Component based Finite Element Design of Seismically Qualified Joints

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Abstract. This paper describes the principles of Component Based Finite Element Modelling (CBFEM) and the system response quantity and application features for design of prequalified joints for seismic applications. The elements are analysed by geometrically and materially non-linear analysis with imperfections (GMNIA). The proper behaviour of components, e.g. of bolts, anchor bolts, welds etc., is treated by introducing components representing its behaviour in term of initial stiffness, ultimate resistance and deformation capacity. To show this process a contribution is prepared, which summarises the history of achievements of FEA application in structural connections. Contribution shows the currents trends in advanced modelling of connection components and differences of the research oriented and design-oriented models. Both have their roles in advanced analyses.

1. Introduction
The ductility demand on joints and connected members for use with special moment frames and intermediate moment frames differs between the different countries. The recently finished European research project EQUALJOINTS provides prequalification criteria of steel joints for the version of EN 1998-1:2025. The research activity covered the standardization of design and manufacturing procedures with reference to a set of bolted joint types and a welded reduced beam section with heavy profiles designed to meet different performance levels. There was also the development of a new loading protocol for European prequalification, representative of European seismic demand. The experimental campaign devoted to the cyclic characterization of both European mild carbon steel and high strength bolts achieved the required behaviour and four types of pre-standardised joints: the haunched bolted joints, the unstiffened extended end-plate bolted joints, the stiffened extended end-plate bolted joints, and the welded reduced beam section joints, see Figure 1.

Extended end-plate connections with haunches are usually used in steel moment-resisting frames. Its component-based design is based on Zoetemeijer’s work [1]. Gross [2] proposed to adopt a haunch depth equal to 0.33 times the beam depth, with an angle of the haunch equal to 30° to limit the haunch web slenderness. This assumption was based on the Whitmore theory of the propagation of internal stress in elastic system of about 30° slope. However, increasing the slope can be convenient because it allows reducing the size of the haunch as well as the design forces on the connection. This finding gives the simple detailing rules: the steel grade of the haunch should match that of the member, the flange size and the web thickness of the haunch should not be less than that of the member, and the angle of the haunch flange to the flange of the member should not be greater than 45°. The results experimentally reached within the EQUALJOINJTS project are summarised in [3].
The Extended stiffened end-plate bolted connections are most common among European steel fabricating industries and are widely used in European practice as moment-resistant joints in low and medium rise steel frames due to the simplicity and the economy of fabrication and erection. The design criteria and related requirements for bolted extended stiffened end-plate beam-to-column joints are deeply investigated and critically discussed and currently codified in EN 1993 [5] based on a parametric study based on finite element analyses. A capacity design procedure was developed unfortunately only in the framework of component method. It also accounts for the presence of ribs and is able to control the joint response for different performance levels.

Unstiffened extended end-plate joints are commonly used in steel construction to connect a steel I or H beam to a steel I or H column in the case where significant bending moments have to be transferred. This configuration allows an easy erection by bolting while welding of the end-plate to the beam is automated in shop. Their bending resistance is mostly lower than the bending resistance of the connected members. These joints are considered as partial strength [5], which is caused by the partial resistance character of the constitutive connections or of the column web panel in shear. Reaching an equal strength situation in which the plastic resistance of the joint is roughly equal to the plastic resistance of the beam section may be contemplated through an appropriate design. Their ductility in bending highly depends on the detailing of the joints, which influences the failure mode [6]. If the joint component governing the failure is a ductile one and if the resistance of the brittle active components is significantly higher a ductile joint response may be contemplated. In the opposite case no reliance should be made on the capacity of the joint to redistribute plasticity and in a seismic area to absorb energy.

For the welded reduced beam section moment resisting connections, also referred as Dog-bone, the two main strategies were adopted by strengthening the connection or weakening the beam. Among the different options for the profile of section reduction, the radius cut tends to exhibit a relatively more ductile behaviour, delaying the ultimate fracture [7]. The work showed that reduced beam section members are more prone to lateral torsional buckling due to the decreased area of their flanges. Further experimental and analytical research focusing on the application of deep columns [8] indicated that the presence of a composite floor slab can greatly reduce the amount of twisting developing in the column, as it offers bracing to the beam and reduces the lateral displacement of the bottom flange.

