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Parametric finite element analyses on flush end-plate joints under column removal

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ABSTRACT

Flush end-plate beam-to-column joints are widely used in gravity load resisting parts of steel frames. Although generally designed to behave as pinned connections, this type of joint is generally semi-rigid and partial strength, thus providing a reserve of strength against progressive collapse. In order to enhance the joint performance under column loss scenario, a design criterion is proposed and its effectiveness is investigated and discussed by means of parametric finite element analyses. The investigated parameters are the bolt diameter, the end-plate thickness, the number of bolt rows, the type of beam profile and the orientation of column axis. The obtained results enable to characterize the moment-rotation response curve and ultimate rotation capacity under compressive arching and catenary action.

1. Introduction

The progressive collapse of structures has been systematically addressed since the partial collapse of the Ronan Point tower [1]. In recent years, terrorist attacks (e.g. the Alfred P. Murrah Federal Building, the World Trade Centre towers, etc.) triggered renewed interest on the study of progressive collapse and structural robustness.

Current design guidelines [2–4] require that secondary structural elements, like those constituting the gravity load resisting parts in a multi-storey building, should be modelled with zero stiffness and resistance and verified for displacement acceptance criteria. However, the contribution of these elements and their joints can be favourable for arresting progressive collapse [5–9]. The role of joints is even more important and it deserves detailed investigation, especially considering that current codes provide limited design rules and requirements for ductile performance under catenary actions.

In Europe, flush end-plate (FEP) bolted beam-to-column joints are widely used in steel frames, especially for gravity load resisting parts and secondary members, due to their simplicity and low constructional costs. FEP joints are generally conceived as pinned, thus disregarding their contribution in terms of strength and stiffness to the building capacity against the design loads. However, the flexural capacity of this type of joints is far from being negligible and it can be beneficial in case of severe loading conditions as those imposed by column loss, providing redundancy that is favourable for arresting progressive collapse. The flexural performance of FEP joints has been widely studied both numerically and experimentally by a number of Authors [10–19]. Nevertheless the behaviour under catenary action under column loss has not been thoroughly analysed and further research is still required to characterize both joint strength and rotation capacity.

These considerations motivated the present study, which is aimed at investigating the rotation capacity of FEP joints and its flexural interaction with catenary forces developing under column loss in the framework of EN 1993-1-8 [20].

To this end a parametric study based on finite element analysis (FEA) is carried out and the following variables are examined: the bolt diameter, the end-plate thickness, the number of bolt rows and the type of beam cross section (i.e. IPE and HE) and the column axis orientation. The discussion on the influence of the different variables enables to identify the details that maximize the joint capacity under column loss.

The paper is organized into three main parts. In the first part, the basic features of the behaviour of connections under column loss scenario are discussed and a design criterion is proposed to enhance the performance of FEP joints under column removal scenario. In the second part, the selection of parametric variables, the modelling assumptions and the relevant validation are presented. In the third part, the main outcomes obtained from the parametric FEAs are described and discussed, showing the influence of each variable on both the response and the design of joints, as well.

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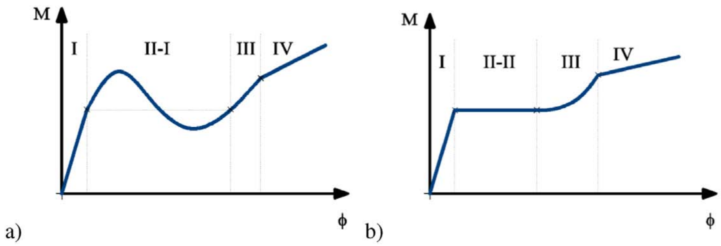

c)

STAGE	DESIGNATION
I	Elastic
II-I	Compressive arching
II-II	Plastic bending
III	Catenary transient
IV	Catenary final

Fig. 1. Typical joint moment-rotation response under column loss [26]: a) initial compressive arching effect followed by tensile catenary action; b) solely tensile catenary action; c) description of each performance stage.

2. Behaviour of bolted beam-to-column joints under column removal

2.1. Literature review

In recent years several experimental and numerical studies were carried out to investigate structural issues related to the progressive collapse under column loss scenario, such as the behaviour of steel beam-to-column joints [21–24], the overall structural response [25–29,56], the flexural behaviour of axially restrained beams [30] and full scale experimental tests involving column removals [8,31,32,57]. These studies highlighted the importance of the behaviour of joints on system behaviour under extreme actions.

Bolted beam-to-column joints can experience two types of inelastic response under column loss [26], namely: (i) the post-elastic response is initially affected by compressive arching-like mechanism with subsequent catenary effects (see Fig. 1a); (ii) the post-elastic behaviour is solely influenced by catenary-like mechanism (see Fig. 1b).

