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New proposal for classification of steel flexural members based on member ductility

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Abstract
Classification of cross-section is an important concept in the design of flexural steel members as it addresses the susceptibility of a cross-section to local buckling and defines its appropriate design resistance. In fact the section ductility concept is employed in the most of current steel design codes where section behavior is governed by the buckling of flange or web plates, for which independent limitations are imposed. This assumption is unreasonable because, obviously, the flange is restrained by the web and the web is restrained by the flanges, so the interaction between the two local buckling modes must be considered. Furthermore at the member level, there are some additional factors that influence the ductility (e.g. the beam span, flange to web width ratio, member slenderness, moment gradient, etc). As a consequence of these factors, it seems that the section behavioral classes should be substituted by the concept of member behavioral classes. The present study employs experimentally verified nonlinear finite element modeling techniques for the study of classification based on member ductility. Attention is given to the interaction between local and lateral buckling of I-sections and their influence on inelastic flexural ductility for members subjected to a constant moment. To determine the member rotation capacity for uniform moment loading, a new equivalent method was proposed. An extensive parametric study was carried out on welded beams with I-cross sections. The results illustrate that there are great differences between the two classification criteria presented and that the member classification is more appropriate for checking the structural ductility than the cross-sectional classes. On this basis a new proposal for a possible classification is made based on member ductility with taking into account the interaction between local and local-overall buckling modes.

1. Introduction
Current seismic design procedures implicitly permit the inelastic structural deformations under strong ground motions for economical reasons. The ductility is used as a parameter for evaluating the available inelastic performances of structures. The factors that affect the ductility

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of these members are the section dimensions, support conditions, type of loading, manufacturing distortions (imperfections and residual stresses) and material properties.

Before the 1960s, the ductility notion has been used only for characterizing the material behavior, after Baker's studies in plastic design and Housner's research works in earthquake problems. This concept has been extended at the level of structure and is associated with the notions of strength and stiffness of the whole structure. However, after years of using this concept, this parameter still has been remained ambiguous. In the practice of plastic design of structures, ductility defines the capacity of a structure to undergo deformations after its initial yield without any significant reduction in ultimate strength. The ductility of a structure allows prediction of the ultimate capacity of a structure which is the most important criteria for designing structures under ultimate loads. Limit design of structures postulated that plastic hinges have a sufficient rotation capacity. Therefore, it is clear that the cross-section of members must satisfy stringent geometrical requisites in order to allow plastic deformations until the collapse mechanism of the structure is reached without losing its load carrying capacity. The rotation capacity of steel members is damaged by the occurrence of local buckling in the plate elements which constitute the member cross-section and, if torsional restraints are not provided, by the occurrence of lateral torsional buckling.

1.1 Definition of rotation capacity

To evaluate rotation capacity at the member level, one must recognize that there is no standard definition of the ultimate rotation capacity for flexural members, which is universally accepted by all the specialists. There are three different approaches to determine the rotation capacity.

1- Definition of rotation capacity related to the maximum moment.
2- Definition of rotation capacity related to the fully plastic moment.
3- Definition of rotation capacity related to the post-buckling slope.

According to research conducted by Gioncu and Petcu (1997), the approach related to the fully plastic moment which is also proposed by EC8, seems to be the most reliable (See Fig. 1). The formula to calculate the ultimate rotation capacity is given by

$$\mu_\theta = \frac{\theta_{ru}}{\theta_p} = \frac{\theta_u}{\theta_p} - 1 \tag{1}$$

Where θ_u is the ultimate rotation, θ_p is the rotation corresponding to the first plastic hinge, θ_{ru} is the plastic ultimate rotation.

In the case of uniform moment, where the central part remains in plastic range without excursion into the hardening range, in many cases during experimental tests, the yielding plateau occurs under the reference values M_p. Therefore the above definition of rotation capacity cannot be used. For this case, the rotation capacity can be determined for a reduced plastic moment $0.9\,M_p$, in accordance with EC8 proposal (See Fig. 1):

$$\mu_{\theta 0.9} = \frac{\theta_{ru0.9}}{\theta_p} = \frac{\theta_{u0.9}}{\theta_p} - 1 \tag{2}$$

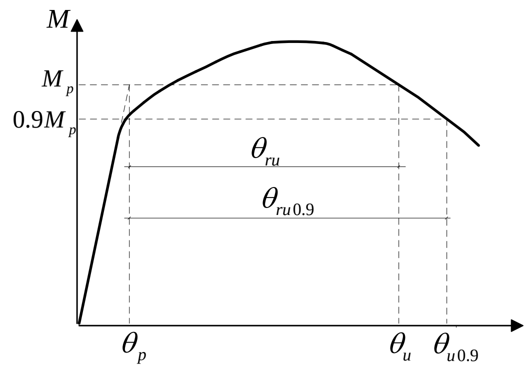


Figure 1: Definition of rotation capacity related to fully plastic moment

Where the index 0.9 refers to the case when the rotation capacity is determined for $0.9M_p$.

The alternative to use M_p or $0.9M_p$ for evaluating the rotation capacity remains a decision of the designer, in function of the beam behavior. For gradient moment it is possible to choose both procedures, but in case of quasi-constant moment the reduced moment use is recommended (Nakashima, 1992, 1994; Gioncu and Mazzolani, 2002). In this research a four point bending load was employed which produce a constant moment in beam models. A new method to determine the ductility for uniform moment loading is presented in current research based on an equivalent reduced plastic moment instead of theoretical plastic moment value M_p. Afterward a verification study was carried out to ensure the accuracy of results of this method. The main concern of the current study is to present a possible classification of beams with I-cross section for the Chinese steel design code in which local and overall buckling modes are examined. In this proposal local buckling is considered interactively for the cross-sectional walls, as well as lateral-torsional buckling modes. Therefore the classification refers to the member rather than the cross section properties.

1.2 A new method to determine the ductility for uniform moment loading

In the case of constant moment loading, which is used in the current research, the moment-rotation curve in most cases didn't reach to theoretical value of plastic moment (M_p). Hence a reduced plastic moment value had to be used to determine the rotation capacity. In this research a new method was employed to calculate the ultimate rotation capacity. Based on the obtained results of numerical study for different moment gradient loading in simply doubly supported beams, it was observed that there is merely one plastic hinge almost in most cases (See Fig. 2). Number of plastic hinges can be determined by N=r+1, where N is number of plastic hinges in structure and r is degree of indeterminacy. Thus in a simply supported beam with any loading conditions the number of the plastic hinge is equal to one. With this assumption, the rotation value of 3-point bending can be taken equal to that of 4-point bending when it is focused on collapse mechanism.

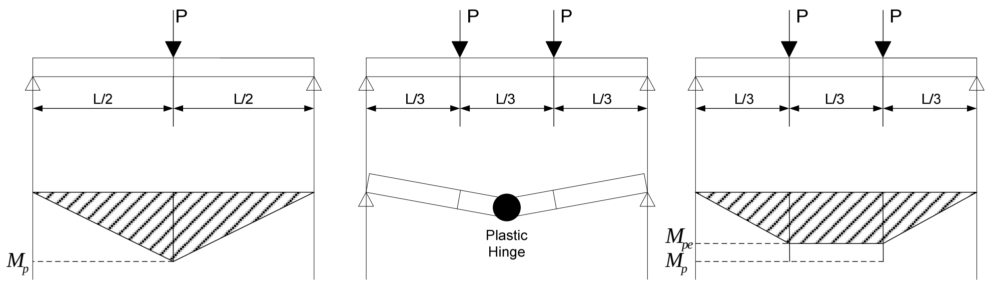


Figure 2: Moment gradient VS constant moment loading and forming plastic hinge,

Based on this assumption, an equivalent rotation for 4-point bending is defined in this paper as below:

$$\theta_{3P} = \theta_{4P-Eq} \tag{3}$$

The beam end rotation can be determined by the integration of the curvature diagram between support and mid-span (See Fig. 2), the rotation corresponding to the plasticization of flanges results:

$$\int_0^{L/2} \chi_{3P} = \int_0^{L/2} \chi_{4P-Eq} \tag{4}$$

$$\frac{M_p L}{4EI} = \frac{M_{pe} L}{3EI} \tag{5}$$

For 3-point bending (moment gradient) plastic moment is:

$$M_p = Z_p f_y \tag{6}$$

For 4-point bending (constant moment) an equivalent value for plastic moment and plastic modules is considered in this research as:

$$M_{pe} = Z_{eq} f_y \tag{7}$$

$$Z_{eq} = f_{eq} Z_p \tag{8}$$

Where M_p is plastic moment, Z_p is plastic modulus, f_y is yield stress, M_{pe} is equivalent plastic moment, Z_{eq} is equivalent plastic modulus, and f_{eq} is equivalent factor. Results illustrated that equivalent factor for 4-point bending is equal to 0.75 (See Fig. 2).

Above procedure may adjust the moment-rotation curve to meet the theoretical value of plastic moment. Nonetheless the theoretical study shows that there is some difference between the amount of ductility for 3-point and 4-point bending. Hence the theoretical values of rotation capacity for these two different loading patterns are required to be determined. Afterward a correction coefficient to adjust the value of rotation capacity with that of 3-point bending is derived. For 3-point bending (See Fig. 3), the length of the fully plastic zone is given by Gioncu and Mazzolani (2002):

$$\frac{L_p}{L} = 1 - \left(\frac{M_p}{M_{ph}}\right) \tag{9}$$

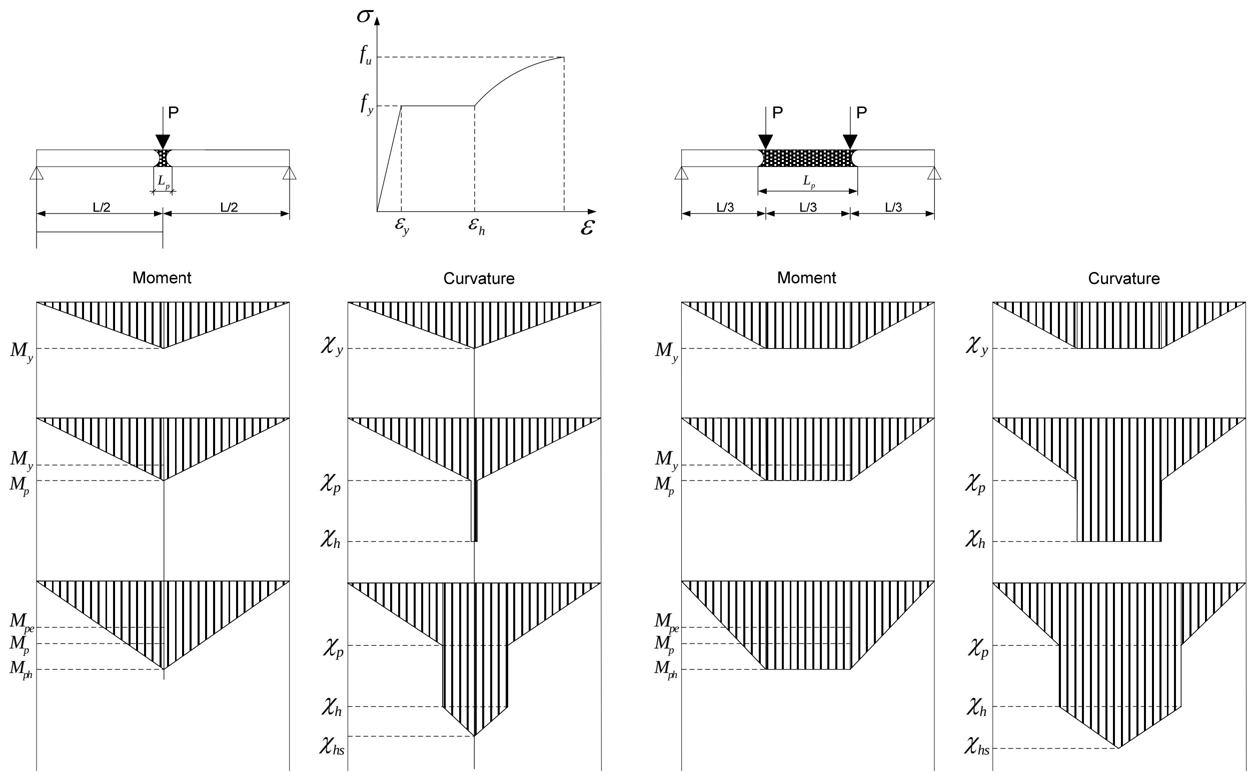


Figure 3: Moment and curvature diagram for constant moment and moment gradient in hardening strain portion

The beam end rotation can be determined by the integration of the curvature diagram (See Fig. 3).

$$\theta_p = \int_0^{L/2} \chi d_x \tag{10}$$

$$M_{ph} = m_h M_p \,;\ \chi_{ph} = s_h \chi_p \,;\ s_h = s + (m_h - 1)e_h \,;\ e_h = \frac{E_h}{E}$$

