# Local Buckling Rules for Rotation Capacity

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## ABSTRACT

Structural frames designed or proportioned to resist seismic forces must possess adequate ductility to redistribute internal forces or have needed energy absorbing capability. This paper addresses the interaction effects of a steel member's plate slenderness values and its lateral slenderness on rotation capacity. It is shown that members with slenderness values close to the limits specified by the codes of practice may not be able to redistribute moments adequately under seismic loading. A critical value for the lateral slenderness is proposed to be used with current local buckling rules prescribed in North American standards for a structure to develop its full ductility potential.

## INTRODUCTION

A desirable attribute of a steel beam subjected to static loading is the maintenance of its full moment capacity under extensive plastic deformation. Such response is perhaps best exhibited in the member's moment-curvature relationship. In recent design codes, members classified as plastic design sections are assumed to sustain a resisting moment without loss of strength until a required rotation capacity is achieved, thus permitting full redistribution of bending moments to other parts of the structure. Figure 1 is indicative of this behaviour. The response of such members to transverse loading is generally well understood up to the initiation of local buckling. However, concern arises about its ability to achieve adequate plastic rotation in the inelastic range. A number of researchers have pointed out weaknesses of some structural elements that satisfy the requirements of plastic design. Among the points mentioned are the rotation capacity and premature local buckling of the members. Thurston[1] expressed concern about the small safety margin implicit in the New Zealand code limits for flange slenderness ($b/2t$) and web slenderness ($h/w$). His opinion was based on earlier than anticipated local flange buckling in the testing of classified plastic design beam sections that resulted in premature moment capacity deterioration. As such, he proposed that the ($b/2t$) ratio limits be reduced. Beamish[2] noted meanwhile, that under cyclic loading, local flange buckling can occur at relatively low ductility in members having plate slenderness ratios close to the plastic design limits. Mitani et al.[3] and Takanashi et al.[4] highlighted the poor ductility achievement of some members complying with the requirements of plastic design. Kemp[5] pointed out the possibility of some steel sections with less than adequate ductility for plastic analysis; he mentioned in particular the influence of the interaction between the different modes of buckling, not considered in existing codes. More recently, Ghobarah et al.[6] pointed out that the slenderness limits ($b/2t$) and ($h/w$) for beams in MRFs in highly seismic areas may be, in some situations, unconservative thus leading to a reduction in load carrying capacity.

The flanges and webs of a beam tend to behave as plate elements.[7] It is well known from the post-buckling theory of plates that, even when distorted after local buckling, plates having edge restraint are still able to sustain significant levels of load. This has been observed through beam bending experiments showing that considerable rotation capacity exists after local buckling is observed.[8]

Frequently, local flange buckling is observed simultaneously with lateral buckling. Alternatively, it has been noted that an interaction of local flange buckling with local web buckling may occur also. In this context, Kato[9,10] proposed interaction relationships relating flange and web slenderness; however, no lateral slenderness was included in thc analysis. About the same time, Kuhlmann[11] showed that the rotation capacity is governed by three parameters—flange slender-

