A Theoretical Approach for the Prediction of the Rotational Capacity of Steel Column Base Joints

M. Latour & G. Rizzano
Department of Civil Engineering, University of Salerno

SUMMARY:
The last version of Eurocode 8 has opened the door to the design of Moment Resisting steel Frames (MRFs) promoting the dissipation of the seismic input energy in the joints. Within this framework, in order to provide the basic tools for the prediction of the main parameters characterizing the monotonic and cyclic behaviour of joints, in the last two decades, many experimental and theoretical studies have been carried out. As a result of this effort, in Eurocode 3 the formulations for predicting stiffness and resistance of common joint typologies have been codified. Despite this, dealing with the rotation capacity of base joints, there is still a lack of knowledge. In order to overcome the knowledge gap surrounding the evaluation of the plastic rotation capacity of base joints, in this paper the development of a mathematical model able to evaluate the ductility supply of exposed base plate joints is presented. Within the framework of the component method, a mechanical approach for predicting the rotational capacity of base plate joints is set up starting from the definition of the ductility supply of the single joint components. Furthermore, the proposed mechanical model is validated by means of the comparison with three full scale experimental tests carried out at the University of Salerno.

Keywords: Base Joints, Ductility, Model, Component Method

1. INTRODUCTION

Aiming to obtain a cost/effective structural design, current seismic codes are based on performance criteria. In case of MRFs, the classical protection strategy requires the dissipation of the earthquake input energy through the plastic engagement of the so-called dissipative zones. Within the framework of the capacity design, the location of the dissipative zones can be controlled by means of a proper design of beams, columns and joints which allows to select the elements that have to withstand plastic deformations in case of rare seismic events. Generally, columns are prevented from plasticization by designing their cross-section according to the members' hierarchy criterion and the plastic hinges are located into beams or joints. In particular, when full strength connections are adopted, joints are designed to be over-strength with respect to the connected members and, therefore, the plastic hinges are located at beam ends. Conversely, when partial strength connections are employed, as far as the joints are weaker than the beam, the plastic zones are concentrated in the connections. This last approach is allowed by Eurocode 8 which clearly states that the plastic hinges can be developed at the end of the beam or in the joint. Therefore, it is clear that the joint behaviour has a significant influence on the overall response of the frame and must be included into the structural model in terms of stiffness, resistance and rotational capacity. According to Eurocode 3 part 1.8, depending on the assumed joint behaviour, three methods for frame global analysis are defined: elastic, rigid-plastic and elastic-plastic. In the case of elastic global analysis only the joint stiffness properties are relevant for the modelling. In the case of rigid-plastic analysis, flexural strength and the rotational capacity of joints are needed. In case of elastic-plastic analysis the complete knowledge of the joint moment-rotation curve up to failure is required.

The basic approach to joint design proposed by Eurocode 3 is the so-called component method in which joints are broken down into basic components characterizing strength, deformability and ductility. In its last version, the code provides the information for twenty components but in twelve cases no indication on the rotation capacity are given. Such a lack of information appears in contrast with Eurocode 8 provisions. In fact, in order to obtain a determined global ductility of the frames, the code requires the knowledge of the rotational capacity of joints. In particular, for Ductility Class Medium (DCM) and Ductility Class High (DCH) MRFs, Eurocode 8 states that connections have to sustain a drift angle of 25 mrad and 35 mrad respectively. Therefore, when plastic zones are concentrated in connections, the assessment of their rotational capacity becomes an aspect of paramount importance.

In last decade some models have been suggested to characterize the rotational capacity of steel beam-to-column joints and recently a new classification, based on ductility, to be introduced in EC3 has been proposed (Jaspart, 2002). This classification should integrate the existing ones based on stiffness and strength. The author proposes to classify connections as ductile, semi-ductile and brittle, depending on their capacity to allow redistribution of internal actions. Dealing with the modelling of the joints rotational capacity several theoretical and empirical approaches are available in technical literature for predicting the ductility of welded and bolted beam-to-column joints, but no models for defining the rotational capacity of base plate joints exist. In particular, regarding the prediction of the ductility of beam-to-column joints, in more recent times, many researchers have proposed models based on the component method. Within these approaches, starting from the definition of the ductility of the single joint components, a mechanical model is assembled in order to find the rotational capacity of the joint (Faella et al., 2000; Piluso et al., 2001; Kuhlmann & Kuhnemund, 2000; Beg et al., 2004; Simoes Da Silva, 2001; Girao Coelho et al., 2004; Iannone et al., 2011; Latour et al., 2011).