Where s is a factor measuring the length of the yielding plateau
The rotation in the strain-hardening range is:

$$\theta_u = \int_0^{\frac{L}{2}-\frac{L_p}{2}} \chi d_x + \int_{\frac{L}{2}-\frac{L_p}{2}}^{\frac{L}{2}} \chi d_x \tag{11}$$

The rotation ductility corresponding to plastic results (Ginocu, 1999) is:

$$\mu_\theta = \left(\frac{\theta_u}{\theta_p}\right) - 1 \tag{12}$$

$$\mu_{\theta-3p} = \left(\frac{1}{m_h}\right)(m_h - 1)(s + e_h m_h - e_h) \tag{13}$$

Where $\mu_{\theta-3p}$ is theoretical value of rotation capacity for 3-point bending. With the same procedure it is possible to derive a formula for 4-point bending as below (See Fig. 3):

$$\frac{L_p}{L} = 1 - \left(\frac{2M_p}{3M_{ph}}\right) \tag{14}$$

$$\mu_{\theta-4p} = \left(\frac{1}{2m_h}\right)\left(1 + 3sm_h - 2s + 3e_h m_h^2 - 5e_h m_h + 2e_h - 2m_h\right) \tag{15}$$

Where $\mu_{\theta-4p}$ is theoretical value of rotation capacity for 4-point bending.
The ratio of 3-point to 4-point bending is used in this paper as correction coefficient:

$$Coeff_{\left(3p/4p\right)} = \frac{\mu_{\theta-3p}}{\mu_{\theta-4p}} = \frac{2\left(m_h - 1\right)\left(s + e_h m_h - e_h\right)}{\left(1 + 3sm_h - 2s + 3e_h m_h^2 - 5e_h m_h + 2e_h - 2m_h\right)} \tag{16}$$

The results of current method to determine the rotation capacity of 4-point bending beams illustrate that the new method is more accurate than other methods and results are closer to 3-point bending as a reference value of ductility for flexural members. In this research, a comparative study between the obtained results was carried out to verify the present method against other methods.
Fig. 4 shows three moment-rotation curves which all resulted from numerical study. 3P represents 3-point bending and its obtained result was selected as reference value in this comparative study. 4P-Original and 4P-New Method illustrate the results of constant moment loading before and after applying the equivalent method, respectively.

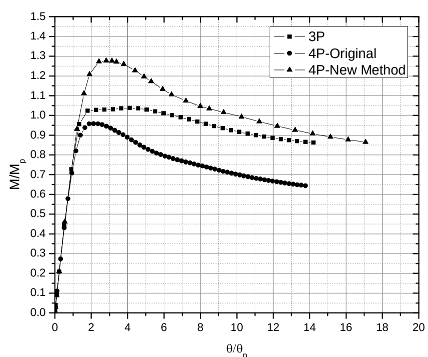


Figure 4: Moment-rotation results to compare proposed equivalent method with three point bending

As it can be seen from Fig. 4, the original moment-rotation curve for 4-point bending cannot reach to M_p. It means according to definition of ductility, the amount of ductility cannot be determined for this case directly. As it mentioned earlier, in previous researches, a reduced plastic moment was proposed which can be used for the constant moment loading. The results show that the equivalent method presented in this research is more accurate when compared with previous methods. Table 1 presents ductility values for four different flange slenderness ratios (b represents free outstands of a compression flange) of a homogenous built-up I-section made of Q345 steel grade. These values were calculated using different methods to obtain the ductility of

4-point bending. The new method presented herein illustrates the obtained results are in closer agreement with 3-point bending.

Table 1: Results of ductility of different method

Flange Slenderness (b/t_f)	3	5	9	11
3P	17.58	15.02	4.42	1.92
4P-New method	16.67	15.92	3.57	2.02
$4P-0.95M_p$	20.96	17.75	0.85	0.54
$4P-0.90M_p$	21.55	18.30	1.61	0.98

2. Section and member classification

In this section after a short review of the recent study on the evaluation of ductility at the section and member level, a comparative study on the design provisions of some major steel design codes was carried out to investigate the differences between these codes.

The moment, shear, and concentrated load bearing resistances of beams with slender plate elements can be significantly influenced by local buckling considerations. For this reason, beam cross-sections are classified as different classes based on the ability of the elements to resist local buckling. For practical purposes, Kato (1988, 1989, 1990), presented the interaction between the two buckling modes for I-sections and square box sections another relationship for interaction flange-web buckling was proposed by Yabuki et al. (1995) (See Table 2). A comprehensive review of the local buckling and section ductility classes of a number of specifications was presented by Bild and Kulak (1991). The analysis was performed for Canada, USA, Germany, Switzerland, UK, Australia specifications and international codes such as ISO and Eurocode 3.

Table 2: Different proposals for classification based on section ductility (N, mm)

	Ductility Class		
	I	II	III
Kato's proposal (width to thickness limitation for sections)	$\frac{b}{2t_f}+0.09\frac{d}{t_w}\le\frac{248}{\sqrt{f_y}}$	$\le\frac{297}{\sqrt{f_y}}$	$\le\frac{333}{\sqrt{f_y}}$
	Applicable for $d\ge b$, $\frac{b}{t_f}\le\frac{249}{\sqrt{f_y}}$, $\frac{d}{t_w}\le\frac{1143}{\sqrt{f_y}}$		
	Plastic sections	Compact sections	Semi-compact sections
Yabuki's proposal (flange and web slenderness)	$\left(\frac{\bar{\lambda}_f}{0.4}\right)^2+\left(\frac{\bar{\lambda}_w}{0.5}\right)^2\le1$	$\left(\frac{\bar{\lambda}_f}{0.5}\right)^2+\left(\frac{\bar{\lambda}_w}{0.8}\right)^2\le1$;	$\left(\frac{\bar{\lambda}_f}{0.7}\right)^2+\left(\frac{\bar{\lambda}_w}{1.1}\right)^2\le1$
		$\left(\frac{\bar{\lambda}_f}{0.4}\right)^2+\left(\frac{\bar{\lambda}_w}{0.5}\right)^2>1$	$\left(\frac{\bar{\lambda}_f}{0.5}\right)^2+\left(\frac{\bar{\lambda}_w}{0.8}\right)^2>1$
	For flange:	$\bar{\lambda}_f=\left(\frac{f_y}{\sigma_{crf}}\right)^{1/2};\sigma_{crf}=0.425\frac{\pi^2E}{12(1-\upsilon^2)}\left(\frac{t_f}{b/2}\right)^2$	
	For web:	$\bar{\lambda}_w=\left(\frac{f_y}{\sigma_{crw}}\right)^{1/2};\sigma_{crw}=23.9\frac{\pi^2E}{12(1-\upsilon^2)}\left(\frac{t_w}{d}\right)^2$	

Where b is flange width, d is web depth, t_f is flange thickness, t_w is web thickness, f_y is nominal yield stress, E is elastic modules and υ is Poisson's ratio

For ductility at the member level which is mentioned earlier, there are some literatures proposing classification criteria for members according to their ductility classes. Among these researches, the Mazzolani and Piluso (1993) proposal seems to be the most rational one.