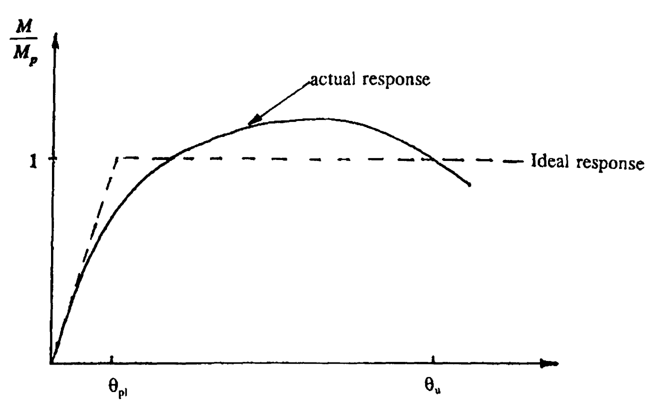


*Fig. 1. Behavior of Class 1 (plastic design) sections under monotonic loading.*

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ness, web slenderness, and the steepness of the moment gradient, defined in terms of unsupported length over the width of the section, $L/2b$. The interaction of the different phenomena mentioned above is of a highly complex nature. As far as design standards are concerned, the Japanese code[12] has attempted to address this issue, while other codes do not cover combined responses even for monotonic loading. For example, slenderness limits for flanges and webs do not reflect combined buckling phenomena. Thus, making design rules simpler may result in potentially inaccurate member selection.

With the aforementioned in mind, a procedure accounting simultaneously for flange and web plate width or depth to thickness ratios and lateral slenderness is proposed. Critical values of normalized flange and web slenderness ratios are obtained in accordance with the normalized lateral slenderness and the required rotation capacity for a condition of plastic design. A tool is offered to researchers and design engineers for defining maximum plate slenderness ratios (flanges and webs), and lateral slenderness requirements to achieve a given amount of rotation capacity.

## REQUIRED ROTATION CAPACITY

For a plastically designed member subjected to bending moment and laterally braced in accordance with the specification in use, the available rotation capacity, $R_u$ is a ductility parameter that determines how effectively internal moments can be redistributed once $M_p$ is reached at a given cross section. It is here defined by the amount of total rotation beyond the plastic limit that can be sustained before unloading below $M_p$ takes place and is herein given by:

$$R_u = \frac{\theta_u}{\theta_{pl}} - 1$$

In general, steel members of an open style cross section may fail by either local buckling or lateral buckling or both. Because of requirements in design to prevent lateral buckling, the most important aspect which governs the rotation capacity of a member is its potential for local buckling. Since the latter mode is crucial, the width-thickness ratios of compression elements will dramatically affect the delivered rotation capacity.

Recently, Bild and Kulak[13] compared rules governing local buckling of plastic sections for 13 different specifications. They noted that the modified flange slenderness, $(b/2t)\sqrt{\sigma_y}$, expressed in SI units, ranges from 131 to 174. Meanwhile, the modified web slenderness, $(h/w)\sqrt{\sigma_y}$, was observed to range from 981 to 1,365.

In seismic design, there is no agreed-upon value for the required rotation capacity. The situation is complicated further by the fact that there are not only different types of ductility to be considered, but also the notion of force modification needs to be embraced. Popov[14] distinguishes among the material capacity, $R_m$, the member capacity, $R_u$, and the system capacity, $R_s$, and suggests the relationship that $R_m > R_u > R_s$. Hall[15] recommends employing an $R_u$ value of $1.5R_s + 0.5$. Meanwhile Chopra and Newmark[16] suggest that it is prudent to provide a required rotation capacity $R_u$ of six to eight while Fukomoto and Itoh,[17] quoting from AIJ LSD[12] propose a value of four. Meanwhile, Cosenza et al.[18] link a member's rotation capacity, $R_u$ to the system ductility, $\mu_s$. They suggest a value for $R_u$ of $(\mu_s - 1)/\psi$ with $\psi$ fluctuating between 0.36 and 1. The current Canadian Code[19] for the design for fully ductile moment resisting frames employs a force reduction factor of four for a system ductility $\mu_s$. If Cosenza's suggestion is to be used, and with $\psi = 0.5$ representing an average of the maximum and minimum suggested values, a required member ductility of six, corresponding to the minimum value suggested by Chopra and Newmark, would then apply for seismic design in Canada. A graphical representation of the system ductility-rotation capacity relationship is given in Figure 2.

In the United States, the force reduction factors $R$ and $R_w$ for NEHRP[20] and UBC[21] respectively are formulated[22] by the expressions $R = R_\mu \Omega$ and $R_w = R_\mu \Omega Y$, where $R_\mu$ is the ductility reduction factor, $\Omega$ is the overstrength factor, and Y is a factor evaluated at 1.4. Newmark and Hall[23] relate the ductility reduction factor to the system ductility as follows:

$R_\mu = \mu_s$ for velocity and displacement amplification regions.

