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A limit states approach to flexural ductility of plain steel beams used in plastic design

Synopsis

In order to validate the principal assumptions inherent to plastic design of frames and continuous beams, it is necessary to predict the available rotation capacity at positions in the structure where plastic hinges are likely to form at collapse. The required rotation capacity is a function of the geometry of the structure and the loading and may be quantified within certain broad parameters. In order to predict the rotation capacity available at critical points in the structure, it is necessary to satisfy limit states criteria on a consistent basis. This paper deals with a proposed method of predicting the available rotation capacity and classifying members rather than sections. The available rotation capacity is predicted on the basis of two principal parameters and an interaction equation. The proposed prediction model is compared with experimental results.

Samevatting

In die plastiese analise van rame en deurlopende balke word daar aangeneem dat voldoende rotasiekapasiteit beskikbaar is by posisies in die struktuur waar plastiese skarniere waarskynlik by swigting sal vorm. Om die geldigheid van hierdie aanname te bevestig, is dit nodig om die rotasiekapasiteit by kritiese posisies te kan voorspel. Die hoeveelheid rotasiekapasiteit benodig is afhanklik van die geometrie van die struktuur en die belastings wat daarop inwerk, en kan gekwantifiseer word binne sekere parameters. Die voorspelling van die beskikbare rotasiekapasiteit veroorsaak dat sekere grenslimietstaatkriteria konsekwent bevredig moet word. In hierdie verhandeling word daar 'n metode voorgestel waarmee die beskikbare rotasiekapasiteit voorspel kan word. Dit word duidelik dat struktuurelemente, eerder as snitte, gegroepeer kan word in terme van rotasiekapasiteit. Die voorgestelde wiskundige model voorspel die rotasiekapasiteit aan die hand van twee hoofparameters en 'n interaksievergelyking. Die voorgestelde model word vergelyk met eksperimentele resultate.

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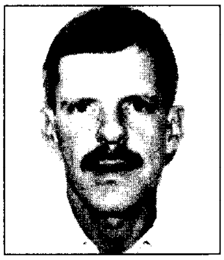


Introduction

Methods of plastic analysis and design require sufficient rotation capacity in members in order to achieve the large strains associated with the formation of plastic hinges at critical locations in the structure. A method is proposed in this paper whereby members rather than sections may be optimized in order to achieve the required flexural ductility to validate the fundamental assumptions pertaining to plastic design.

Researchers have identified the two principal parameters that control plastic collapse of structures as being the rotation required in the structural members at critical points and the rotation available in structural members.

In this context the available rotation is defined as the range of inelastic rotation over which the bending moment exceeds the fully plastic moment of the section.

Consistent with limit states philosophy, the required rotation may be classified as an action effect, while the available rotation may be defined as a resistance effect.

Kemp (Kemp and Dekker, 1991) has proposed a limit states criterion whereby the available rotation (resistance effect) should exceed the required rotation (action effect) in regions of the member or at end-connections where inelastic behaviour occurs. In line with current limit states philosophy, this criterion is formulated as follows:

$$\frac{\theta_a}{\gamma_{mr}} \geq \theta_r \qquad (1)$$

where θ_a is the total inelastic rotation at a plastic hinge or end connection consistent with the design resistance moment exceeding the fully plastic bending moment M_p, θ_r is the rotation required at critical points in a structure to achieve a fully plastic distribution of bending moments, and γ_{mr} is a partial material factor to account for uncertainties in both the required and the available rotations at critical points in the structure.

A value of γ_{mr} of between two and three has been proposed by Kemp (Kemp and Dekker, 1991) for Eqn 1, the lower value for relatively ductile modes of failure such as commonly observed in local and lateral buckling, with the higher value applying to sudden or brittle fractures.

The total inelastic rotation at a plastic hinge may be provided by plasticity within the member or by elastic and inelastic deformation of the connection itself. This paper will focus on rotation within the member only,