1) HD, high ductility, $\mu_{\theta r} \geq 7.5$
2) MD, medium ductility, $4.5 \leq \mu_{\theta r} < 7.5$
3) LD, low ductility, $1.5 \leq \mu_{\theta r} < 4.5$

Where $\mu_{\theta r}$ is rotation capacity of flexural member.

A comparative study was carried out by Gioncu (2000), between above-mentioned member behavioral classes and cross-sectional classes of EC3. Results illustrate great differences between the two classification criteria. They presented that the member classification is more appropriate for checking the structural ductility than the cross-sectional classes. Another classification of beams with I-sections was proposed by Vayas and Rangelov (2001), taking into account the interactive effects between local buckling and lateral torsional buckling but only considering the moment gradient loading. The limits of the different classes for welded I-sections, according to the provisions of some major steel design codes include Eurocode 3, DIN18800, BS5950-1, AISC, AIJ, GB50017 are summarized in Table 3.

Table 3: Limiting width to thickness ratio for flanges and webs classification of welded I cross-sections in bending

Design code	Class 1	Class 2	Class 3
Eurocode 3 Part 1.1	$c/t_f \leq 9\varepsilon$ $d/t_w \leq 72\varepsilon$	$c/t_f \leq 10\varepsilon$ $d/t_w \leq 83\varepsilon$	$c/t_f \leq 14\varepsilon$ $d/t_w \leq 124\varepsilon$
BS 5950 Part 1	$b/t_f \leq 8\varepsilon$ $h_w/t_w \leq 80\varepsilon$	$b/t_f \leq 9\varepsilon$ $h_w/t_w \leq 100\varepsilon$	$b/t_f \leq 13\varepsilon$ $h_w/t_w \leq 120\varepsilon$
DIN 18800 Part 1	$c/t_f \leq 9.1\varepsilon$ $d/t_w \leq 64.7\varepsilon$	$c/t_f \leq 11.1\varepsilon$ $d/t_w \leq 74.8\varepsilon$	$c/t_f \leq 13\varepsilon$ $d/t_w \leq 134.4\varepsilon$
AISC LRFD	$b/t_f \leq 11\varepsilon$ $h_w/t_w \leq 110\varepsilon$	$b/t_f \leq 18\varepsilon$ $h_w/t_w \leq 166\varepsilon$	
AIJ LSD	$\left(\frac{b/t_f}{13\varepsilon}\right)^2 + \left(\frac{h_w/t_w}{83\varepsilon}\right)^2 \leq 1$	$\left(\frac{b/t_f}{13.5\varepsilon}\right)^2 + \left(\frac{h_w/t_w}{89\varepsilon}\right)^2 \leq 1$	$\left(\frac{b/t_f}{15\varepsilon}\right)^2 + \left(\frac{h_w/t_w}{97\varepsilon}\right)^2 \leq 1$
GB50017		$b/t_f \leq 13\varepsilon$ $h_w/t_w \leq 80\varepsilon$	

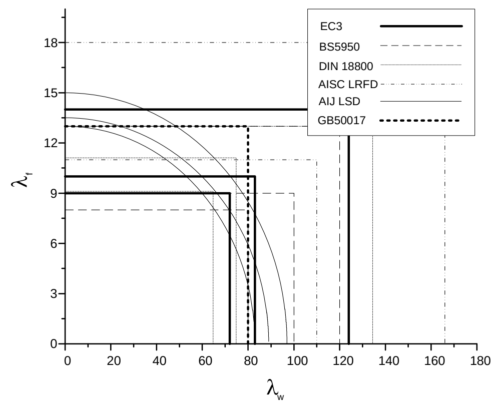


Figure 5: Cross section classification according to various design codes

Accordingly to clarify limit values of flange and web slenderness, Fig. 5 shows limit values of different design codes based on Table 3. The dimensions of the cross sections are shown in Fig. 6. The parameter ε is given by $\varepsilon = \sqrt{235/f_y}$, f_y being the yield strength of the steel in N/mm^2 .

As it can be seen in Fig. 5, most of these design codes such as EC3, BS, DIN and AISC just considering the width to thickness ratios of flanges and web independently without paying attention to interaction modes except the Japanese standard. In the current Chinese steel design code, GB50017, there is no classification criterion, which is the main concern of current research to propose a new method based on China steel productions for new version of the Chinese steel design code. Furthermore except AISC and GB 50017 all of codes in Table 3 consider three boundary lines to separate four distinguished classes, unlike AISC which dividing cross sections into three classes.

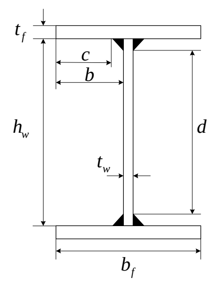


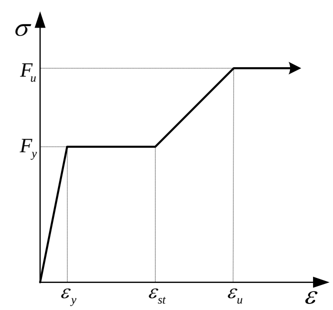
Figure 6: Notation of dimension for cross sections

Generally, the member resistance to lateral-torsional buckling has to be verified after the performance of the global analysis. As a result, the member resistance or ductility may be reduced below the limits of the stated requirements for a specific type of analysis. DIN 18800 does not include any specific requirements for lateral bracing. EC3 states that when the plastic-plastic method is used, lateral bracing must be provided at plastic hinges without any specification of the maximal distance. AISC LRFD goes further and proposes limiting values of the slenderness $\overline{\lambda}_{LT}$ against lateral-torsional buckling for cases where the plastic moment is used for member design. Here again, only the Japanese Code includes lateral-torsional buckling in classification procedure, setting limiting values of $\overline{\lambda}_{LT}$ for various classes. The member may then be classified in a particular class if both its cross-section and its slenderness for lateral-torsional buckling comply with the limits of this class. However, the limits for local and lateral-torsional buckling are considered independently.