The presented document summarizes design guidelines for three types of bolted joints prequalified within EQUALJOINTS project: haunched, stiffened extended end-plate and unstiffened extended end-plate beam-to-column joints as well as welded dog bone joints. Information is provided about experimental setup and measurement instrumentation, performance parameters of the tested joints and non-linear moment-rotation response [10]. Design procedure for qualified joint types of technological requirements, description of the joint configurations, list of systems for which the joints are prequalified, list of limit values for prequalified data and design procedure are in [10].
The global analyses of steel structures are today carried out FEA and all the traditional procedures are not used any more. Currently, fast development of software ability of connection design by FEA and thousands of experiments are available for the validation process. In such situation the verification process performed through benchmark tests gains crucial importance. The source and the extent of such benchmark tests for the field of structural connections is yet to be established. To achieve this goal a set of small benchmark tests that can be used as a reference in the verification process of simulations was developed.

The recommendation for design by advanced modelling in structural steel is ready to be used in Chapter 5 and Annex C of EN 1993-1-5:2005 [11]. Development of modern general-purpose software and decreasing cost of computational resources facilitate this trend. The FEA of structural joints is the coming step in structural steel design. As the computational tools become more readily available and easier to use even by relatively inexperienced engineers the proper procedure should be employed when judging the results of computational analysis.

FEA for connections are used from 70s of last century as research-oriented FEA (RFEA). Their ability to express real behaviour of connections is making numerical experiments a valid alternative to testing and source of additional information about local stresses. Material model for FEA uses true strain stress-strain diagram. Validation & Verification (V&V) process of models is integral part of the procedure [12], and the FEA studied are based on the researcher’s own experiments. During preparation of CM for EN1998-1-3:2006 all basic components were deeply modelled [13]. Special attention was given to modelling of the T stub, which represents the end plate connections of beam to column joints, beam splices and column bases [14]. Last generation of FEA models of connections is utilised in studies focussed on application of high strength steel [15] and bolts [16] in the connections. Prediction of hollow section joints is based on experimental evidence approved by FEA numerical experiments, see e.g. [17]. Due to large variety of geometry some types were studied only numerically [18]. V&V of the FEA design models (DFEA) of steel connection design is native part of its preparation [19]. The detailed procedure for verification of CBFEM was prepared [11].

2. Validation & Verification

2.1. Validation

The CBFEM models of rigidity, load bearing capacity and deformation capacity of pre-designed joints were validated by Montenegro in [20] on set of experiments available from EQUALJOINT project [20]. The examples of structural solutions are in Figure 2. The results of validation of the failure mode are shown in Figure 3. The summary of validation of resistance and deformation capacity for 15% strain are shown in Figures 4 and 5.

![Figure 2. Joints used for V&V](image_url)

2.2. Verification

The CBFEM model was verified to CM according to Ch. 6 in EN1993-1-8:2006 [10] in [20]. The selection of results is presented in Table 1 and Figure 6. The results show the loss of accuracy of CM for larger joints, where the rough assumption of lever arm is guiding the accuracy.
Figure 3. Validation of failure mode of the extended end-plate joints with haunch E1-TS-F-C2 [3].

Figure 4. Validation of resistance on experiments from EQUALJOINTS project [3].

Figure 5. Validation of rotational capacity on experiments from EQUALJOINTS project [3].
### Table 1. Verification of CBFEM to CM

| Typology         | Resistance | Decisive component       |
|------------------|------------|--------------------------|
|                  | CM, $M_R$ [kNm] | CBFEM, $M_R$ [kNm] | CBFEM/CM% |
| Haunched joint   | 901.2      | 899.1                   | 1         | Endplate in bending |
|                  | 959.3      | 875.0                   | 10        | Endplate in bending |
|                  | 547.5      | 533.2                   | 3         | Column flange in bending |
|                  | 1389       | 1920                    | -27       | Column flange in bending |
|                  | 347.8      | 389.0                   | -11       | Endplate in bending |
|                  | 577.0      | 681.0                   | -15       | Endplate in bending |

