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Ultimate behavior of steel beams under non-uniform bending

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ABSTRACT

The ultimate behavior of steel beams deeply influences the overall performance of steel frames. The main response parameters are the rotation capacity and the flexural ultimate resistance. The former is the source of the local ductility supply needed to achieve a global dissipative behavior of structures under seismic actions, whereas the latter governs the flexural overstrength whose knowledge is needed for an appropriate application of hierarchy criteria in seismic design of structures. Therefore, a twofold classification of steel members according to their ductility and overstrength is the most appropriate approach for seismic design applications. Currently, modern international design codes are based on the classification of steel sections for both plastic and seismic designs of structures, providing misleading emphasis mainly on local buckling as the primary strain-weakening effect. Even though different methods are available in the technical literature for predicting the ultimate behavior of steel members under non-uniform bending, the problem still deserves further investigations, because of the high number of parameters affecting the ultimate response and the variety of cross-sectional shapes. Therefore, a new experimental program dealing with a wide range of cross section typologies (I and H sections, Square and Rectangular Hollow sections) under monotonic and cyclic loading has been carried out by testing specimens with different local slenderness ratios properly selected to integrate the data already available in the technical literature. The obtained results are herein presented and discussed.

1. Introduction

The development of adequate plastic rotation capacity is a fundamental requirement both for plastic design and for seismic design of building structures. Indeed, plastic design can be carried out provided that members are able to develop plastic hinges rotating until the collapse mechanism is completely developed, without dropping their moment capacity, thus assuring the required redistribution of bending moments. The rotation of plastic hinges required to fully develop the collapse mechanism and/or to achieve needed displacement levels provides the plastic rotation demand, which varies for different structural configurations, loading arrangements, geometry, material strengths and levels of the seismic intensity measure. It has been widely investigated in the literature, particularly for continuous beams and frames [1–4].

In particular, in earthquake-resistant design, rotation capacity is essential to assure that a determined portion of the input seismic energy is dissipated by plastic behavior. Therefore, steel beams need to develop a ductile behavior with high rotation capacity. To this scope appropriate geometrical limitations to the geometry of the plate elements constituting the cross-section and to the laterally unrestrained length need to be considered, because the flexural behavior of steel beams can be undermined by the occurrence of plastic local buckling of compressed elements and/or by inelastic flexural–torsional buckling. Therefore, in order to perform a reliable structural analysis, it is essential to quantify clearly the meaning of "sufficient rotation capacity" or "sufficient local ductility".

Nowadays, Eurocode 3 [5] provides the subdivisions of cross-sections into four classes, depending on the properties of compression elements (Fig. 1). For plastic global analysis, it is required that all members containing plastic hinges shall belong to class 1, i.e. to be made of ductile sections. According to Eurocode 8 [6], the cross section classes defined in Eurocode 3 provide a limitation to the selected behavior factor requiring class 1 for $q>4$, class 2 for $2<q\leq4$, and class 3 for $q\leq2$ (as shown in Table 1). However, the main criticism to Eurocode classification is the small number of parameters considered to characterize the beam performance. In fact, they relate rotation capacity to material and cross-section factors only, neglecting very important behavioral issues, such as the flange–web interaction, the overall member slenderness, the moment gradient, the lateral restraints, and the loading conditions (monotonic or cyclic).

These considerations led several authors to develop other classification criteria. In the recent past, classification criteria accounting for both cross-section slenderness and member slenderness were early proposed by [7–10] for I and H shaped members.

Furthermore, regarding seismic applications it is important to note that it is not possible to directly extend the criteria developed for

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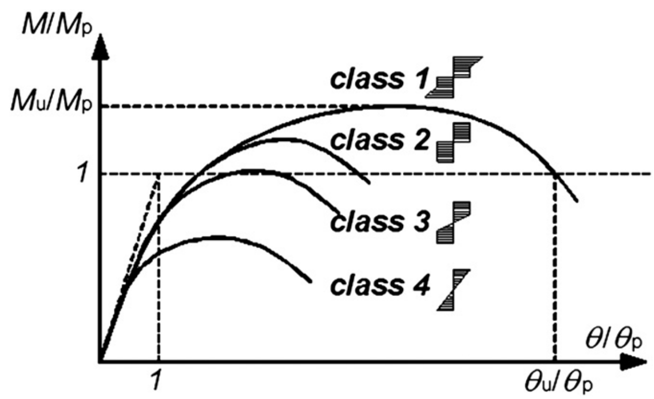

Fig. 1. EN 1993:1-1 classification criterion.