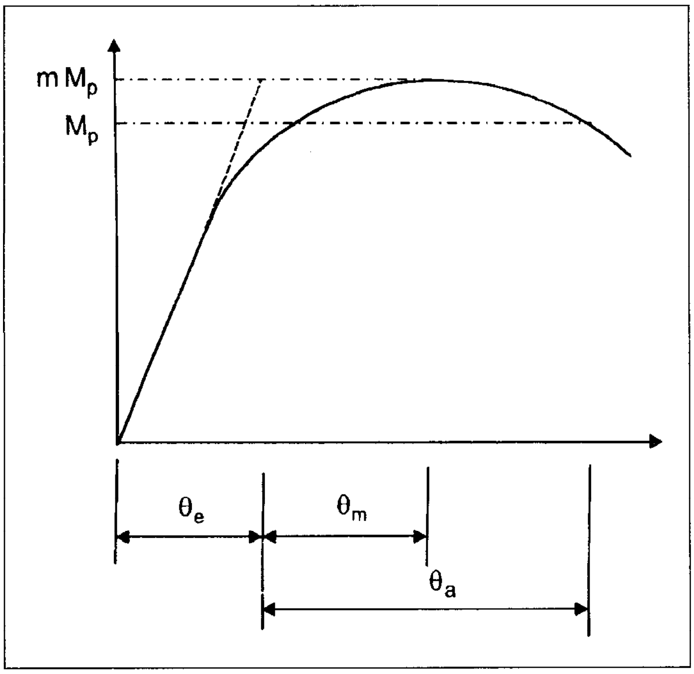


Fig 1: Typical moment-rotation curve for a member containing a plastic hinge

while recognizing that deformation in the connection itself serves to alleviate the required rotation.

In order to provide a comparison between levels of flexural ductility in different members, the rotation at a plastic hinge may be expressed in a non-dimensional form as the ratio of the total rotation over which the bending moment exceeds the fully plastic moment of resistance, to the elastic rotation. This ratio is defined as the rotation capacity of a member as shown in Fig 1.

In a non-dimensional form, the rotation capacity of the member at a plastic hinge may be defined in terms of the rotation capacity at maximum moment, referred to as:

$$r_m = \frac{\theta_m}{\theta_e}$$

or in terms of the total available rotation capacity:

$$r_a = \frac{\theta_a}{\theta_e}$$

It is generally recognized that the rotation capacity at maximum moment can be measured with greater accuracy in experiments on plain steel beams than the available rotation capacity. Despite the complications involved in predicting or even measuring the falling branch of a moment-rotation curve, experimental evidence would indicate that the available rotation capacity would be approximately equal to double the rotation capacity at maximum moment.

To satisfy the limit states criterion formulated in Eqn 1, it is necessary to quantify the required rotation or rotation capacity, which is a function of the geometry of the structural frame and the arrangement of the loading, as well as the rotation available in the structural members at critical positions in the structure. A simplified and consistent method of quantifying the available rotation capacity is presented in this paper and accepted criteria of required rotation capacity are used as a basis for classifying members rather than sections in terms of being suitable for plastic design or not.

Required rotation capacity in structures designed by the plastic method

The required hinge rotation at critical points of any framed structure or continuous beam as determined by a rigorous elasto-plastic analysis may be reduced if the following factors are taken into account:

- Elastic and inelastic rotation in the connections
- The degree of displacement control present in statically indeterminate structures as opposed to the total load-control prevalent in statically determinate structures
- Strain-hardening, which can increase the flexural resistance at critical points and reduce the hinge-capacity required

Kemp (Kemp and Dekker, 1991) has proposed that a value of rotation capacity measured at maximum moment of 1,0 would be consistent with members that are required to achieve at least their fully plastic moment of resistance, commonly referred to as class 2 sections in design codes.

Members that are required to achieve the necessary redistribution of bending moments to form a fully plastic collapse mechanism, commonly referred to as class 1 sections in design codes, have been shown to require rotation capacities at maximum moment in the region of 3,0, corresponding to a maximum redistribution of bending moments of some 35 per cent based on an elastic analysis of a continuous member with uniform section properties. The proposed requirement of a rotation capacity at maximum moment of 3,0 would therefore be consistent with an available rotation capacity of 6,0.

Strain-weakening mechanisms in flexural elements

The moment-rotation relationship shown in Fig 1 is typical of tests on plain steel beams. The maximum moment in the member is greater than the fully plastic bending moment owing to the effects of strain-hardening. The amount of inelastic rotation that will occur after achieving the fully plastic bending moment is limited by some form of instability in the compression zone of the section.

Measurement of rotation after the maximum moment is achieved is complicated by the energy stored in testing apparatus and is considered to be somewhat similar to the actual behaviour in statically-indeterminate structures where some form of displacement control would normally exist, in contrast to the total load control prevalent in tests on statically-determinate beams.

The amount of inelastic rotation that will occur at a plastic hinge is directly related to the strain-hardening properties of structural steel and the ratio of the maximum moment to the fully plastic moment, defined in this paper as:

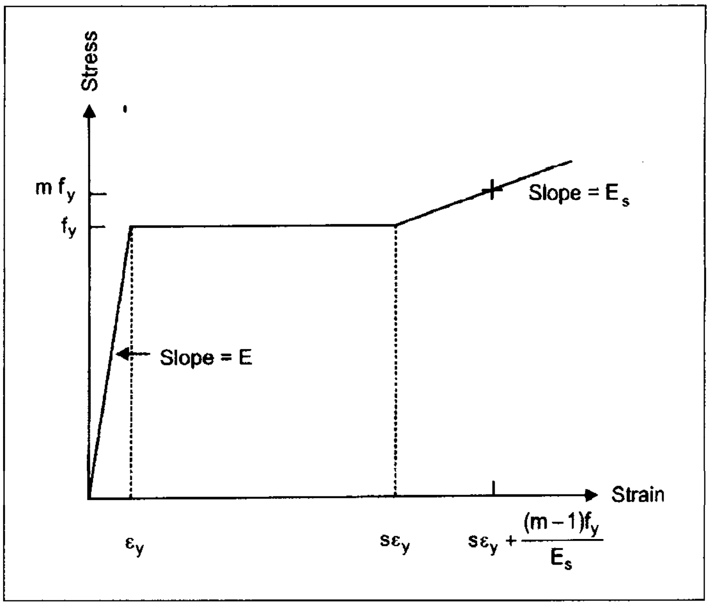
$$m = \frac{M_m}{M_p}$$

where:

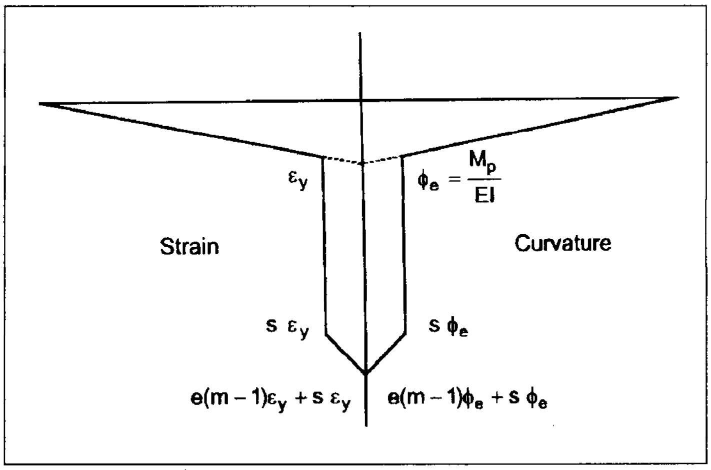
M_m = maximum moment at the plastic hinge prior to the onset of strain-weakening behaviour

M_p = fully plastic moment or design moment

Consider the idealized stress-strain curve for structural steel shown in Fig 2.



Idealized stress-strain curve for structural steel



Idealized moment strain relationship for a steel beam containing a plastic hinge

Fig 2: Idealized stress-strain curve and moment-strain relationship for a structural steel beam

Based upon the idealized tri-linear stress-strain curve for structural steel and a linear moment-strain and moment-curvature relationship using Lay's (Lay and Galambos, 1965) discontinuous yield theory, as shown in Fig 2, Kemp (1985) has shown that the inelastic rotation capacity at maximum moment (r_m) may be expressed as:

$$r_m = \frac{m-1}{m}\,[2s - 1 + (m-1)] \qquad (2)$$

where s = ratio of strain at the onset of strain-hardening to the yield strain and e = ratio of strain-hardening modulus to the elastic modulus.

There is a common tendency in current design codes to classify sections

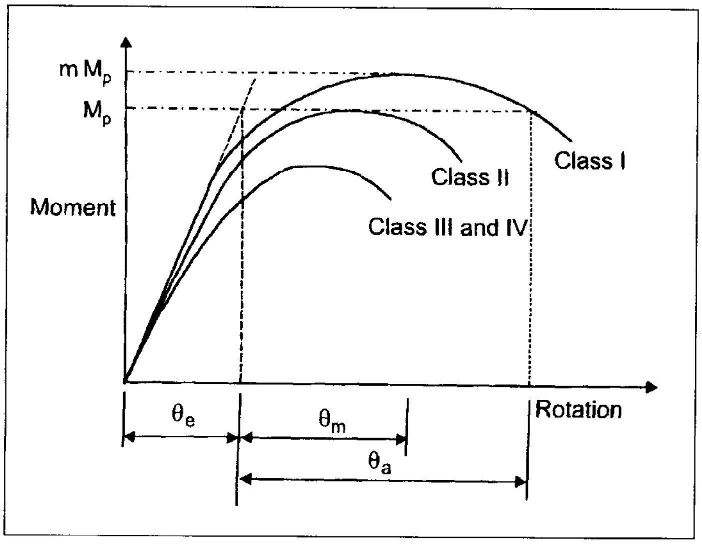


Fig 3: Classification of sections in terms of rotation capacity

in terms of rotation capacities that can be achieved subject to certain restrictions on lateral slenderness. This classification of sections is best illustrated by comparing the theoretical moment-rotation curves applicable to each classification as shown in Fig 3.

The current basis of classification of sections in terms of rotation capacity only considers the geometry and slenderness of the compression zones of the cross-section and is therefore intended to prevent the following forms of buckling:

- Local buckling of the compression flange of the section
- Local buckling of the portion of the web in compression

Two possible buckling modes of the compression flange have been identified in experiments on plain steel beams, viz a symmetrical buckling mode where little or no rotation of the flange occurs about the web and an anti-symmetrical mode related to sympathetic buckling of the portion of the web in compression. These modes of buckling are shown in Fig 4.