Within this framework, the present paper proposes to extend the approach for the prediction of ductility of beam-to-column joints to the case of extended base plate joints for which, up to now, only few research studies have been carried out (Thambiratnam & Paramasivam, 1986; Jaspart & Vandegans, 1998; Gomez et al., 2010; Melchers, 1992). In order to reach this goal, in the first step, the ductility supply of the main sources of deformability of a base plate joint are characterized by properly accounting for the existing scientific literature. Then a procedure able to evaluate the overall base plate rotational capacity starting from the prediction of the ductility of the single joint components is presented. Finally, the accuracy of the proposed approach is verified by means of comparison with experimental results carried out by the same authors at the laboratory on materials and structures of the Salerno University.

2. DEFORMATION CAPACITY OF THE JOINT COMPONENTS

The scope of the paper is the proposal of a theoretical model able to predict the rotational capacity of base plate joints. Therefore, in order to apply the component method the following three steps have to be carried out (Jaspart, 2002):

- Identification of the sources of deformability;
- Characterization of the ductility of each joint component;
- Assembly of the mechanical model.

According to the component method codified in last version of EC3 the base plate connection depicted in Fig.1 can be represented by considering four sources of deformability, two in the compression side and two in the tension side, i.e. the concrete in compression including grout, the column flange and web in compression, the base plate in bending and the anchor bolts in tension. As far as the ductility is of concern, in the following, the ductility of each joint component will be characterized.

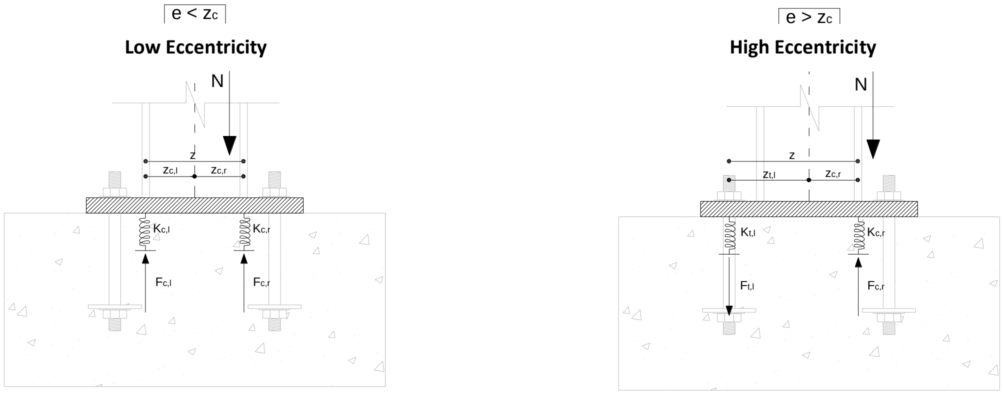

Figure 1. Mechanical model of Base Plate joints proposed by EC3

2.1. Concrete in compression including grout

A column base joint is the set of the mechanical elements devoted to transfer shear forces, bending moments and normal load to the foundations. In general, in order to withstand to bending actions, in the low eccentricity range the joint is assumed to react with an internal couple constituted by two equivalent T-stubs in compression located under the column flanges. Conversely, in the high eccentricity range the mechanical model is constituted by an equivalent T-stub in tension under the column tension flange and a T-stub in compression under the column compression flange. The T-stub in compression is modelled in EC3 with the joint component called "concrete in compression including grout", which is intended to account for the overall contribution of base plate, concrete constituting the foundation and bedding grout.

In past decade, several scientific works have been devoted to model the strength and stiffness of concrete and grout in compression (Sokol & Wald, 1997; Steenhuis & Bijlaard, 1999). In particular, the experimental tests carried out by (Sokol & Wald, 1997) on isolated T-stubs subjected to compression loadings, demonstrate that the monotonic force-displacement behaviour of concrete in compression is usually characterized by a steep elastic stiffness and a limited deformation capacity which is usually contained in the magnitude of few millimetres. Dealing with the deformation capacity of concrete in compression including grout, EC3 does not give any information because the deformation capacity of concrete is so limited that can be neglected in practical cases. In fact, concrete and grout in compression are very stiff compared to anchor bolts and base plate in bending.
For this reason, in the following, the contribution of the deformability of concrete in compression will be neglected, assuming that the spring in compression provides only a limitation to the resistance of the joint. Therefore, only the case of connections failing in the tension zone due to the collapse of anchor bolts or of the base plate in bending will be considered. In fact, it is only under this assumption that the base plate joint can exhibit sufficient rotational capacity.