In case of FEP joints, both experimental and numerical studies have been carried out in order to investigate their response under monotonic and cyclic bending [10,11], showing that FEP joints can experience large rotation capacity, provided that both the failure of welds and mode 3 are prevented. The possibility of extending the components method for combined bending moment and axial force was experimentally investigated by [12] and results indicated that axial force significantly influences the mechanical behaviour. An experimental investigation on high strength steel end-plate moment connections [13] showed that EN 1993-1-8 [20] accurately predicts the resistance, but overestimates the stiffness experimentally obtained. The tests on the seismic behaviour of FEP joints carried out by [14] showed that the failure modes can be correctly predicted by the EN 1993-1-8 [20].

The response of FEP joints under catenary actions has also been evaluated through some studies [9,12,28], although limited data is currently available on this topic. Experimental tests carried out by [9] on different types of bolted beam-column joints under column removal scenarios, clearly showed that flush end-plate connections can provide ductile deformation and are capable of developing catenary action. The behaviour of FEP joints under normal and elevated temperatures has been also recently investigated both experimentally and numerically [15,33]. In particular, Wang et al. [33] showed that catenary actions developing into the beams are limited by the deformation capacity of the FEP joints to sustain very large deflections. Guo et al. [34] carried out a numerical study on composite frame with FEP joints under column loss action, showing that resistance to progressive collapse depends on mechanical properties of bolts. In particular, increasing bolt diameter and/or bolt fracture strain can improve the resistance against progressive collapse. Yang and Tan [21] carried out a series of numerical simulations on different types of steel beam-to-column joints, including FEP joints, subjected to catenary action induced by column loss and they concluded that connections designed with bolt rows closer to the centroid of the beam cross section provide enhanced resistance and ductility at the large rotations, contrarily to what is generally expected under pure bending where the bolt rows are more effective if largely spaced out and located close to the beam flanges.

This concise review of existing literature about FEP joints highlights that design criteria and recommendations for ductile detailling under column loss need further investigation.

2.2. Proposed design criterion

In order to enhance the performance (i.e. both the strength and ductility) of FEP joints under column loss, both end-plate thickness (i.e. end-plate flexural strength) and bolt diameter (i.e. bolt strength) should be selected in a given range so as to mobilize their corresponding strength, but avoiding mode 3 that typically corresponds to reduced ductility of T-Stub connections [10,14]. On the basis of this consideration, the thickness of end-plate can be rationally selected between the minimum and maximum values inducing mode 2 for a given bolt diameter (which is generally designed a-priori to resist the shear forces due to gravity loads).

EN 1993-1-8 [20] recommends a ductility criterion for bolted joints that relates the thickness of the end-plate to the bolt diameter as follows:

$$t_{EN1993:1-8} \le 0.36 \bullet d \bullet \sqrt{\frac{f_{ub}}{f_y}} \tag{1}$$

where d is the bolt diameter, f_{ub} is the ultimate stress of the bolt material and f_y the yield strength of the material of the connected plate.

The thickness given by Eq. (1) is a threshold between mode 1 and mode 2 depending on the yield line pattern (i.e. circular or non-circular). Indeed, the criterion of Eq. (1) imposes that the resistance of each individual bolt ($F_{t,Rd}$) is greater than the resistance ($F_{p,Rd}$) of the connected plates (end-plate or column flange), having assumed γ_{M0} and γ_{M2} respectively equal to 1.0 and 1.25.

However, considering the random variability of material strength and the strain hardening that can be developed by the connected plates in plastic range, the ultimate strength of joints should be properly evaluated as follows:

$$F_{t,Rd} \ge \gamma \cdot F_{p,Rd} = \gamma_{ov} \cdot \gamma_{sh} \cdot F_{p,Rd} \tag{2}$$

The random material overstrength factor γ_{ov} in Eq. (2) depends on the steel grade. In this study it is taken equal to 1.25, as recommended by Eurocode 8. The strain hardening factor γ_{sh} is assumed as the ratio between the ultimate stress f_u and the yield stress f_y of the plate material. For European mild carbon steel, the ratio f_u/f_y can be conservatively assumed equal to 1.5. Thus rearranging the inequality in Eq. (2) and introducing the EN1993:1-8 [20] design equations for the strength of the yield line mechanism into the end-plate and the bolt strength, the minimum thickness to activate mode 2 can be obtained as follows:

$$t_{min,Mode2} \ge \frac{0.40 \bullet d}{\sqrt{\gamma_{ov} \bullet \gamma_{sh}}} \bullet \sqrt{\frac{\gamma_{M0} \bullet f_{ub}}{\gamma_{M2} \bullet f_y}} = 0.26 \bullet d \bullet \sqrt{\frac{f_{ub}}{f_y}} \quad (\le t_{min,EN1993:1-8}) \tag{3}$$

The upper bound value of thickness can be determined in order to avoid mode 3, namely imposing the following inequality: