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Review

Plastic design of eccentrically braced frames, I: Moment–shear interaction

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ABSTRACT

With reference to eccentrically braced frames (EBFs), the issues regarding moment–shear interaction in plastic design are examined. In particular, a procedure for computing the ultimate shear forces and end moments of intermediate links for a given collapse mechanism is presented. The requirements to be fulfilled are the yielding condition, the flow rule, the kinematic compatibility condition depending on the examined collapse mechanism and, finally, the minimisation of the internal work according to the kinematic theorem of plastic collapse.

Moreover, through the definition of an *equivalent moment*, the link is modelled as an element with plastic hinges in simple bending. This is obtained by imposing the equivalence between the internal work developed by the actual link and the internal work corresponding to the simplified theoretical model. Therefore, moment–shear interaction in plastic design can be accounted for by means of formulations which are formally coincident with those commonly used for plastic hinges in simple bending.

Finally, the kinematic theorem of plastic collapse is used to define the design requirements to be fulfilled to prevent yielding of the beam part outside of the link and yielding of the bracing element. The additional design rules to be satisfied for failure mode control of multi-storey EBFs are presented in a companion paper.

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Contents

1. Introduction

Shear yielding of members is an effective mechanism of energy dissipation for earthquake resistant structures. Short members yielding in shear were proposed first as shear links in eccentrically braced frames (EBFs) [1–3] leading to a wide spreading of this structural typology as an effective mean to withstand severe earthquakes. The idea of exploiting shear yielding members as energy dissipation devices has promoted the development of new structural systems such as knee braced frames [4], aluminium shear links [5] and shear links made of low yield point steel [6,7].

Modelling efforts for member yielding under moment–shear interaction are usually based on simple one-component or two-component models.

The two-component model was originally devoted to the cyclic inelastic flexural response of beams [8] whose modelling is obtained by means of two parallel beams, one of them having elastic-perfectly plastic behaviour and the other one having elastic behaviour with a reduced stiffness depending on strain-hardening to be obtained. In fact, the combination of such two components gives rise to a bilinear response with strain-hardening occurring after yielding of the first component. This modelling has been successively improved [9] to account also for moment-axial load interaction effects.

The lumped plasticity approach based on one-component models requires rigid elastic–plastic hinges, located at the end of an elastic beam element, providing flexibility and plastic deformation after yielding [8–10].

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All the above models do not account for shear effects. A first attempt to account for shear effects was based on the sandwich beam model [11] where the web withstands shear forces and the flanges resist to bending moments. In addition, the two-component model was used with bilinear kinematic flexural hardening and combined isotropic and kinematic shear hardening.

A different approach, namely the inclined truss model, has been also proposed to model an active link by assuming equal end moments [12]. The same assumption has been adopted within the framework of an inelastic two-component flexural element by specifying equivalent flexural capacities, so that the corresponding maximum shear force matches the link shear capacity.

A more sophisticated one-component model consists of a linear elastic beam element with non-linear hinges at each end whose behaviour results from a series of three sub-hinges where plastic deformations are lumped [13]. The formulation is plasticity based and includes shear and flexural yielding. In addition, combined isotropic and kinematic shear strain-hardening effects are accounted for, as well as flexural strain-hardening. However, the function describing the isotropic shear expansion of the yield surface is calibrated on the base of experimental data.

The need for parameter tuning for different loading and support conditions makes the approach empirical in nature, so that it is subjected to the well-known calibration limitation of the one-component model requiring the assumption of the location of the point of inflection, usually assumed at midspan.

In order to overcome the above drawback, a new beam element for the simulation of shear yielding members has been recently proposed [14]. The element is based on a three-field variational formulation with independent force, displacement and deformation fields [15]. The displacement field is based on Timoshenko's shear beam theory. The integration of biaxial stress–strain relations over several control sections along the element length allows one to account for non-linear response including axial force-shear-flexure interaction.

The above brief presentation of available studies dealing with shear yielding members shows that their primary concern is the analysis of structures, i.e. a finite element to be adopted for push-over analyses and/or dynamic non-linear analyses.

Conversely, the primary aim of the modelling presented in this paper is the setting up of a simple design tool for structures, such as eccentrically braced frames, where shear yielding members are adopted as dissipative zones. Reference is made to the initial design phase requiring that member sections have to be properly selected according to capacity design principles stating that dissipative zones, i.e. the links, have to be designed considering the internal actions occurring under the code provided load combinations, while non dissipative zones have to be designed considering the maximum internal actions that yielded and strain-hardened dissipative zones are able to transmit.

The proposed design approach is based on plastic design, i.e. rigid-plastic analysis, as already proposed for moment-resisting frames [16]. Therefore, this paper deals with the problem of moment–shear interaction within the framework of rigid-plastic analysis where the attention is focused on the collapse mechanism independently of the deformations occurring before the complete development of a kinematic mechanism. In addition, simple rules for failure mode control are provided with reference to one-storey frames, while the case of multi-storey frames, aiming at the setting up of a design procedure able to assure a collapse mechanism of global type, is dealt with in a companion paper [17]. It means that the aim of the developed research activity is to provide designers with a procedure assuring the formation of a collapse mechanism characterised by the yielding of all the link elements, along the building height, and by the yielding of the base sections of first storey columns.

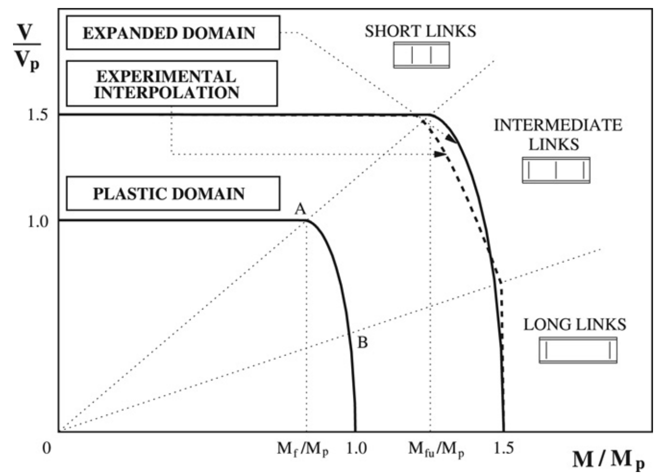

Fig. 1. Plastic V–M interaction diagram and comparison between ultimate interaction domains.

2. Plastic and ultimate domains under the moment–shear interaction

Eccentrically braced frames constitute, perhaps, the most important structural typology for which the interaction between bending moment and shear force is not negligible for predicting the ultimate behaviour of the structural system. Usually, EBFs are classified depending of the length e of the link element. In particular, by progressively increasing the link length, the plastic behaviour of the link is dominated by pure shear (*short links*), by the interaction between shear force and bending moment (*intermediate links*) and, finally, by simple bending (*long links*). In technical literature the following classification is commonly accepted [3]: *short link* for $e \leq 1.6M_p/V_p$, *intermediate links* for $1.6M_p/V_p \leq e \leq 3M_p/V_p$, *long links* for $e \geq 3M_p/V_p$ being V_p and M_p the plastic shear resistance and the plastic moment resistance of the link cross section, respectively. Therefore, this paper is mainly devoted to intermediate links.

The link plastic behaviour is described by the interaction diagram reported in Fig. 1. The behaviour of the member can be assumed to be in pure shear, i.e. the effect of the bending moment can be neglected, provided that it is less than the moment M_f representing the contribution of the flanges to the plastic moment of the section. Conversely, for a bending moment exceeding M_f, the shear resistance of the member is reduced and the interaction between bending moment and shear force has to be accounted for.

The analytical expression of the plastic domain is given by [18]:

$$\left(\frac{|M| - M_f}{M_p - M_f}\right)^2 + \left(\frac{V}{V_p}\right)^2 = 1 \quad \text{for } M_f \leq |M| \leq M_p \tag{1}$$

$$V = V_p \quad \text{for } |M| \leq M_f. \tag{2}$$

An important design issue, reported since earlier tests by Hjelmstad and Popov [19] and Malley and Popov [20], is the overstrength developed by the link. Link overstrength is the ratio between the maximum shear force developed by the link and the plastic shear strength of the link. The interpretation of testing results is usually based on the evaluation of the plastic shear resistance as:

$$V_p = 0.6 f_y (d - 2t_f) t_w \tag{3}$$

where f_y is the yield stress, d the link section depth, t_f and t_w are the thicknesses of flange and web, respectively, according to the American practice.

Link overstrength is primarily due to strain-hardening, but can also be due to the contribution of link flanges to the

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