### Figure 6. Validation of rotational capacity on experiments from EQUALJOINTS project [3].

3. Design

3.1. Prequalified joints

According to design procedure developed within the project EQUALJOINTS, the joint is considered as made of three macro-components, i.e. the column web panel, the connection zone, and the beam zone. Each macro-component is individually designed according to specific assumptions and then capacity design criteria are applied, in order to obtain three different design objectives defined to compare the joint: web panel and connection, strength to the beam flexural resistance, namely full strength, equal strength and partial strength joints. Full strength joints are designed to guarantee the formation of all plastic deformations into the beam, which is consistent with EN 1998-1:2005 strong column-weak beam capacity design rules. Equal strength joints are theoretically characterized by the contemporary yielding of all macro-components, i.e. connection, web panel and beam. Partial strength joints are designed to develop the plastic deformation only in the joint. In function of the resistance of the connection and column web panel for both equal and partial strength joint an additional classification can be introduced. For strong web panel the plastic demand is concentrated in the connection for partial strength joint, or in the connection and in the beam for equal strength joint. In balanced web panel the plastic demand is distributed between the connection and the column web panel for partial strength joint, and in the connection, in the web panel and in the beam for equal strength joint. For weak web panel the plastic demand is concentrated in the column web panel for partial strength joint or in the web panel and in the beam for equal strength joint.
The joint ductility depends on the type of failure mode and the corresponding plastic deformation capacity of the activated component [6]. Roughly the deformation capacity may be predicted by satisfying the CM’s developed criteria or may be calculated by CBFEM. The examples of design of two prequalified joint configurations described in EQUALJOINJTS project materials and in ANSI/AISC 358-16 [21] standard are presented below taking into account the behaviour of macro components separately.

3.2. Unstiffened extended end-plate joints prequalified in EQUALJOINJTS project

For sensitivity study a prequalified unstiffened extended end-plate joint was selected. The beam IPE 450 is connected to column HEB 300 by an extended end plate 25 mm thick with twelve M30 10.9 bolts, with and without web doubler 10 mm thick [6]. Steel grade S 355 was used for all plates. To determine the contribution of each macro component separately, the material diagram of the selected macro component was elasto-plastic while the rest of the joint was with only elastic material diagram. The strains at resistance of the whole joint, the column web panel in shear with web doublers only, and the bolted end-plate connection only are compared to the beam macro component only in Figure 7. The influence of each macro component on behaviour of the joint is presented in Figure 8, where column web panel with and without web doublers is shown. The joint behaviour shows higher resistance of the connection micro component.

![Figure 7](image_url)

**Figure 7.** The strains at resistance for a) the whole joint, b) the macro-components bolted end-plate connection only c) the macro-components column web panel in shear with web doublers only, d) the macro-components beam macro component only.
3.3. Welded reduced beam section joint prequalified in ANSI/AISC 358-16
A prequalified welded reduced beam section joint according to ANSI/AISC 358-16 [21] was selected for this study. The beam IPE 450 is connected to column HEB 300 by butt welds at flanges and fin plate 12 mm thick with three preloaded M30 10.9 bolts, with and without web doubler 10 mm thick. Steel S 355.

Figure 8. Influence of macro-components, the column web panel in shear with web doublers the bolted end-plate connection and beam to behaviour of the whole joint.

The strains at resistance of the whole joint and macro component column web panel in shear with web doublers only are shown in Figure 9. The influence of each macro component to behaviour of the joint is presented in Figure 10, where column web panel with and without web doublers is shown. The joint shows well optimised resistance of the joint micro components.
Figure 10. Influence of macro-components, the column web panel in shear with web doublers the bolted end-plate connection and beam to behaviour of the whole joint, a), b) joint, c) M-\(\varphi\) diagram.

4. Conclusion
The global analyses of steel structures are today carried out by FEA and all the traditional procedures are not used any more. In new generation of structural Eurocodes after 2025 is expected to stress the principles of safety of applications of FEA in EN1993-1-14:2025 Design of steel structures - Part 14 Design by finite element by System response quantity process for numerical experiments and design models separately. The FEA analyses of the structural steel connections is replacing the curve fitting and component design methods. For its proper use it is necessary to apply a good Validation and Verification procedures with well-defined hierarchy to allow a safe use.
The presented results show the good accuracy of CBFEM applied in [22] validated to EQUALIJOINTS experiments and verified to CM in Ch. 6 in EN 1993-1-8:2006 [10] and features to consider the behaviour of macro components separately.

For end-plate joints EN 1993-1-8:2006 specifies that the compression centre is located in the middle of thickness of beam flange, or at the tip of the haunch in case of haunched joints. Experimental and numerical results showed that the location of compression centre depends on both the joint type and the rotation demand due to the formation of plastic modes with different engagement of each joint component. According to the proposed design procedure and based on both experimental and numerical results from literature [3,4,11] contact at about the centroid of the section made by the beam flange and the rib stiffeners is expected, for the stiffened endplate joints or at 0.5 the haunch height in case of haunched joints [4]. This rough assumption may be easily précised by CBFEM procedure, which gives correct values during loading and initial yielding of parts of joint.

Prediction of the cyclic behaviour of beam-to-column joints is for CM prepared by compressing component cycles to joint behaviour. For application of CBFEM in dynamic analyses for each type of the European prequalified joint four shape parameters of cycling are calibrated by modified Richard Abbott method with and without pinching effect [23].

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