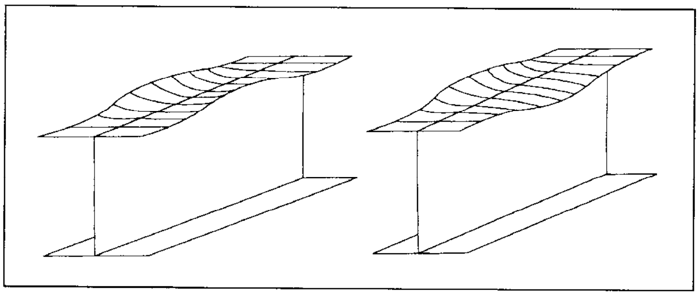


Fig 4: Local buckling of compression flange and web

Despite this tendency towards classification in terms of section geometry only, lateral buckling has been identified as a principal cause of strain-weakening behaviour.

Code requirements have traditionally treated the three forms of buckling in isolation, thereby implying no interaction between the different modes. Yura (Yura et al, 1963), Hancock (1978) and Kemp (1985) have discussed the interactive nature of local and lateral buckling and have proposed models reflecting this behaviour. Dekker (1989) showed that, provided lateral buckling of the member is effectively prevented, very high values of rotation capacity can be achieved after local buckling. In these tests, T-sections were used to prevent local web buckling interacting with local flange buckling. Dekker (1998) has proposed that local buckling of the compression flange should be considered in the anti-symmetrical mode only and should allow for local buckling of the web by considering the slenderness of the portion of the web in compression.

Prediction model for local buckling

By considering Eqn 1 it becomes clear that in order to quantify rotation capacity for a member containing plastic hinges, it is necessary to predict the ratio of maximum moment to fully plastic moment, ie the extent to which strain hardening can occur in the member at the plastic hinge. A suitable prediction model should consider the influence of both local and lateral buckling and should correctly reflect the interaction between the buckling modes. In this paper the prediction models for local and lateral buckling are first considered independently and a suitable interaction equation is then proposed.

The prediction model for local buckling used in this paper is based on the proposals of Kemp (1985), with certain simplifications. The proposed local flange/web buckling model considers supercritical and subcritical conditions in the web and allows for an independent assessment of the local flange/web buckling resistance as a function of the slenderness of the flange outstand, the slenderness of the portion of the web in compression and the material properties of the section.

The resistance of the flange and web to local buckling is expressed in a dimensionless form as a proportion of the fully plastic moment and defined as m_f. Dekker (1998) has previously proposed an interactive flange/web buckling model based on a modification of the proposals of Kemp (1985) where the resistance to local flange buckling may be expressed as a proportion of the fully plastic moment of resistance of the section. The proposed model considers local buckling of the portion of the web in compression. By assuming typical values as suggested by Haaijer (1969) of the material properties of steel in the plastic region, elastic modulus E = 200 GPa, plastic modulus E_s = 4 GPa, and the plastic strain-hardening ratio s = 10, the dimensionless local buckling parameter m_f may be expressed as given by Eqn 3:

$$m_f = 270\left(\frac{235}{f_y}\right)\left[1 - 0{,}017\left(\frac{h_{fc}}{t_w}\right)\right]\left(\frac{t_f}{b}\right)^2 + 0{,}6 \qquad (3)$$

where:

h_{fc} = depth of web in compression

t_f = thickness of the compression flange

b = width of compression flange

t_w = thickness of the web

Eqn 3 may be represented graphically as a family of curves whereby the local buckling parameter is expressed as a function of the flange slenderness ratio, for a given value of web slenderness.

Fig 5 clearly demonstrates the principle of compensation of one parameter in terms of another. A value of the local buckling parameter equal to one that is consistent with the minimum value required for this class of section may be achieved with various combinations of web and flange slenderness.

A value of web slenderness h_{fc}/t_w=35 is consistent with many current code requirements for class 1 sections and is shown to require a value of flange slenderness of b/t_f <16. The model also demonstrates that a value of the flange slenderness parameter b/t_f of as high as 20 can still achieve a value of the local buckling parameter in excess of 1,0 provided that the value of the web buckling parameter is reduced to 25 or lower.