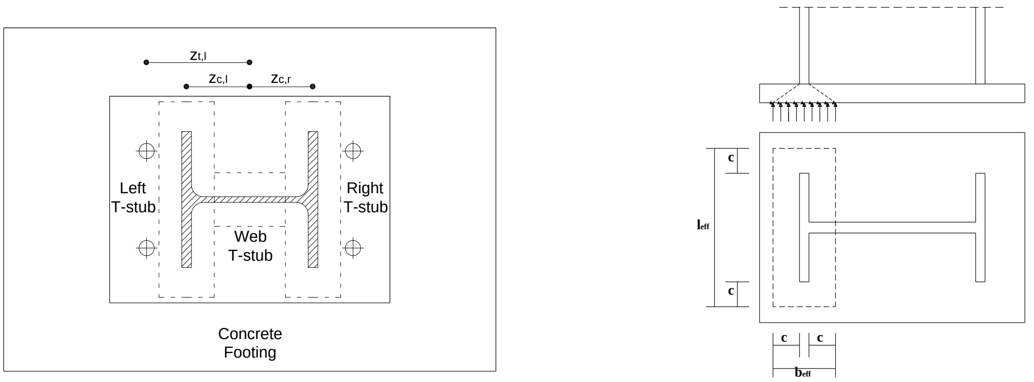

Figure 2. Left: Individuation of the T-stubs according to EC3. Right: Definition of the equivalent rigid plate

2.2. Anchor bolts in tension

In case of base plate joints, anchor bolt in tension is one of the most important components. In classical T-stub theory, which is developed for beam-to-column connections, three basic failure modes are hypothesized: mode 1, namely plate failure with formation of four plastic hinges, mode 2, namely contemporary failure of plate and bolts and mode 3, namely bolt failure. In case of exposed base plate joints another aspect, which normally is negligible in case of beam-to-column joints, has to be taken into account. In fact in many cases, in base plate connections the anchor bolt elongation is such that, in comparison to the flexural deformability of the base plate, no prying forces develop. In this case, the plate works as a cantilever beam loaded by the anchor force failing due to the formation of a plastic hinge at the clamped section. In Eurocode 3, such particular failure mechanism is called mechanism 1*, i.e. plate failure without prying forces. The deformability of anchor bolts obviously can affect also the rotational capacity of the base plate joint. Bolt deformability may reduce the joint resistance, but has a beneficial effect on the ductility supply. In fact, a greater deformability of the anchor results in a higher rotational capacity of the base plate joint. In EC3 no information are given with reference to the ultimate displacement of anchor bolts. In addition, since classically bolts are designed to remain in elastic range, in technical literature there are only few studies dealing with the characterization of the ductility supply of bolts (Girao Coelho et al., 2004). The plastic deformation supply of an anchor bolt axially loaded can be defined according to the following expression:

$$\delta_{pb} = \varepsilon_{pb} L_b \tag{2.1}$$

where ε_{pb} is the ultimate plastic deformation of the material composing the bolt and L_b is the bolt length. For anchor bolts, according the Eurocode 3, the conventional length can be defined in the following way:

$$L_b = \mathbf{8}d_b + t_g + t_p + t_w + \frac{t_n}{\mathbf{2}} \tag{2.2}$$

where d_b is the bolt diameter, t_g, t_p, t_w and t_n are the thickness of the grout layer, base plate, washer, and nut respectively. In (Girao Coelho et al., 2004), within a wide experimental program dealing with the assessment of the behaviour of isolated T-stubs subjected to tension, four series of tests on high strength bolts axially loaded have been carried out. The average ultimate deformation resulting from experimental tests indicated by the author for short-threated bolts of 8.8 and 10.9 class, is contained in the range between 0.11-0.13. Furthermore, in (Beg et al., 2004)., the authors indicate a ultimate bolt deformation capacity equal to 0.1. As far as a common approach does not exist, in this paper a conservative value equal to 0.1 is adopted.

2.3. Base plate in bending

Experimental results demonstrate that stiffness, resistance and ductility of base plate connections strongly depend on the base plate geometry, i.e. width, position of the bolt and thickness. It is well known that the common approach adopted to model bolted plates in bending is the study of the response of the so-called equivalent T-stub, i.e. the assemblage of two Tee elements fastened through the flanges by means of bolts. In past 30 years T-stubs in tension have been experimentally and theoretically studied by many authors and several models are available in scientific literature to predict their monotonic and cyclic behaviour (Clemente et al., 2005; Faella et al., 1998a; Piluso & Rizzano, 2008; Leon & Swanson, 2000). In last version of EC3, the T-stub failure mode depends not only on the resistance of fasteners and plates, but also depends on the stiffness of the anchors, which govern the development or not of the prying forces. If the bolts are sufficiently stiff with respect to the fastened plate, the possible failure mechanisms are three: type-1 (plate failure), type-2 (contemporary bolt and plate failure) and type-3 (bolt failure). Conversely, if bolt elongation is such that prying forces cannot develop, only two failure mechanisms may occur: type-1* (Plate failure without prying forces) and type-3 (bolt failure).