$R_\mu = \sqrt{(2\mu_s - 1)}$ for acceleration amplification region.

The standards NEHRP and UBC do not explicitly specify the value of the overstrength factor. This leaves the possibility of combining the system ductility, $\mu_s$, with the overstrength factor in various ways. If, however, the overstrength factor is chosen to have a value of 2.3 as suggested by Uang, then the force reduction factor of NEHRP and UBC, gives for fully ductile moment resisting frames, average system ductility values, $\mu_s$, of 3.6 in the velocity region and 6.9 in the accel-

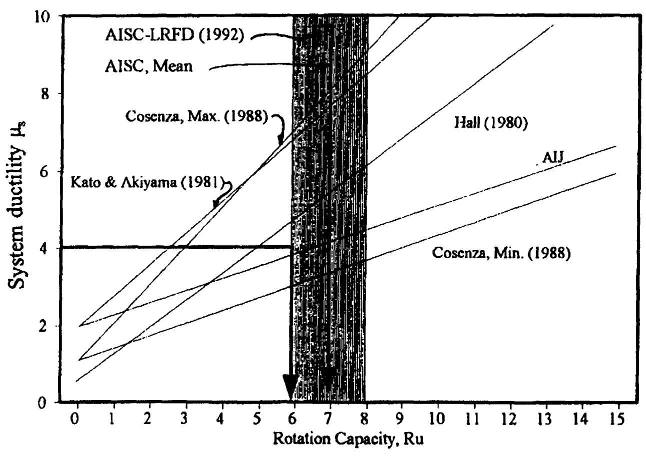


*Fig. 2. System ductility—rotation capacity relationship.*

eration region. If Cosenza's suggestion is to be used, an average required member rotation capacity, $R_u$ of about 4.9 and 11.1 would result for the velocity and acceleration regions respectively. These $R_u$ values represent clearly a much wider range than range six to eight suggested by AISC. For member assessment, we will, however, simply use an average AISC value of seven in the United States.

## AVAILABLE ROTATION CAPACITY

From tests conducted on rolled I-section steel beams in which lateral deflection was prevented at the critical moment section, an empirical expression was developed by Kemp,[23] to fit the predictions of a model for the relationship between the available rotation capacity, $R_u$, and the effective lateral slenderness ratio, $\lambda_e$ given by:

$$R_u = 3\left(\frac{60}{\lambda_e}\right)^{1.5} \quad (1)$$

where:

$$\lambda_e = \frac{1}{23{,}948}\frac{b}{2t\varepsilon}\sqrt{\frac{h}{w\varepsilon}}\,\frac{L}{r_y\varepsilon}\left(460 - 1.13\frac{L}{r_y\,\varepsilon}\right),$$

in which:

$$\varepsilon = \sqrt{\frac{\sigma_o}{\sigma_y}},$$

and $r_y$ is the radius of gyration of the section about its weak axis of bending. Normal stresses $\sigma_o$ and $\sigma_y$ represent nominal and actual yield values respectively. For typical proportions of I-shaped sections, let us assume that $w = 0.6t$ and $2b = 0.5d$. Furthermore, only one flange and one-sixth of the height of the web is assumed to be effective at the onset of the interaction of local plate buckling with lateral buckling. With these assumptions, the lateral slenderness reduces to:

$$\frac{L}{r_y} = 3.76\frac{L}{2b} \quad (2)$$

Substituting from Equation 2 into Equation 1, $\lambda_e$ can be expressed as a function of three normalized slenderness parameters given by:

$$\lambda_e = \alpha_f\sqrt{\alpha_w\alpha_l}\,(.07224 - .000667\alpha_l) \quad (3)$$