Prediction model for lateral buckling

The complexities associated with quantifying the lateral buckling re-

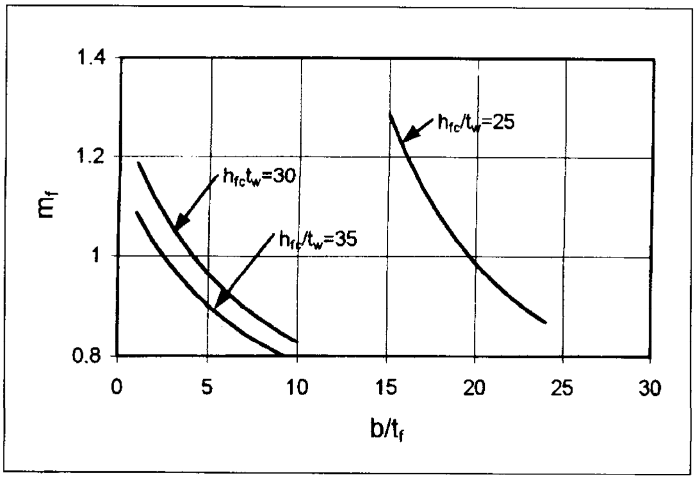


Fig 5: Variation of local buckling parameter as a function of flange and web slenderness

sistance of a member containing a plastic hinge have been discussed by Kemp (1984). The prediction model used in this paper (Dekker, 1998) is based on the principle of modal extrapolation and considers non-uniform material properties, a linear variation in the stress in the compression flange caused by moment gradient and linear moment-strain and moment-curvature relations allowing for a physical length of the plastic hinge.

The resistance to lateral buckling is expressed in a form similar to that used for local flange buckling and defined as m_L.The lateral buckling of beams containing plastic hinges has been considered by modelling the compression flange of the beam, containing both yielded and elastic portions, as a strut subjected to a varying axial load, consistent with the variation in bending moment as shown in Fig 6.

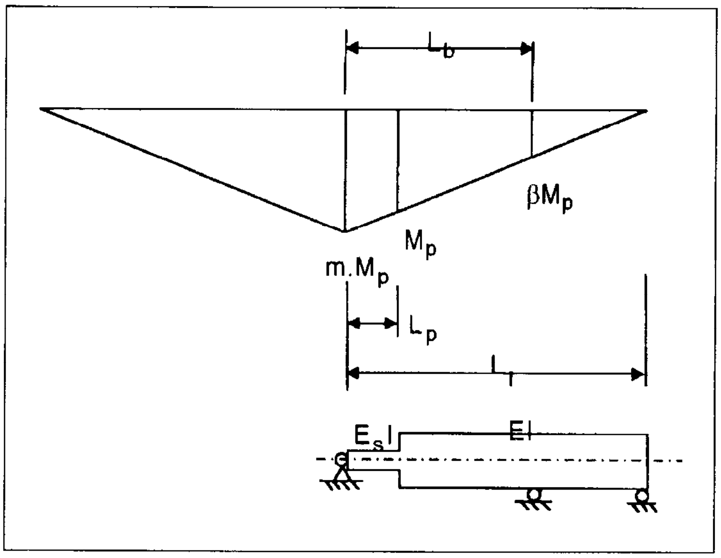


Fig 6: Idealized model for lateral buckling

Lateral restraints are assumed at the position of maximum bending moment and at the ends of the beam. In this manner an effective length factor for the plastic zone is derived, which, when used in conjunction with the classic beam-buckling equation, may be used to quantify the lateral buckling resistance of beams containing plastic hinges. The theoretical solutions previously proposed by Dekker (1998) are tedious to apply, but may be represented in a simple form as given by Eqn 4.

$$m_L = \frac{1}{1{,}029 - 41\left(\dfrac{L_i\sqrt{f_y}}{r_y}\right)^{-0{,}829}} \qquad (4)$$

where L_i = distance between lateral restraints or half span of simply supported beam containing a plastic hinge.

Eqn 4 is valid for the most common case where the bending moment varies from zero to $m_L M_p$ over a laterally unsupported distance equal to L_i. This condition is also consistent with most tests on steel beams. The variation of the lateral buckling parameter as a function of the lateral slenderness ratio is shown in Fig 7.

Eqn 4 is illustrated graphically in Fig 6 and shows the variation of the

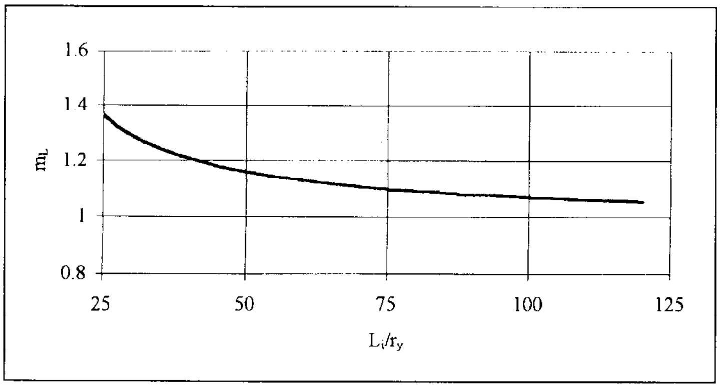


Fig 7: Variation of lateral buckling parameter as a function of lateral slenderness

lateral buckling parameter m_L as a function of the lateral slenderness ratio L_i/r_y. It is of interest to note that values of the lateral buckling parameter m_L greater than 1,0 are obtainable at lateral slenderness ratios exceeding 140.