In some recent studies, Brune (1999) derived limiting width-to-thickness ratios of webs and flanges for the different classes of cross sections. The study was based on the application of the effective width method using a strain-oriented formulation for the plate slenderness, as proposed by Vayas and Psycharis (1992). Similar studies were conducted on I-girders composed of high strength steel (Earls, 1999). It was observed that a de-coupling of local and lateral-torsional buckling phenomena is not possible. Kemp (1986) proposed a model to account for the interaction between local flange, local web, and lateral-torsional buckling. Kemp (1996) analyzed test results and noted the difficulty in the derivation of a relationship between the available rotation capacity and the slenderness ratios of the cross-sectional walls when considered separately. He found the existence of a much better relationship between rotation capacity and generalized slenderness, if the latter included the slenderness of both local and lateral-torsional buckling.

3. Numerical model

An experimentally verified nonlinear finite element modeling techniques were used in this research to carry out an extensive parametric study on doubly symmetrical homogeneous I-section steel beams to investigate the influence of local and overall slenderness as well as their interaction on the member ductility. All models considered in this paper were subjected to uniform moment loading and cross sections considered have a wide range of flange and web slenderness as well as lateral support configurations. Q345 steel grade is used for all cross section in this paper, according to Chinese Standard for High Strength Low Alloy Structural Steels GB/T1591. The representative material response parameters are given in terms of an idealized uniaxial material response data in Fig. 7.



Material	F_y	F_u/F_y	E_{st}/E	$\varepsilon_{st}/\varepsilon_y$	$\varepsilon_u/\varepsilon_y$
Q345	345	1.36	0.00347	14.93	119.42

Figure 7: Uniaxial stress-strain response of Q345 steel used in models

The multipurpose finite element software package, ANSYS 14.5, was used for the finite element modeling. ANSYS is widely recognized as being a very appropriate tool for use in structural analysis problems involving a high degree of geometric and material nonlinearities. The models of the beams considered in this study were constructed from a dense mesh of four node nonlinear shell finite elements. The SHELL181 from the ANSYS element library employed in this research

is a 4-node element with six degrees of freedom at each node and is well-suited for nonlinear applications.
To produce the constant moment in the finite element models, two concentrated loads located at $L/3$ along the span length of a simply-supported I-shaped beam assembly are imposed. The length of the beams is chosen to be 3.0 m, and four stiffeners are considered in each side of beam model. Compression and tension flanges are restrained against out of plane translation for several bracing configurations in an idealized way. In this research 6 bracing configurations were considered as LT1: $l_1 = (L/30)$, LT2: $l_1 = (L/12)$, LT3: $l_1 = (L/6)$, LT4: $l_1 = (L/3)$, LT5: $l_1 = (L/2)$, and LT6: $l_1 = (L)$, (where l_1 is the ratio of the unsupported length, and L is beam span) in order to investigate the effects of lateral-torsional buckling on the member ductility and its interaction with local buckling modes. Meanwhile various section geometrical configurations employed in the present work to investigate the influence of local buckling modes on the member ductility for classification of flexural members.
In this research a verification study was carried out, the numerical verification model is a representation of the experimental work performed by Green et al. (2002) on a simply supported doubly symmetric beam subjected to a point load at mid span. This point load produces a moment gradient along the longitudinal axis of the beam. The numerical model was created in accordance with the model dimensions and material values given for Test Specimen 5. The material used for the verification numerical model is the conventional steel, A36. The rotation capacity (R) reported in the experimental results of Test Specimen 5 was 9.69 (Green, 2000). The current finite element verification model of Test Specimen 5 obtained a rotation capacity of 9.6. Moment-rotation curves for experimental and finite element verification model of Test Specimen 5 are provided in Fig. 8. As can be seen in this figure, the experimental test of Test Specimen 5 and the numerical verification modeling results of Test Specimen 5 are in close agreement.

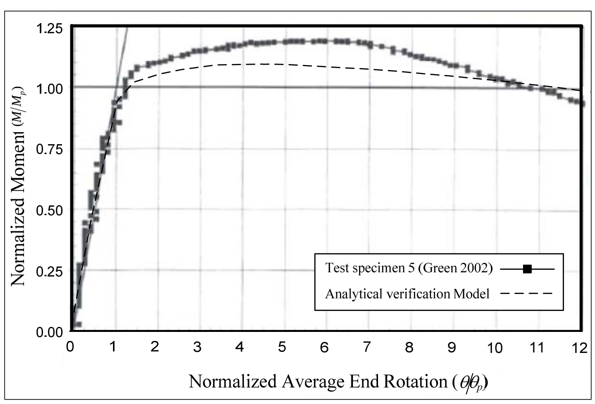


Figure 8: Moment rotation response of verification study

4. Parametric study

To investigate the behavioral classes of member subject to bending, a parametric study was employed in this research. This study was carried out on welded beams with I-cross sections by using the following parameters to primarily influence the flexural member response:

- Flange slenderness, λ_f, in the form of the limiting width to thickness ratios adopted by EC3: $\lambda_f = c/\varepsilon t_f$

Where $c = 0.5(b_f - t_w) - 1.1a$ (for fillet welds) and a is weld size (Anastasiadis et al, 2012)

- Web slenderness, λ_w, accordingly:

  $\lambda_w = d / \varepsilon t_w$

- Relative slenderness for lateral-torsional buckling defined as

  $\bar{\lambda}_{LT} = \sqrt{M_p / M_{cr}}$

To cover a wide range of possible practical applications, the values of the above parameters were varied as listed in Table 4.

Table 4: Extent of parametric study

Parameter	λ_f	λ_w	$\bar{\lambda}_{LT}$
	6, 8, 10, 12, 14, 16, 18, 20	60, 70, 80, 90, 100, 120, 140, 150, 160, 180	0.03 ~ 1.15

Based on the main objective of the current research, the results are expressed in terms of rotation capacity R to distinguish where the boundary would be between Classes 1 and 2, the strength ratio M_u / M_p to establish the limit between Classes 2 and 3, and the strength ratio M_u / M_y for the limit between Classes 3 and 4.

Fig. 9 illustrates the moment rotation curves for different buckling modes. Each group of curves shows the independent influence of each buckling mode. Interaction between local buckling and local-overall buckling modes are not included in these figures. As it can be seen in Fig. 9a and Fig. 9b, increasing the flange and web slenderness ratios may decrease the ultimate rotation θ_u of flexural member. On the other hand, compared with flange slenderness limit ratio, the web slenderness ratio has more influence on reducing the value of moment capacity. Fig. 9c shows that the lateral support configuration has a drastic influence on rotational behavior of members subjected to bending. It was also observed that lateral slenderness may rapidly change the maximum value of the M/M_p ratio compared with the other two local slenderness ratios, which can have a direct effect on the boundary locations of Class 2 and Class 3.