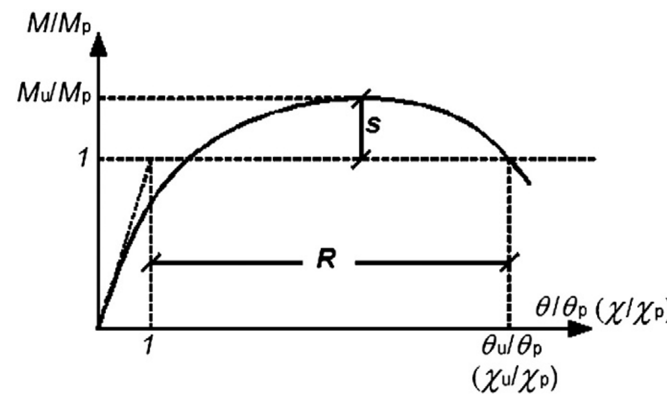

Fig. 3. Generalized moment–rotation curve for a steel beam.

monotonic loading to cyclic loading conditions. Therefore, under this point of view it is useful to underline that Eurocode 3 classification is basically derived from experimental test results under monotonic loading conditions, highlighting the weakness of Eurocode 8 classification when reference is simply made to Eurocode 3. In fact, it is worth noting that the comparison between the width-to-thickness limits given by Eurocode 3 and those given by AISC [11] reveals considerable differences, particularly in case of rectangular and circular tubular members, being the limits of seismically-compact sections in AISC [11] significantly more severe than those corresponding to class 1 of Eurocodes. Therefore, the assessment of the adequacy of class 1 sections to satisfy the cyclic demands imposed under seismic conditions is needed.

Another design issue neglected by Eurocode 8 classification is the flexural overstrength of steel beams, which plays a key role in the application of hierarchy criteria in seismic design. Indeed, Eurocode 8 accounts only for the possible overstrength due to random material variability by considering an overstrength factor equal to 1.1 γ_{ov}, for all types of members, in the application of capacity design rules. Conversely, the amount of strain-hardening which can be exhibited before the complete development of local buckling is neglected, thus underestimating the actual ultimate strength of members made of class 1 sections.

Starting from these considerations, it is clear that, dealing with seismic applications, a twofold classification of steel beams in ductility and overstrength classes is needed. Indeed, the subdivision of beams into ductility classes remains of primary concern, because rotation capacity directly governs the global ductility of structures, but, at the same time, a subdivision of members into overstrength classes is also desirable in order to properly apply capacity design criteria both at local level (i.e. to design connections among dissipative and not dissipative zones) and at global level (i.e. to provide adequate overstrength to non-dissipative members). This approach can be achieved implementing the concept of member behavioral classes [12], which has been early adopted by the late Italian code for seismic design OPCM 3274 [13] (as shown in Fig. 2 and in Table 3).

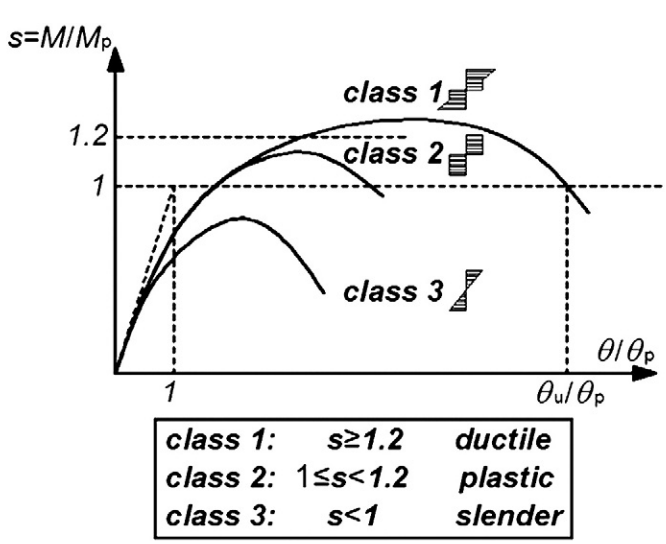

Fig. 2. OPCM 3274 classification criterion.

Therefore, aiming at the setting up of reliable design formulas for predicting both the rotation capacity and the flexural overstrength of steel beams a new experimental program has been carried out and its outcome is herein illustrated pointing out the influence of geometrical and mechanical parameters.

2. Rotation capacity

2.1. Generality

The moment–curvature approach has been widely used in literature to study cold-formed steel sections [14–17]. According to this approach, the cross-section local ductility is defined in terms of curvature ductility as follows:

$$\mu_\chi = \frac{\chi_u}{\chi_p} - 1 \tag{1}$$

where χ_p is evaluated as M_p/EI and χ_u is the ultimate limiting curvature at which the moment resistance drops below M_p.

Conversely, rotation capacity (R) is the measure of the local ductility at the member level and, therefore, it is a rotation ductility evaluated by means of the moment–rotation relationship [18–20] as follows:

$$R = \mu_\theta = \frac{\theta_u}{\theta_p} - 1 \tag{2}$$

being θ_p the rotation corresponding to flexural yielding and θ_u the ultimate beam rotation. The rotation capacity can be divided into two parts: the stable part of rotation capacity developed up to the occurrence of local buckling and the unstable part due to post-buckling behavior [21].

Table 1
EC8 cross-section requirements for dissipative elements depending on ductility class and behavior factor.

Ductility class	Behavior factor	Cross section class (dissipative zone)
DCM	$1.5<q<2$	Class 1, 2 and 3
	$2<q<4$	Class 1 and 2
DCH	$q>4$	Class 1