Interactive lateral torsional/local flange-web buckling

The effective or interactive buckling moment may be expressed as a proportion of the fully plastic moment of resistance, therefore $m_i M_p$ is used to define the value of the interactive buckling resistance of the member. It is important to recognize the relative importance of local and lateral buckling. Two separate cases are considered:

Case 1: Lateral torsional buckling precedes local flange/web buckling ($m_L < m_f$) – where m_L is given by Eqn 4 and m_f is given by Eqn 3

By considering lateral torsional buckling as the principal cause of strain-weakening behaviour, it is proposed that for the case under consideration the overall member resistance should be based on the lateral buckling resistance, therefore:

$$m_i M_p = m_L M_p \text{ or } m_i = m_L$$

Case 2: Local flange/web buckling precedes lateral torsional buckling ($m_f < m_L$)

In this case the lateral buckling resistance may be reduced by local flange buckling to a value defined as $m_i M_p$ where m_i is the reduced or interactive moment multiplier. The value of m_i will be upper-bound by the lateral buckling resistance and lower-bound by the local flange/web buckling resistance.

Local web buckling will limit the stress due to bending at the flange tip to a value determined by the local flange buckling resistance, $m_f f_y$. In the absence of local flange buckling the stress in the flange would be governed by lateral buckling and it is therefore assumed that this condition would also apply at the centre line of the beam.

A condition of stress in the compression flange is therefore assumed where the stress at the flange tip is limited by local buckling to a value equal to the local flange/web buckling resistance, $m_f f_y$, and the stress at the centre line of the flange is governed by lateral buckling to a value equal to $m_L f_y$. By combining the two conditions, the stress distribution for the case where local flange buckling precedes local flange/web buckling is shown in Fig 8.

The variation in stress between the flange tip and the centre line of the beam is dependent on the extent to which the local flange buckle has developed and therefore on the ratio of m_f/m_L.

The simplest variation in stress between the two limiting values would be linear and this form is considered to be consistent with the required degree of accuracy.

It is then possible to express the interactive resistance to lateral buckling preceded by local flange/web buckling as follows:

$$m_i = \frac{m_L + m_f}{2} \qquad (5)$$

The influence of moment gradient

The derivation of Eqn 5 assumes that the point of maximum lateral curvature coincides with the point of maximum amplitude of the local flange buckle. For partially yielded beams under moment gradient, the point of maximum lateral curvature will occur close to the transition point between the elastic and plastic length, while the maximum deflection of the local buckle will occur in the middle of the plastic length.

Under conditions of moment gradient, Eqn 5 reflects the influence of the local buckle on a beam with an unbraced length approximately half the actual length or a lateral buckling resistance approximately four times the actual resistance, as shown in Fig 9. A revised form of Eqn 5 is therefore proposed as Eqn 6, in which the ratio of $m_f{:}m_L$ is adjusted in the proportion of 1:4, thereby providing a better reflection of the influence of local buckling on the interactive buckling resistance.

$$m_i = \frac{m_f + 4m_L}{5} \qquad (6)$$

The proposals discussed in this paper are illustrated in Fig 10.

The solid line in Fig 10 represents a moment-rotation curve for a beam in which local buckling did not occur before or after lateral buckling while the moment at the plastic hinge exceeded M_p. The dotted line A shows a moment-rotation curve based upon Eqn 6 for a beam having the same

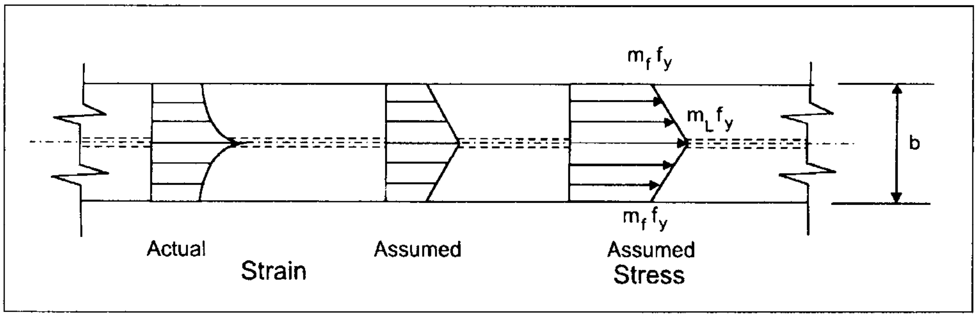


Fig 8: Assumed stress condition in the compression flange at the onset of lateral torsional buckling preceded by local flange/web buckling