where:

$$\alpha_f = \frac{b}{2t\varepsilon},\ \alpha_w = \frac{h}{w\varepsilon},\ \alpha_l = \frac{L}{2b\varepsilon},$$

$$\varepsilon = \sqrt{\frac{300}{\sigma_y}} \text{ in S.I. units or}$$

$$\varepsilon = \sqrt{\frac{43.5}{\sigma_y}} \text{ in ksi units}$$

To best fit the values of Kemp's experimental observations, the simplified empirical expression for $R_u$ (in Equation 1) is multiplied by a correction factor, $2.277 - 0.1433\alpha_l$, that was obtained from a linear regression analysis to give a revised expression for the rotation capacity as:

$$R_u = (6.831 - .43\alpha_l)\left(\frac{60}{\lambda_e}\right)^{1.5} \quad (4)$$

Values of rotation capacity obtained with Equation 4 were then compared with only Class 1 sections or plastic design sections test data of Lukey's[24] and Kuhlmann's[25] experiments. They were found to be in good agreement; a summary of the compared values is given in Table 1. Moreover, a sensitivity analysis was performed on the selected cross-sectional dimensions. Three cases were considered; these are, namely: (1) $2b = 0.5d$ and $w = 0.6t$, (2) $2b = d$ and $w = 0.5t$, and (3) $2b = 0.5d$ and $w = 0.3t$. In all three cases, the ratio of the predicted rotation capacities to the experimentally observed ones had a mean of respectively 0.98, 1.045, and 1.04 and coefficients of variation of 0.133, 0.137, and 0.136.

## DISCUSSION

From Equation 4, available rotation capacity curves (Figures 3–5) for various normalized lateral slenderness and normalized web slenderness values have been developed. Values of normalized flange slenderness were selected to have a range from five to nine while the normalized web slenderness ranged from 25 to 75; these values cover the usual spectrum of Class 1 (plastic design) sections. Three basic values for the normalized lateral slenderness, specifically eight, nine, and ten were examined. It is clearly seen in Figures 3 to 5 that the normalized lateral slenderness considerably influences the local buckling rules. Thus, for example, within the limits $\alpha_f = 8.37$ and $\alpha_w = 63.5$ specified by CSA S16.1 M-89[25] and $\alpha_f = 7.91$ and $\alpha_w = 78.8$ specified by AISC,[26] it is evident that the values of the available rotation capacity decrease by the same order of magnitude both by increasing flange slenderness and lateral slenderness. Meanwhile, $R_u$ decreases by a lesser order of magnitude with increasing web slenderness.

If Canadian buckling limits are used as reference lines (CSA S16.1 M-89), then a careful study of Figure 4 shows that members, with a normalized lateral slenderness of nine and having plate elements satisfying the Class 1 section limits, will reach a required rotation capacity of about six. A normalized lateral slenderness of nine can therefore be taken as the critical limit which should be used in parallel with the current buckling rules in order for a section to be able to redistribute moments in accordance with plastic design. For a normalized lateral slenderness greater than nine, it is clearly shown in Figure 5 that the required value of the rotation capacity of six is reached only if more stringent buckling limits are imposed on the plate elements. Thus, for example, with $\alpha_l = 10$, a normalized flange slenderness of seven would