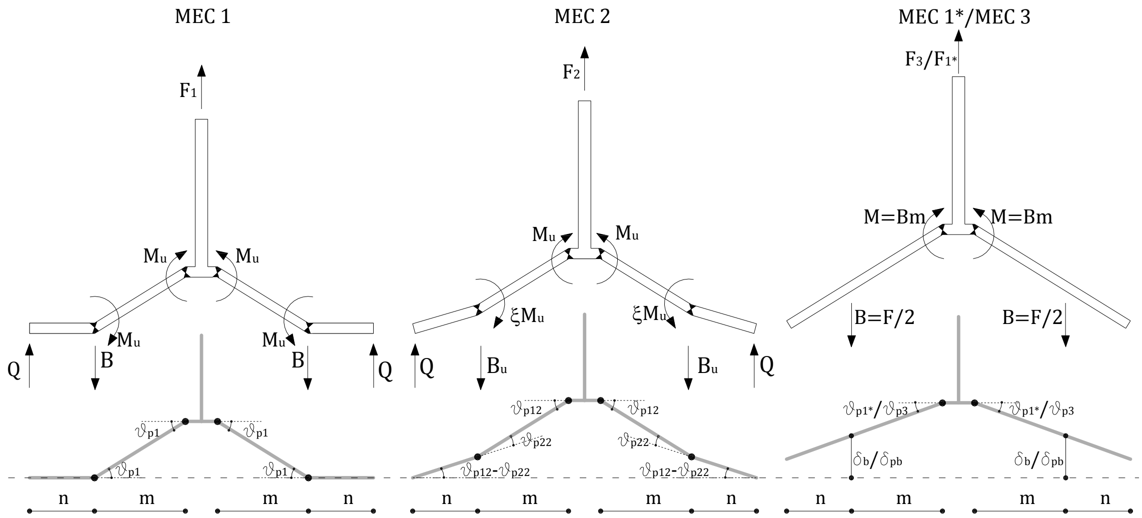

Figure 3. Kinematic collapse mechanism of T-stub in tension

Dealing with the evaluation of the T-stub ductility it is clear that the knowledge of the failure mode is of paramount importance. In fact, the kinematic mechanism at collapse governs not only the resistance of the T-stub but also its ability to withstand plastic deformations. It is worth noting that, concerning the T-stub ductility Eurocode 3 provides only an empirical relationship individuating the cases for which a satisfactory ductility supply can be expected.

In this paper reference is made to (Piluso et al., 2001) model which is, historically, one of the first models aimed at the prediction of the ductility supply of a T-stub. In authors' work, the ultimate displacement of the T-stub is determined by defining the rotations of the plastic hinges lumped in the bolt line section and in the flange-web connection starting from the knowledge of the curvatures developing along the flange plate. The theoretical model is based on two hypotheses: the kinematic mechanism is assumed a priori by evaluating the parameter β_u which represents the ratio between the ultimate resistance of the T-stub in hypothesis of mechanism type-1 and the ultimate resistance of bolts; the bending moment diagram acting on the flange plate under ultimate conditions is alike to that corresponding to the design conditions. The work done by (Piluso et al., 2001) was developed only for kinematic mechanisms type-1, type-2 and type-3, but in this paper it is extended also to mechanism type-1*, which is particularly important for base plate joints. Under the hypothesis of mechanism type-1 the T-stub failure mode is characterized by the formation of two plastic hinges with same value of the plastic rotation arising at the bolt line and at the flange-web connection (Fig.3). In this failure mode, according to (Piluso et al., 2001) the ultimate rotation of the plastic hinge can be computed by means of the following relationship:

$$\vartheta_{p1} = \frac{Cm}{2t_p} \tag{2.3}$$

where C is a coefficient corresponding to the complete development of the plastic hinge which depends only on the mechanical properties of material composing the flange plate, m is the distance between the bolt axis and the plastic hinge arising, according to EC3, in correspondence of the Tee stem and t_p is the plate thickness. In case of mechanism type-2 the kinematic failure mechanism is characterized by the development of two plastic hinges arising in correspondence of the bolt and of the flange-stem connection which are characterized by different values of the plastic rotation (Fig.3). (Piluso et al., 2001) model hypothesize that the ultimate condition is attained when the failure of the plastic hinge lumped at the plate-stem connection occurs, while the bolt is checked only for the compatibility requirements with the plate uplift. Nevertheless, it has to be noted that this collapse mode can be characterized by two different failure conditions: the collapse of the plastic hinge located at the T-stub stem, which may attain the ultimate rotation, or to the bolt failure, which can reach its ultimate uplift capacity. Therefore, in this paper, the (Piluso et al., 2001) model is generalized in order to account for both collapse mechanisms. The rotations of plastic hinges and the bolt elongation can be computed in the case of Mechanism type 2 as follows (Fig.3):

$$\begin{cases} \vartheta_{p12} = \dfrac{Cm}{(1+\xi)t_p} \\ \vartheta_{p22} = \dfrac{g'(\xi, n/m)m}{t_p} & \textit{Plate Failure} \\ \delta_{pb} = (\vartheta_{p1} - \vartheta_{p2})n < \varepsilon_{pb} L_b \end{cases}$$
$$\begin{cases} \vartheta_{p12} = \dfrac{g(\xi)m}{(1+\xi)t_p} \\ \vartheta_{p22} = \dfrac{g(\xi)m}{(1+\xi)t_p} - \dfrac{\varepsilon_{pb} L_b}{n} & \textit{Bolt Failure} \\ \delta_{pb} = \varepsilon_{pb} L_b \end{cases} \tag{2.4}$$