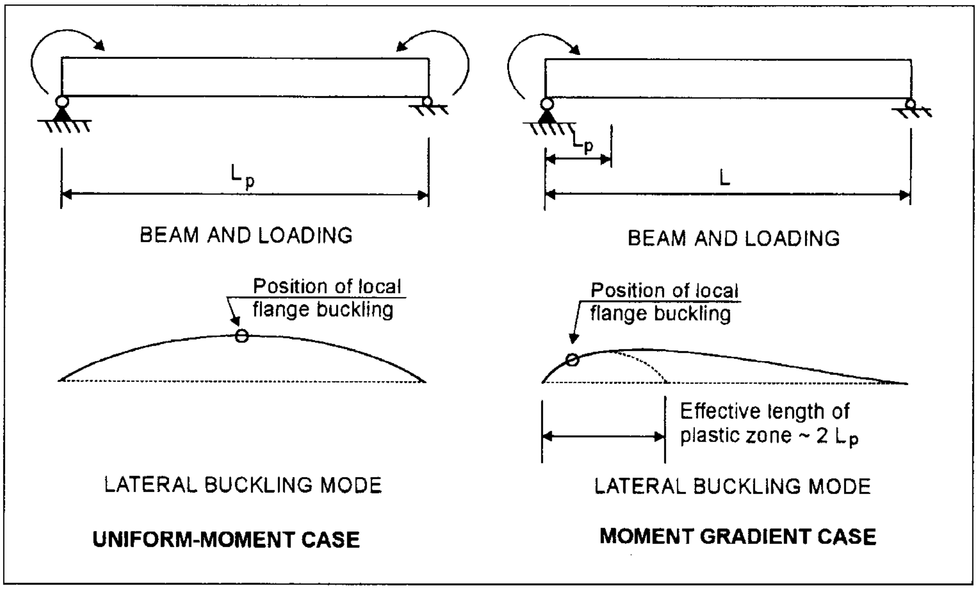


Fig 9: Influence of moment gradient on the position of the local buckle relative to the lateral buckle

level of lateral buckling resistance but in which local flange buckling occurred at a moment equal to $m_f M_p$, where $m_f M_p < m_L M_p$.

This situation reflects a reduction in the available rotation capacity as well as the rotation capacity at maximum moment. The dotted line B shows the moment-rotation curve for a beam in which local flange buckling occurred after lateral buckling and hence the available rotation capacity may be expected to be more than in the previous case. No attempt has been made to quantify the influence of local flange buckling on the falling branch of the moment-rotation curve, owing to the difficulties associated with consistent and accurate measurements in this region, as previously discussed.

Application to limit states criteria for rotation capacity

Combining Eqns 3, 4, 6 and 2 will allow the user to classify members in terms of flexural ductility, bearing in mind that the upper bound limit on flexural resistance will always be governed by lateral stability. Consistent with the proposals of Kemp (1985), a partial material factor of $\gamma_{mr} = 2$ in Eqn 1 would require a value of available rotation capacity equal to six for

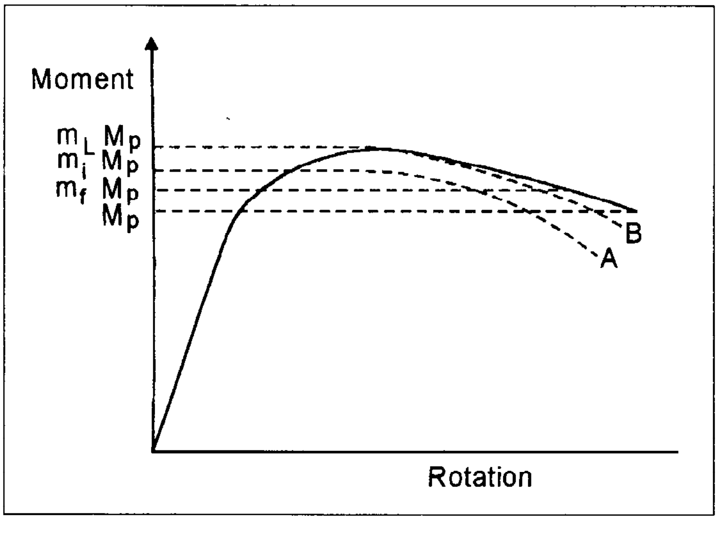


Fig 10: Proposed interactive lateral/local buckling model

a class 1 member.

Experimental evidence indicates that the available rotation capacity would at least equal the rotation capacity at maximum moment. For a class 1 member a rotation capacity at maximum moment of 3,0 would therefore be considered adequate. For a given value of yield stress, the required rotation capacity may be achieved by limiting the lateral slenderness ratio for a given section. In a similar fashion, the lateral slenderness ratio may be extended for a section where the section properties are subcritical ($m_f > 1$). This concept may be best illustrated by considering tests on steel beams having various combinations of super-optimim and sub-optimum levels of local and lateral buckling resistance.