| Table 1. Comparison of Predicted and Observed Values of Rotation Capacity | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Reference | Spec. # | *d* mm | *b* mm | *t* mm | *w* mm | $\sigma_y$ | $\alpha_f$ | $\alpha_w$ | $\alpha_l$ | Obser. $R_u$ | Predic. $R_u$ |
| Lukey (1969) | 1 | 250 | 203 | 10.8 | 7.6 | 277 | 9.06 | 31.4 | 8.2 | 11.8 | 10.49 |
| | 2 | 250 | 176 | 10.8 | 7.6 | 277 | 7.83 | 31.4 | 8.02 | 13.6 | 13.77 |
| | 3 | 200 | 74 | 5.28 | 4.4 | 372 | 7.79 | 50.1 | 7.8 | 10.4 | 10.42 |
| | 4 | 200 | 86 | 5.28 | 4.4 | 372 | 9.06 | 50.1 | 8.09 | 6.7 | 7.63 |
| | 5 | 250 | 74 | 5.25 | 4.6 | 372 | 7.79 | 60.7 | 7.26 | 13.7 | 10.61 |
| | 6 | 250 | 86 | 5.25 | 4.6 | 372 | 9.06 | 60.7 | 7.57 | 8.0 | 7.69 |
| | 7 | 250 | 90 | 5.25 | 4.6 | 372 | 9.53 | 60.7 | 7.69 | 6.5 | 6.89 |
| Kuhlmann (1989) | 8 | 260 | 141 | 10.2 | 5.5 | 333 | 7.27 | 44.5 | 11.2 | 5.1 | 4.57 |
| | 9 | 259 | 150 | 10 | 5.5 | 333 | 7.91 | 49.6 | 11.26 | 3.8 | 3.64 |
| | 10 | 258 | 160 | 10.4 | 5.5 | 333 | 8.08 | 49.6 | 11.55 | 3.6 | 3.20 |
| | 11 | 220 | 160 | 10 | 5.5 | 333 | 8.43 | 42.1 | 7.26 | 9.5 | 12.4 |
| | 12 | 298 | 160 | 10 | 6 | 333 | 8.43 | 52.3 | 6.58 | 12 | 12.99 |
| | 13 | 299 | 160 | 10 | 6 | 333 | 8.43 | 52.5 | 7.91 | 8.7 | 8.66 |
| | 14 | 299 | 160 | 10 | 6 | 333 | 8.43 | 52.5 | 9.24 | 7.2 | 5.87 |

be required for $\alpha_w = 55$. For the American buckling limits, it is found that a critical lateral slenderness of about eight should be used along the current rules in order to achieve a rotation capacity of about 7.5.

Interaction curves, relating flange slenderness and web slenderness for three basic values of the lateral slenderness and three required rotation capacities are shown in Figures 6 to 8. These were developed using Equation 4. As reference lines in the interaction diagrams, vertical lines positioned at $\alpha_f = 8.37$ and 7.91 represent the CSA S16.1 M-89 and the AISC-LRFD flange buckling limits respectively, while horizontal lines positioned at $\alpha_w = 63.5$ and 78.8 denote the web buckling limits. A study of the interaction curves in Figure 8 shows that members with lateral slenderness values, $\alpha_l$, less than or equal to nine, together with the Canadian code limits will develop an available rotation capacity of six. For greater

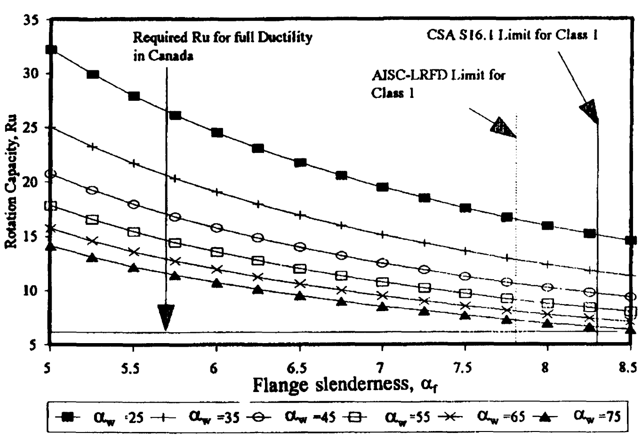


*Fig. 3. Ultimate rotation capacity, $R_u$, for $L/B\xi = 8$.*

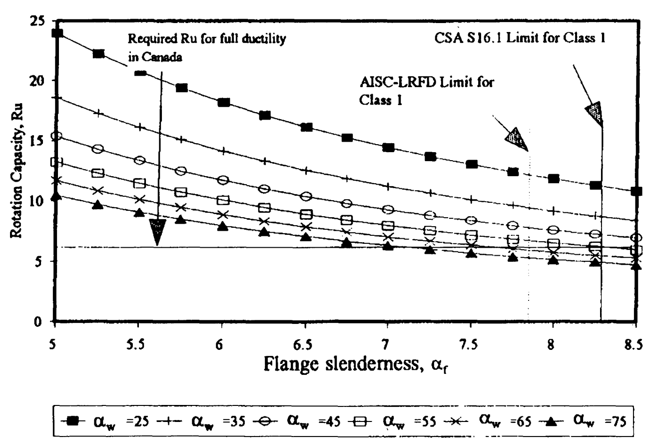


*Fig. 4. Ultimate rotation capacity, $R_u$, for $L/B\xi = 9$.*

values of the lateral slenderness, more stringent limits have to be satisfied for a member to reach an available rotation capacity of six. Interaction diagrams for $R_u$ = 5, 6, and 7 are shown in Figures 7 to 9 respectively.

## EXAMPLE

For the purpose of illustration, two cantilever beam sections subjected to tip loading are selected. The length $L$ is taken as 1,650 mm (65 in.) and $\sigma_y$ = 300 MPa (43.5 ksi).

1. According to CSA S16.1 M-89 local buckling design rules where S.I. units are used, a W310×39 section has:

- $(b/2t) = (165/2 \times 10) = 8.25 < (145/\sqrt{\sigma_y}) = 8.37$
- $(h/w) = (290/6) = 48.33 < (1{,}100/\sqrt{\sigma_y}) = 63.5$
- $(L/r_y) = (1{,}650/38.4) = 43 < (980/\sqrt{\sigma_y}) = 56.6$

Hence, the member would satisfy plastic section and lateral support requirements in Canada.

Now, according to the procedure developed in this paper, a member having:

- $\alpha_f = b/2t\sqrt{(300/\sigma_y)} = 165/2\times10\sqrt{(300/300)} = 8.25$ (in Mpa) or $b/2t\sqrt{(43.5/\sigma_y)} = 8.25$ (in ksi)
- $\alpha_w = h/w\sqrt{(300/\sigma_y)} = 290/6\sqrt{(300/300)} = 48.33$ (in Mpa) or $h/w\sqrt{(43.5/\sigma_y)} = 48.33$ (in ksi)
- $\alpha_l = L/2b\sqrt{(300/\sigma_y)} = 1{,}650/38.4\sqrt{(300/300)} = 10$ (in Mpa) or $L/2b\sqrt{(43.5/\sigma_y)} = 10$ (in ksi)

would develop, according to Figure 5, an available rotation capacity of about five. This is less than the minimum required rotation capacity of six required for fully ductile structures.