where ξ is a parameter expressing the ratio between the bending moment acting in the two plastic hinges, $g'(\xi, n/m)$ is a function depending only on the mechanical characteristics of the steel composing the plate and *n* is the distance between the bolt line and the free edge. Mechanisms type-1* and type-3 model the same loading situation. In fact, in these cases the plate works as a cantilever beam between the stem and the anchor bolt, but in the former case the failure is attained due to the plate failure and in the latter case is reached due to the bolt failure. In case of mechanism type-3, in order to define the T-stub ultimate displacement, starting from the bolt ultimate deformation given in Eq.2.1, the plastic hinge rotation related to the moment corresponding to bolt failure can be evaluated by means of the following relationship:

$$\vartheta_{p3} = \frac{g'(\xi)m}{t_p} \tag{2.5}$$

where, also in this case $g'(\xi)$ is a function depending only on the mechanical characteristics of the steel composing the plate. The (Piluso et al., 2001) model can be easily extended also to mechanism type-1*. In this failure mode, the bolt elongates and the ultimate rotation is attained in the plastic hinge developed at the clamped section. Therefore, it is easy to verify that, under such assumptions, the ultimate rotation of the plastic hinge is given by the following equation:

$$\vartheta_{p1^*} = \frac{Cm}{t_p} \tag{2.6}$$

and the bolt elastic elongation is provided by:

$$\delta_b = \frac{M_u L_b}{m E_b A_b} \tag{2.7}$$

where M_u is the ultimate moment arising in the plastic hinge, E_b and A_b are the elastic modulus and area of the bolt respectively.

3. PREDICTION OF THE ROTATIONAL SUPPLY OF BASE PLATE JOINTS

Within the framework of the component method, after the definition of the ductility supply of the single joint components, it is possible to assembly a mechanical model able to predict the rotational capacity of the whole base plate joint. As aforementioned, the model herein proposed assumes that the deformation capacity of the connection is governed by the failure of the base plate or of the anchor bolts in tension in the high eccentricity range. From the theoretical standpoint there are two ways to approach the model assembly in order to gain the ductility supply of the joint. The first one provides the joint ductility as the ratio between the ultimate plastic displacement of the T-stub and the lever arm. Conversely, in the second approach, the kinematic failure mechanism of the whole connection is defined and, in order to determine the rotational capacity of the joint the formulation previously exposed are exploited.

This last approach accounts for the actual kinematic mechanism of the joint that may be different from that exhibited by the single components. In fact, as shown in Fig.4 there is a substantial difference between the real joint behavior and that schematized in the case of the T-stub. In an exposed base plate joint, as far as the joint rotates, a part of the ductility supply of the plastic hinge close to column tension flange is spent to allow the rigid rotation of the column web. This behavior is confirmed by the experimental evidence. In fact, the observation of damages occurring in partial strength double split T-stub joints (Latour & Rizzano, 2012; Latour, 2011) shows that when the T-stub is the weakest component, even though the Tee is symmetrical, the failure condition is reached due to the formation of a crack in the flange plate zone external to the beam flanges. Taking into account the above considerations, in the following, the relationships for the rotational capacity of the base plate joint for the four possible collapse mechanism are determined and in Table 3.1 have been summarized.

3.1. Mechanism type-1

The ductility model here presented assumes that the kinematic mechanisms occurring in the base plate are that reported in Fig.4. In addition, the bending moment diagram acting on the flange plate is hypothesized to be equal to that defined in the case of T-stub subjected only to tension loads, i.e. the influence of the rigid rotation on the bending moment diagram is neglected. It has to be noted that, in the actual plate behavior, the rigid rotation due to the column web slightly changes the values of the plastic rotations defined in previous paragraph. This variation is mainly due to the shift of the point of contraflexure of the bending moment diagram.

In case of mechanism type-1 the failure condition is characterized by the formation of two plastic hinges in the plate. The collapse is therefore assumed to occur when the hinge close to the column tension flange attain the ultimate plastic rotation defined by Eq.2.3. Under this assumption it is easy to verify, by means of geometrical considerations, that the hypothesized mechanism leads to following value of the base joint rotational capacity:

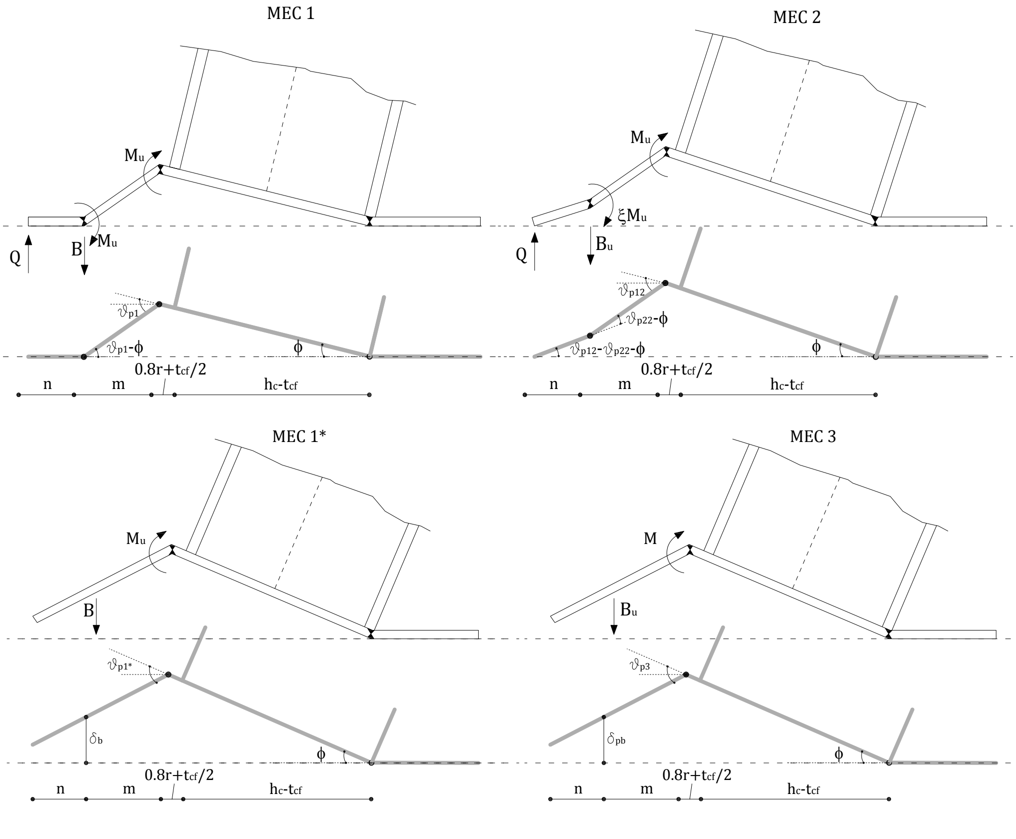

Figure 4. Assumed Kinematic collapse mechanism for base plate joints

$$\phi_{p1} = \frac{\vartheta_{p1} m}{\left(h_c + 0.8r - \frac{t_{cf}}{2} + m\right)} \tag{3.1}$$

where h_c is the column height, t_{cf} is the thickness of the column flange and r is the distance between the flange and the plastic hinge as defined in Eurocode 3.

3.2. Mechanism type-2

In case of mechanism type-2, the same phenomenon described with reference to mechanism type-1 occurs. In fact, also in this collapse mechanism, the joint rotation increases the ductility demand on the plastic hinge located at the plate-column connection. By assuming that the plastic rotations of the hinges are approximately equal to those determined in the equivalent T-stub model, the rotational supply of the base plate joint can be determined as follows:

$$\phi_{p2} = \frac{\vartheta_{p12}(m+n) - \vartheta_{p22} n}{\left(h_c + 0.8r - \frac{t_{cf}}{2} + m + n\right)} \tag{3.2}$$

3.3. Mechanism type-1*

As aforesaid, in this case contact forces do not develop due to the high deformability of the anchors and the collapse condition is reached due to the attainment of the ultimate plastic rotation of the hinge located at the plate clamped section. By evaluating the hinge ultimate rotation and the bolt elongation at collapse by means of Eq.2.6, it is possible to evaluate the plastic rotational capacity of the base connection bye means of the following relationship

$$\phi_{p1*} = \frac{\vartheta_{p1*} m + \frac{M_u L_b}{m E_b A_b}}{\left(h_c + 0.8r - \frac{t_{cf}}{2} + m\right)} \tag{3.3}$$

3.4. Mechanism type-3

Mechanisms type-1* and type-3 model the same loading situation in which the base plate works as a cantilever beam between the clamp and the anchor bolt. When the failure mode is of type-3, the collapse of the connection is reached due to the attainment of the ultimate bolt uplift elongation. In this case the plastic joint rotation is given by two contributes: the bolt plastic deformation and the rotation of the plastic hinge when bolt failure occurs. The plastic rotation of the whole joint can be expressed in this case as:

$$\phi_{p3} = \frac{\vartheta_{p3} m + \varepsilon_{pb} L_b}{\left(h_c + 0.8r - \frac{t_{cf}}{2} + m\right)} \tag{3.4}$$

