of gravity of the beam flanges (according to EN 1993-1-8 - Figure 6.15). This result highlights that for joints where the contribution of the inner bolt rows is significant the simplified approach given by Figure 6.15 of EN 1993-1-8 should be avoided.

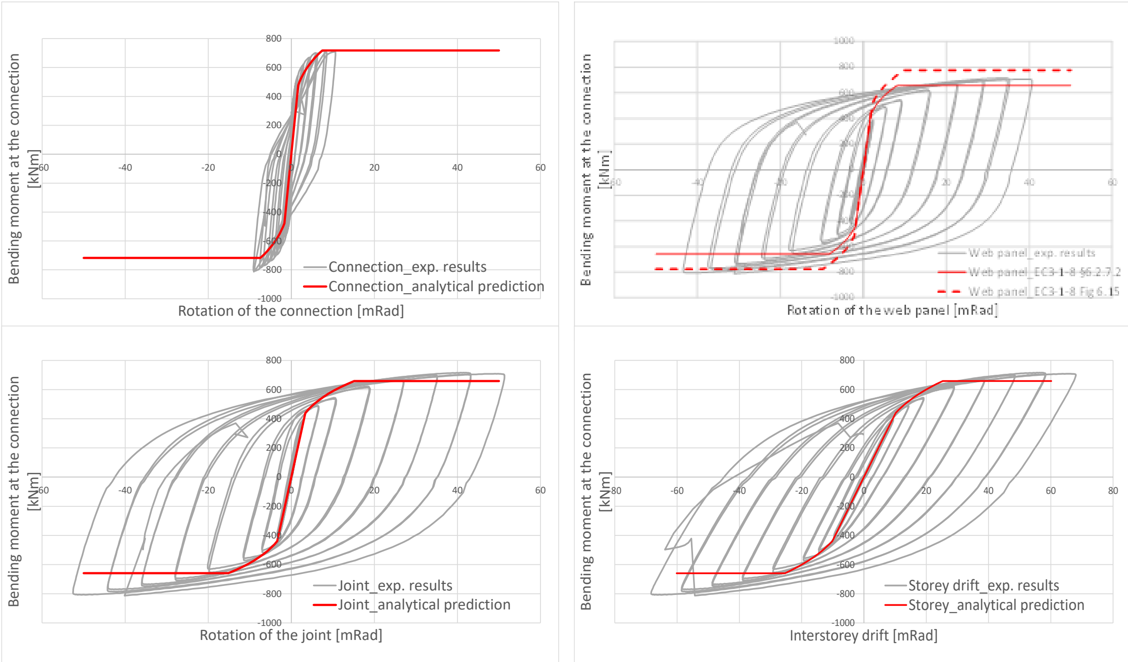


Figure 8. Experimental response vs. EC3:1-8 moment-rotation curves of E2-TB-E-C1 joints

Conclusions

Within the EQUALJOINTS project, three bolted joint configurations currently used in Europe have been prequalified for their used in seismic area. Within the present paper, the investigations conducted on the unstiffened extended end-plate beam-to-column joints have been summarized. In particular, the experimental tests conducted on this joint configuration have been briefly described and the main outcomes from the conducted tests have been summarized. Also, a comparison between analytical predictions obtained using the Eurocodes model and the experimental results have demonstrated the ability of the proposed model to accurately predict the joint mechanical properties.

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