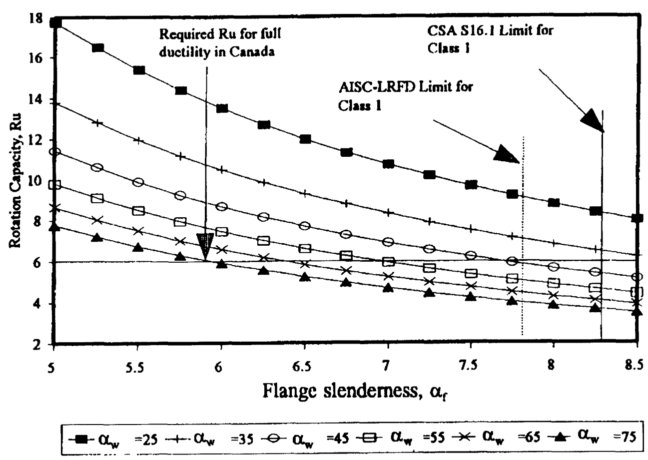


*Fig. 5. Ultimate rotation capacity, $R_u$, for $L/B\xi$ = 10.*

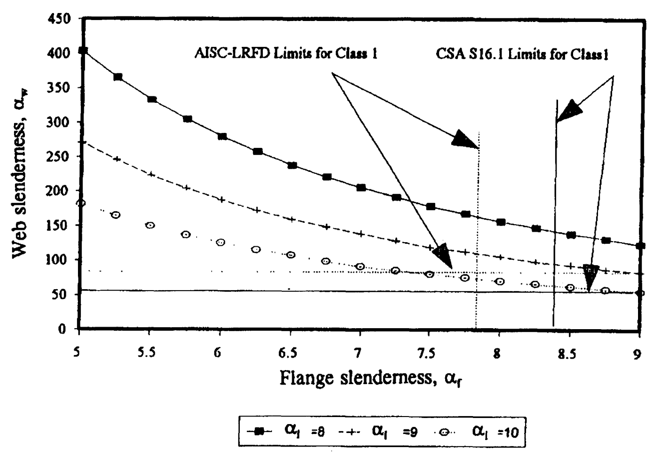


*Fig. 7. Interaction diagram for $R_u$ = 5.*

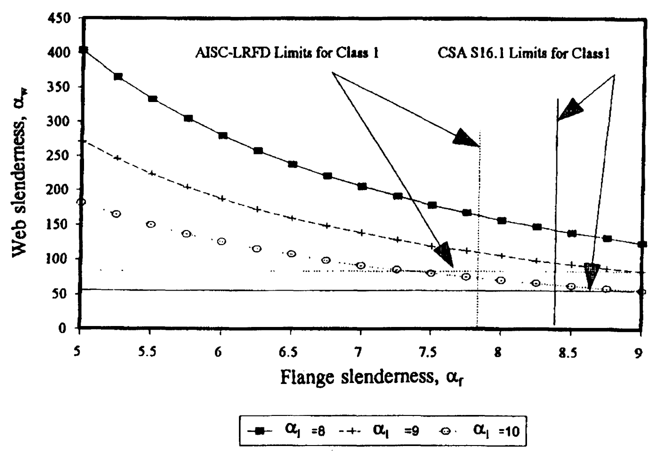


*Fig. 6. Interaction diagram for $R_u$ = 4.*

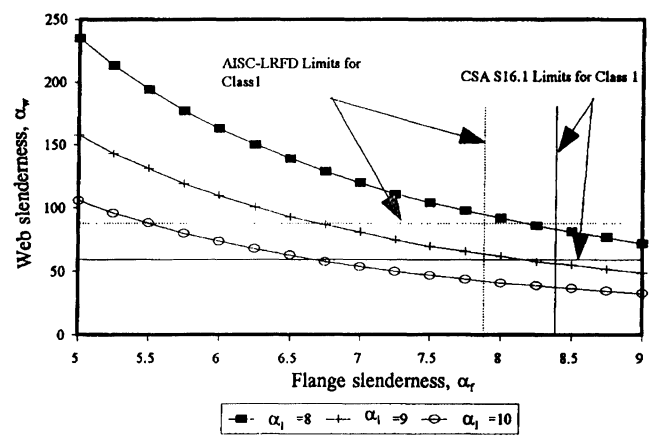


*Fig. 8. Interaction diagram for $R_u$ = 6.*

2. Now, according to AISC local buckling design rules and S.I. units, a W310×45 section has:

- $(b/2t) = (166/11) = 7.55 < (137/\sqrt{\sigma_y}) = 7.91$
- $(h/w) = (291/7) = 42 < (1{,}365/\sqrt{\sigma_y}) = 78.8$
- $(L/r_y) = (1{,}650/38.8) = 43 < (9{,}473/\sigma_y) + 25 = 56.57$

Hence, the member would satisfy plastic section and lateral support requirements in the United States. Now, according to the procedure developed in this paper, a member having:

- $\alpha_f = 7.55$
- $\alpha_w = 42.0$
- $\alpha_l = 10.0$

would develop, according to Figure 5, an available rotation capacity of about six which is less than the minimum required rotation capacity of seven required for fully ductile structures. Therefore, in both cases, the member would not be able to redistribute moments in accord with plastic design unless more stringent buckling limits were applied or interaction curves such as those in Figure 5 used.