Table 3.1. Individuation of the failure mechanism

Prying Forces may develop - $L_b \leq \frac{8{,}8 m^3 A_b}{\sum l_{eff,1} t_p^3}$		**No Prying Forces** - $L_b > \frac{8{,}8 m^3 A_b}{\sum l_{eff,1} t_p^3}$	
$Mec.1 - \beta_u = \frac{4M_u}{2B_u m} \leq \frac{2\left(\frac{n}{m}\right)}{\left[1+\left(\frac{2n}{m}\right)\right]}$	*Eq.3.1*	$Mec.1^* - \beta_u \leq 2$	*Eq.3.3*
$Mec.2 - \frac{2\left(\frac{n}{m}\right)}{\left[1+\left(\frac{2n}{m}\right)\right]} < \beta_u \leq 2$	*Eq.3.2*		
$Mec.3 - \beta_u > 2$	*Eq.3.4*	$Mec.3 - \beta_u > 2$	*Eq.3.4*

4. COMPARISON WITH EXPERIMENTAL RESULTS

In order to evaluate the accuracy of the theoretical model in predicting the ductility supply of base plate joints a comparison with experimental results has been carried out. In this paper a preliminary validation of the model is carried out. In particular reference is made to three authors' own tests carried out at the laboratory on materials and structures of Salerno University (Latour et al., 2012). The tests regard three exposed base plate joint loaded under monotonic condition. Specimens are different for geometry of the plate and of the column. In particular, the specimens are composed by HE240B and HE160A column profiles connected to a 1400x600x600 mm concrete base by means of plates with thickness equal to 15 and 25 mm. All the steel elements, columns and plates, are made of S275 steel grade while the concrete base is made of C20/25 class. The connections between the base plate and the concrete footing have been made by means of M20 threaded bars of 8.8 class. Stress-strain behavior of the plate has been determined by means of coupon tensile tests. Specimens, that are called HE 240B-15, HE240B-25 have been designed so that the weakest joint component is the base plate, while the test HE160A-15-233 has been designed by balancing the resistance of the component on the tensile side and that on the compressed side. Regarding the failure mechanisms, the experimental tests evidenced a good accuracy with Eurocode 3 model. In fact, in all the tests, according to the prediction, collapse has occurred due to the fracture of the base plate at the heat affected zone. The experimental values of the plastic rotations have been computed starting from the experimental moment-rotation curve relating the bending moment to the joint rotation according to Fig.5. The design flexural resistance of the connection is computed, according to Eurocode 3, as the bending moment corresponding to a secant stiffness equal to $K/3$, where K is the initial rotational stiffness. In addition, the bending moment corresponding to the elastic limit is defined as $2/3M$ and the ultimate rotation is defined in correspondence of a strength deterioration of the 20%. Therefore the plastic rotation of the joint is computed as the difference between the ultimate rotation and the first yielding rotation. In Table 3.2 the predicted values of the ultimate plastic rotation of joints are compared with the experimental ones. Furthermore, the ratio between the predicted and the experimental values of the plastic rotation supply are evaluated. The obtained average value is equal to 0.92, while the standard deviation is equal to 0.04. As it is possible to see the obtained results are very accurate and on the safe side in all considered cases.

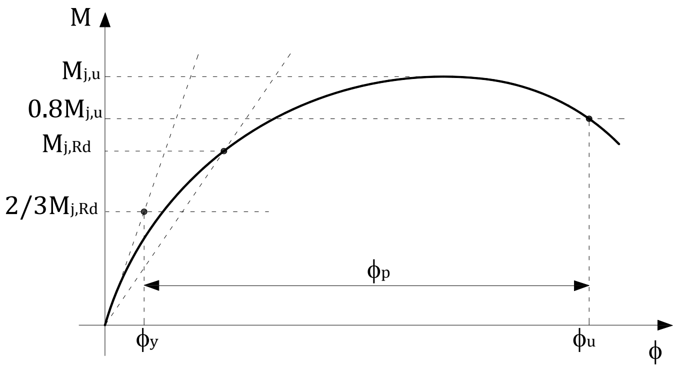

Figure 5. Definition of the experimental plastic rotation

Table 3.2. Individuation of the failure mechanism

Test	Mec	θ_{p1} [rad]	θ_{p2} [rad]	δ_{pb} [mm]	$\phi_{p,mod}$ [mrad]	$\phi_{p,exp}$ [mrad]	$\phi_{p,mod}/\phi_{p,exp}$
HE240B-15	1	0.5519	0.5519	0	115	131.6	0.874
HE160A-15	2	0.466	0	23.3	173.5	183.7	0.944
HE240B-25	2	0.466	0	23.3	168.9	178.5	0.946
						Average	**0.92**
						St.Dev.	**0.04**

5. CONCLUSIONS

In this paper a theoretical model for predicting the rotational capacity of base plate joints has been presented. The proposed approach is based on the component method codified in Eurocode 3. Therefore, in this work, in order to gain the ductility of the whole connection, the ductility supply of the joint components has been characterized, both adopting literature models and proposing improvements to account for the particular failure mechanisms occurring in base plate joints. Then, a procedure to assembly the mechanical model representing the base plate connection has been proposed. Such a procedure, starting from the evaluation of the possible kinematic collapse mechanism occurring in the base plate connection, defines the equations to obtain the joint rotational capacity for all considered failure modes.