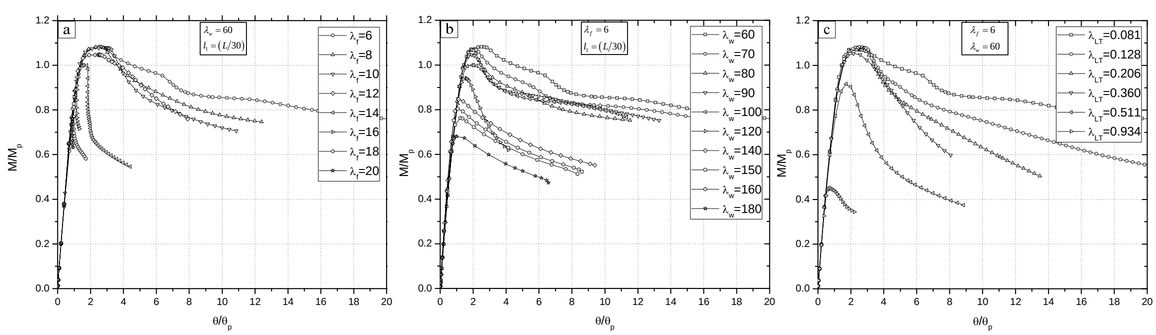


Figure 9: Influence of local and overall slenderness on rotational behavior. (a) flange slenderness, (b) web slenderness, (c) overall slenderness

After applying the definition of member ductility, results for local buckling modes are plotted in Fig. 10, non-dimensional slenderness ratios for flange and web versus member rotation capacity.

As a result, increasing local slenderness ratios decrease ductility of member. The maximum values of flange and web slenderness ratios illustrate a minimum amount of rotation capacity which is almost equal to zero.

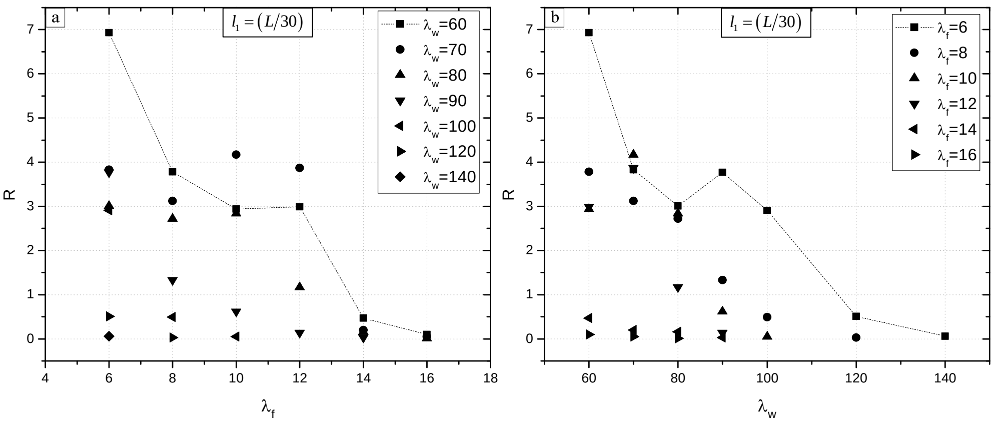


Figure 10: Influence of local buckling modes on rotation capacity, (a) effect of flange local buckling modes (b) effect of web local buckling modes

Fig. 11 shows the mutual influence of flange and web local buckling modes on the rotation capacity in a three-dimensional space, a nonlinear surface was fitted to these points from the current analytical results:

$$R = -0.524 + 92.98 \exp^{(-\lambda_f/6.08)} \exp^{(-\lambda_w/37.63)} \tag{17}$$

Eq. 17 expresses a three-dimensional relationship between flange and web slenderness with rotation capacity for flexural members.

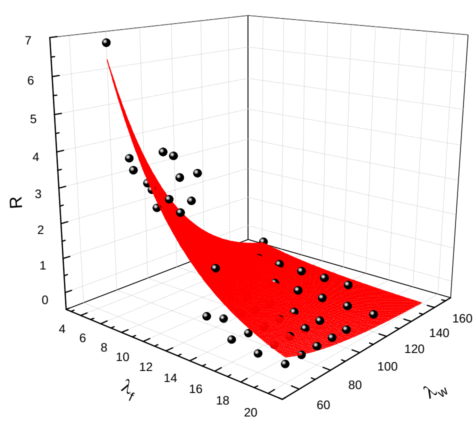


Figure 11: Influence of local interaction buckling modes on rotation capacity in three-dimensional space

In fact, this surface represents the local interaction buckling for I-sections subjected to bending. Substituting various values for rotation capacity R in Eq. 17 provides the relation between the flange and web slenderness for different R values. Fig. 12 illustrates the interaction between the flange and web slenderness for rotation capacity from 0 to 5. This figure indicates that for higher values of R, the interaction bounds getting closer to each other.

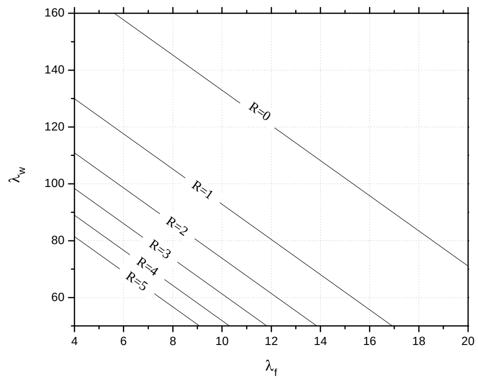


Figure 12: Interaction between flange and web for different values of rotation capacity

According to limits between classes in steel design codes mentioned earlier, the definitions of Classes 2 and 3 are clear (Loorits, 1995). For the limit between Classes 1 and 2 some discrepancy exists between the codes with regards to the appropriate value of the rotation capacity. EC3 does not propose a certain value, stipulating that for Class 1 ‘sufficient’ rotation capacity must be available to allow for a plastic redistribution of the bending moments. The AISC LRFD specification states in the Commentary a value for the rotation capacity R=3 for Class 1 sections, while the AIJ LSD Code stipulates R=4. In this research the former value is adopted. Accordingly, the criteria for the different classes are the following:

- For Class 1: $R \geq 3$
- For Class 2: $M/M_p \geq 1$
- For Class 3: $M/M_y \geq 1$

Fig. 13 shows the flange and web slenderness limits for the current parametric results based on the criteria for compact class of AISC, employed in current research as Class 1.