Tests on plain steel beams for the purpose of measuring rotation capacities are commonly performed on simply supported beams subjected to a central point load.

Such conditions satisfy the assumptions made in the derivation of the lateral buckling model, Eqn 5, as well as the interaction equation as given by Eqn 8. Results of 44 tests of this type have previously been used to demonstrate the proposed model. In this paper certain of these results have been selected to demonstrate the concept of super- and sub-optimum levels of local and lateral buckling resistance, combined to allow classification as class 1 members where a value of rotation capacity at maximum moment of 3,0 is used as a limiting criterion. Comparison of the proposed interactive model with selected laboratory specimens where various combinations of local and lateral slenderness parameters are shown to satisfy different levels of member classification are shown in the accompanying table.

Test parameters such as flange, web and lateral slenderness have all been normalized for the yield stress to a value of $f_y = 235$ MPa (European standard). References to test specimens have been designated as La (Lukey and Adams, 1969), K (Kemp, 1985) or RK (Kuhlmann, 1989).

It is evident that regardless of the code of practice that is adopted, sufficient rotation capacity may be obtained from class 2 sections to allow classification as class 1 members, provided that the lateral slenderness ratio is limited enough to provide an acceptable value of the lateral buckling parameter, m_L.

It is also clear that the lateral slenderness governs the upper-bound bound flexural resistance as reflected by the lateral buckling parameter. Consider as an example the test specimen designated K14. Despite the relatively high value of the local buckling parameter of $m_f = 1{,}5$, consist-

Classification of members by rotation capacity

Test ref	b/t_f	h_k/t_w	L/r_y	m_f *Eqn 3*	m_L *Eqn 4*	m_l *Eqn 6*	m_l *Obs*	r_m *Eqn 2*	r_m *Obs*	*Class member-section*
Laa1	20,7	15,3	38,6	1,06	1,21	1,18	1,39	4,3	5,1	1-2
Lab2	17,6	25,4	44,1	1,09	1,18	1,16	1,17	3,8	4,2	1-1
Lad1	15,4	23,9	77,1	1,27	1,1	1,1	1,09	2,2	1,5	2-1
K14	12	30,6	53,6	1,50	1,15	1,15	1,10	3,5	3,6	1-1
K4	15,1	20,5	60,4	1,37	1,13	1,13	1,17	2,9	3,3	2-1
K7	19,7	17,3	30,1	1,09	1,28	1,24	1,24	6,1	7,4	1-2
RK8	22,1	28,4	32,7	0,89	1,26	1,19	1,33	8,8	11,5	1-2
RK24	22,2	27,6	35,1	0,89	1,24	1,17	1,26	8,0	5,2	1-2/3

ent with the stocky flange geometry $b/t_f = 12$, the flexural resistance was clearly limited by the value of the lateral buckling parameter $m_L = m_l$ = 1,15 (predicted) and m_l = 1,10 (observed). This particular example clearly illustrates the fundamental importance of lateral slenderness in the context of rotation capacity.

Conclusion

A method of classifying members in terms of rotation capacity allows for a consistent limit states approach to rotation capacity. The proposed method considers local and lateral buckling in terms of dimensionless buckling parameters that are then combined by means of an interaction equation. This approach has previously been shown to provide acceptable accuracy when compared with experimental work by others. The method as presented here in its simplified form may be easily applied in a design context.

A section commonly classified as a class 2 section on the basis of flange slenderness may, for example be re-classified as a class 1 member by an appropriate reduction in the lateral slenderness ratio.

Classification of members rather than sections provides a more consistent approach to limit states requirements related to rotation capacity.

Current code provisions relating to class 1 sections would appear to be sub-optimum, requiring sub-critical values of lateral slenderness in order to achieve the necessary rotation capacity.

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Discussion on papers

Written discussion on the technical papers in this issue of the *Journal* will be accepted until 31 August 1999. This, together with the authors' replies, will be published in the First Quarter 2000 issue of the *Journal*, or the issue thereafter. For the convenience of overseas contributors only, the closing date for discussion will be extended to 30 September 1999. Discussion must be sent to the Directorate of SAICE.

Such written discussion must be submitted in duplicate, should be in the first person present tense and should be typed in double spacing. It should be as short as possible and should not normally exceed 600 words in length. It should also conform to the requirements laid down in the 'Notes on the preparation of papers' as published on the inside back cover of this issue of the *Journal*.

Whenever reference is made to the above papers this publication should be referred to as the *Journal of the South African Institution of Civil Engineering* and the volume and date given thus: **J SA Inst Civ Eng, Vol 41, No 3, Third Quarter 1999.**