The comparison with experimental tests carried out by the same authors at laboratory on materials and structures of Salerno University has shown a good accuracy of the model. Although the results presented in this paper are only preliminary, they are encouraging about the possibility of predicting the plastic rotation supply of exposed base plate joints by means of a theoretical approach based on the component method.

REFERENCES

Beg, D., Zupancic, E. & Vayas, I., 2004. On the Rotation Capacity of Moment Connections. Journal of Constructional Steel Research, 60, pp.601-20.

Clemente, I., Noè, S. & Rassati, G., 2005. Experimental and Numerical Analysis of the Cyclic Behaviour of T-stub Components. In Proceedings of XX CTA Conference. Ischia, 2005.

Faella, C., Piluso, V. & Rizzano, G., 1998a. Cyclic behavior of bolted joint components. Journal of Constructional Steel Research, 46(1-3), p.paper number 129.

Faella, C., Piluso, V. & Rizzano, G., 2000. Structural Steel Semi-Rigid Connections. Boca Raton: CRC Press.

Girao Coelho, A.M., Bijlaard, F. & Simoes Da Silva, L., 2004. Experimental Assessment of the Ductility of Extended End Plate Connections. Engineering Structures, 26, pp.1185-206.

Gomez, I., Kanvinde, A. & Deierlein, G., 2010. Exposed Column Base Connections Subjected to Axial Compression andd Flexure. Final Report. AISC.

Iannone, F., Latour, M., Piluso, V. & Rizzano, G., 2011. Experimental Analysis of Bolted Steel Beam-to-Column Connections: Component Identification. Journal of Earthquake Engineering, 15, pp.214-244.

Jaspart, J.P., 2002. Design of Structural Joints in Building Frames. Progress in Structural Engineering and Materials, Vol.4(18-34).

Jaspart, J. & Vandegans, D., 1998. Application of the Component Method to Column Bases. Journal of Constructional Steel Research, 48, pp.89-106.

Kuhlmann, U. & Kuhnemund, F., 2000. Rotation Capacity of Steel Joints: Verification Procedure and Component Tests. NATo advanced research workshop. Dordrecht, 2000. Kluwer Academic Publishers.

Latour, M., 2011. Theoretical and Experimental Analysis of Dissipative Beam-toColumn Joints in Moment Resisting Steel Frames. PhD Thesis ed. Boca Raton, Florida: Universal Publishers.

Latour, M., Piluso, V. & Rizzano , G., 2011. Cyclic Modeling of Bolted Beam-to-Column Connections: Component Approach. Journal of Earthquake Engineering, 15(4), pp.537-63.

Latour, M., Piluso, V. & Rizzano, G., 2012. Column-Base Plate Joints under Monotonic Loads: Theoretical and Experimental Analysis. 7th International Workshop on Connections in Steel Structures, Timisoara, 2012.

Latour, M. & Rizzano, G., 2012. Experimental Behavior and Mechanical Modeling of Dissipative T-Stub Connections. Journal of Structural Engineering, 138(2), pp.170-82.

Leon, R. & Swanson, A., 2000. Cycli Modelling of T-stub Connections. In Proceedings of Steel Structures in Seismic Areas. Canada, 2000.

Melchers, R., 1992. Column-Base Response Under Applied Moment. Journal of Constructional Steel Research, 23, pp.127-43.

Piluso, V., Faella , C. & Rizzano, G., 2001. Ultimate behavior of bolted T-stubs. Part I: Theoretical model. Journal of Structural Engineering ASCE, 127(6), pp.686-93.

Piluso, V. & Rizzano, G., 2008. Experimental Analysis and modelling of bolted T-stubs under cyclic loads. Journal of Constructional Steel Research, 64, pp.655-69.

Simoes Da Silva, L., 2001. A Ductility Model for Steel Connections. Journal of Constructional Steel Research, 45, pp.45-70.

Sokol, Z. & Wald, F., 1997. Experiments with T-stubs in Tension and Compression. Report PECO-AH-132. Praha: CTU.

Steenhuis, C. & Bijlaard, F., 1999. Tests on Column Bases in Compression. In T.a.M.B.T. Institute for Steel, ed. Commemorative Publication for Prof. Dr. F. Tschemmernegg. Innsbruck: G.Huber.

Thambiratnam, D. & Paramasivam, P., 1986. Base Plates under Axial Loads and Moments. Journal of Structural Engineering, 112(5), pp.1166-81.