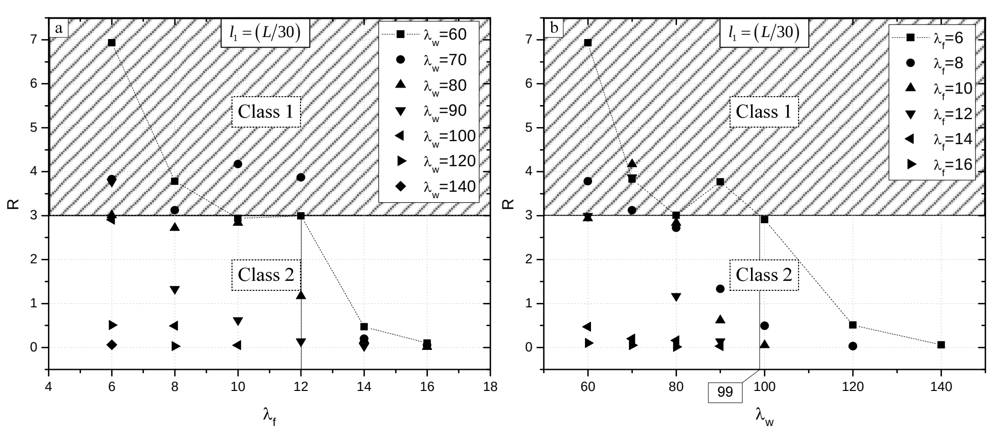


Figure 13: Limit values of flange and web slenderness for Class 1, (a) flange slenderness limit value (b) web slenderness limit value

Fig. 14 and Fig. 15 illustrate flange and web slenderness limits to identify Class 2 and Class 3 plotted for (M/M_p) and (M/M_y) versus flange and web slenderness ratios respectively. As previously outlined the ratio of (M/M_p) and (M/M_y) must be greater than 1 for Class 2 and Class 3 respectively.

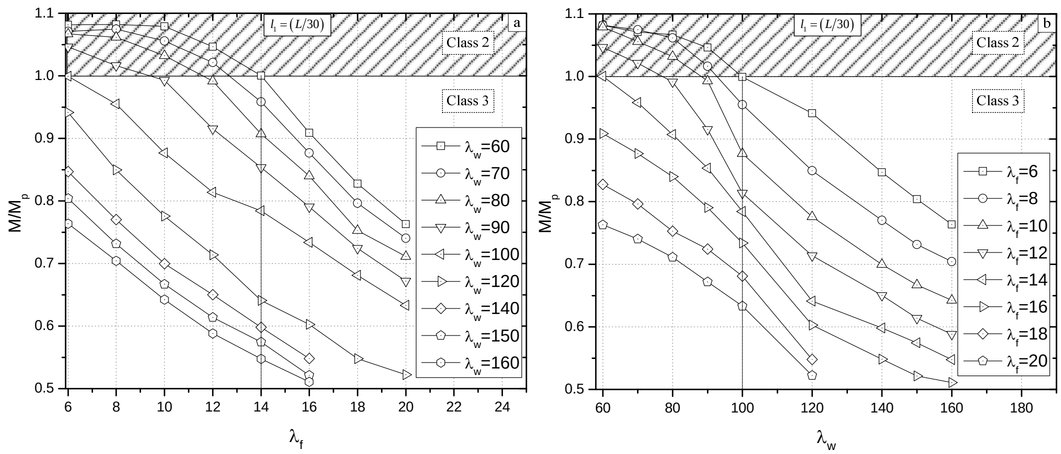


Figure 14: Limit value for Class 2. (a) flange slenderness limit, (b) web slenderness limit

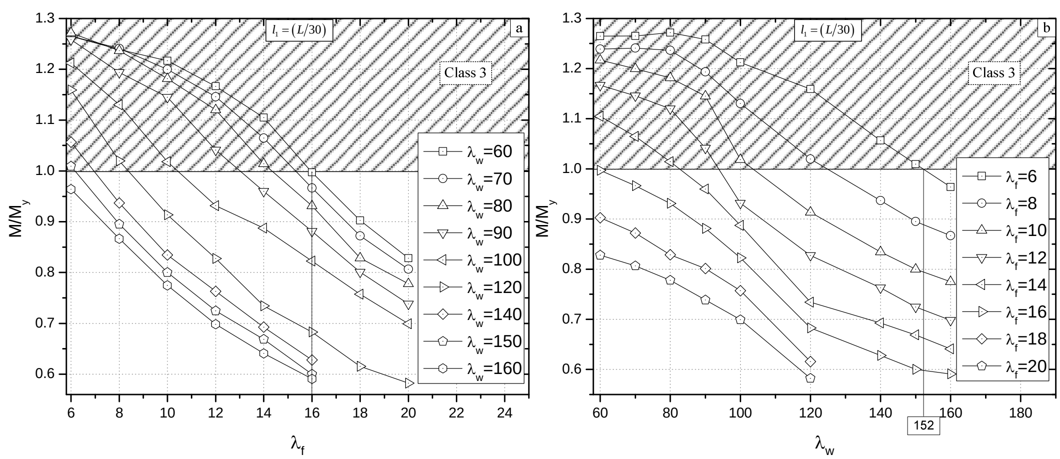


Figure 15: Limit value for Class 3. (a) flange slenderness limit, (b) web slenderness limit

Member ductility requires more parameters to be considered than section ductility. The most effective parameter is relative slenderness for lateral-torsional buckling which was defined earlier. Fig. 16 shows the limit values for this parameter for Class 1 to Class 3 according to the classification criteria that employed in this research. Fig. 16a shows the limit value for the lateral slenderness ratio with a plot of rotation capacity versus lateral torsional slenderness considering ductility requirement for Class 1. Fig. 16b and Fig. 16c identify the limit values of lateral slenderness for Class 2 and Class 3.