We have seen, in Figures 3 to 5, how the local buckling rules are highly influenced by the flange lateral slenderness. Further it was demonstrated through a simple example that the current local buckling limits may sometimes lead to an unconservative design. It is also seen how one can use the interaction curves to select a member with specified slenderness limits in order to achieve a chosen required rotation capacity.

## CONCLUSIONS

From the study at hand, one can see that the member rotation capacity demands imposed by the seismic design codes are not always consistent with the classification of sections. It may be that member capacity values are more than ample in resisting the factored loads, however, the purpose of this study was not to examine that issue. It was simply to accept code requirements and identify weaknesses in classification for design necessitating full ductility. Designing a structure, by employing the current local buckling rules as specified by most codes of practice, may lead to an unsafe selection of members not able to redistribute moments in accord with plastic design. An empirical expression, relating the plate slenderness with the flange lateral slenderness, is used to develop ultimate rotation capacity and interaction curves for selection of local buckling limits needed to achieve a specified required rotation capacity.

An upper limit $\alpha_l$ of nine is proposed as the critical limit for the flange lateral slenderness to be used with the Canadian buckling rules for beam sections required to develop a minimum rotation capacity of six. Similarly, an $\alpha_l$ of eight is suggested for beams required to achieve a minimum rotation capacity of seven in the United States. These values appear to permit design for full ductility. For greater values of the lateral slenderness, interaction curves are proposed to be used to reach a specified rotation capacity. The procedure can easily be applied to any standards of practice by simply replacing the buckling limits by the specified ones.

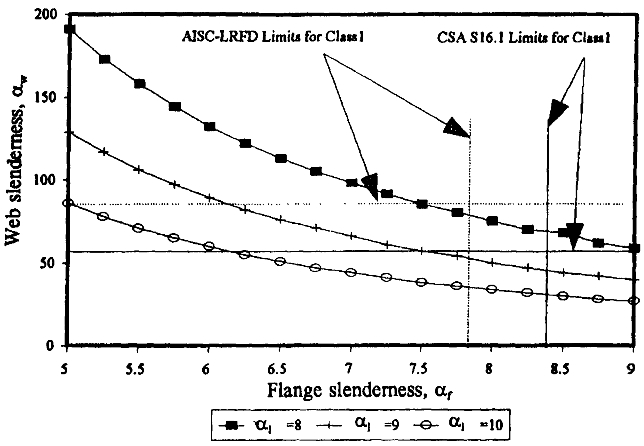


*Fig. 9. Interaction diagram for $R_u$ = 7.*

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## APPENDIX: NOTATION

$b$ = flange width
$h$ = web depth
$t$ = flange thickness
$L$ = length between point of inflexion and point of maximum moment
$M_p$ = plastic moment
$R_u$ = rotation capacity reached before $M_p$ is exhausted
$R_m$ = material capacity
$R_s$ = system capacity
$r_y$ = radius of gyration about a section's weak axis
$w$ = web thickness
$\alpha_f$ = normalized flange slenderness
$\alpha_l$ = normalized lateral slenderness
$\alpha_w$ = normalized web slenderness
$\lambda_e$ = effective lateral slenderness ratio
$\theta$ = end rotation
$\theta_p$ = plastic rotation
$\sigma_o$ = nominal yield stress of a prescribed value
$\sigma_y$ = actual yield stress
$\mu_s$ = system ductility