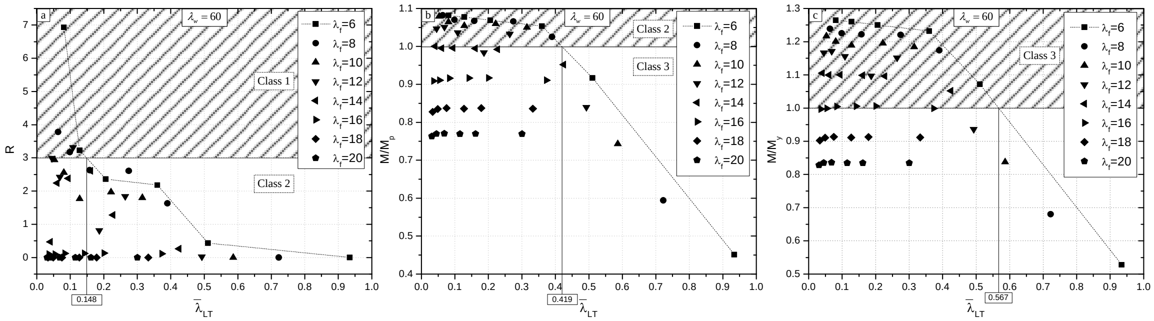


Figure 16: Limit values for relative lateral-torsional buckling slenderness, (a) Class 1, (b) Class 2, (c) Class 3

Considering member ductility instead of section ductility requires that local-overall interaction buckling modes to be taken into account. To account for the interaction of the different buckling modes, a generalized slenderness criterion is required. It must include the flange slenderness λ_f , the web slenderness λ_w and, the lateral-torsional buckling $\bar{\lambda}_{LT}$ slenderness parameter. As it has been proposed by EC3, λ_f and λ_w are normalized by the limiting values for Class 1, λ_{f1} and λ_{w1}, respectively. Following the procedure of EC3, Annex Z has been examined. The following expressions have been proven as most appropriate (Kemp, 1986):

$$\bar{\lambda}_R = \bar{\lambda}_{LT} \left(\frac{\lambda_f}{9}\right)\left(\frac{\lambda_w}{72}\right) \tag{18}$$

$$\bar{\lambda}_M = \bar{\lambda}_{LT} \left(\frac{\lambda_f}{9}\right)^{1/3}\left(\frac{\lambda_w}{72}\right)^{1/5} \tag{19}$$

The present research attempts to identify boundaries between different classes while focusing on local-overall interaction as well as local interaction buckling modes.
Through using a generalized slenderness concept, Fig. 17 presents the local-overall interaction influence as a means to classify flexural members based on member ductility from the obtained FE results of the current research. For each class, a specific slenderness limit value presents to identify the boundary between different classes.

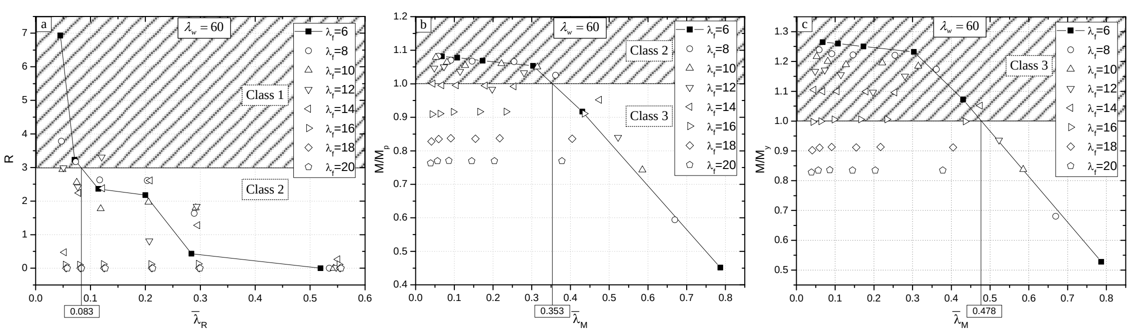


Figure 17: Limit values for generalized slenderness, (a) Class 1, (b) Class 2, (c) Class 3

Fig. 18 shows a comparison between the current proposal and local slenderness limit values of different classes in AISC and EC3. Class 1 of the proposal for both flange and web local slenderness ratios is almost close to the compact sections of AISC, Class 2 is equal to Class 3 of EC3 for flange slenderness limit value, whereas the web local slenderness ratio of Class 2 is almost the same as the Class 1 of this proposed classification and those are close to the compact sections of AISC. Class 2 of the proposal is between the compact and non-compact sections of AISC for flange slenderness limit values. On the other hand both flange and web slenderness for Class 3 of the proposal are almost close to the non-compact sections of AISC.

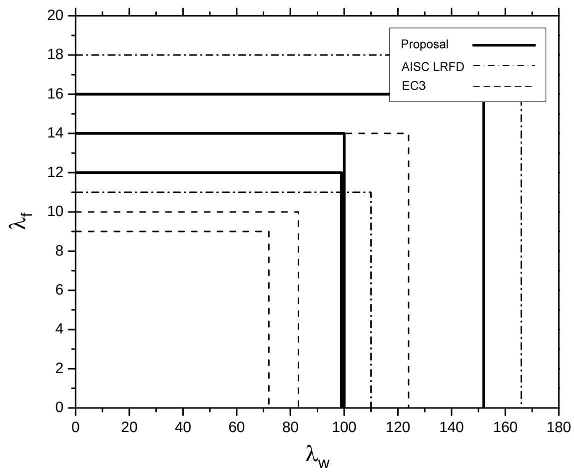


Figure 18: Comparison between the limit values of the proposal and various design codes

5. Conclusion

For I-shaped sections, their flexural behavior is governed by the buckling of flange and web plates, for which independent limitations are proposed by Eurocode 3. This assumption is unreasonable because, obviously, the flange is restrained by the web and the web is restrained by the flanges. Therefore, the interaction between the two buckling modes must be considered. Meanwhile, at the member level, there are some additional factors influencing the ductility (e.g. the beam span, flange to web width ratio, member slenderness, moment gradient, level and eccentricity of axial load, etc). As a consequence of these factors, it seems that this behavioral class should be replaced by the concept of member behavioral classes.

In this research a proposal for the current Chinese steel design code was presented considering member rather than section ductility. This proposal is based on the effects of local interaction as well as local-overall interaction buckling modes which are not explicitly considered by other international steel design codes. This proposed classification has three classes as given in Table 5 showing the limit values for flange, web and lateral slenderness independently, corresponding to each class. Using a generalized slenderness definition has made it possible to define a new criterion considering the effects of flange and web local buckling as well as lateral-torsional buckling simultaneously. In addition to independent slenderness ratios presented in Table 5, the limit values for generalized slenderness are provided to consider the local-overall interaction buckling modes.

Table 5: Proposal for section classification for I-cross sections subjected to moment

	Class 1	Class 2	Class 3
Flange local buckling	$\lambda_f \leq 12$	$\lambda_f \leq 14$	$\lambda_f \leq 16$
web local buckling	$\lambda_w \leq 99$	$\lambda_w \leq 100$	$\lambda_w \leq 152$
Lateral-torsional buckling	$\overline{\lambda}_{LT} \leq 0.15$	$\overline{\lambda}_{LT} \leq 0.42$	$\overline{\lambda}_{LT} \leq 0.57$
Generalized slenderness	$\overline{\lambda}_R \leq 0.08$	$\overline{\lambda}_M \leq 0.35$	$\overline{\lambda}_M \leq 0.48$

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