Section C

BEAM DESIGN

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This paper is the result of deliberations of the Society's Study Group for the Seismic Design of STEEL STRUCTURES.

1. CONTENTS

2. DEFINITIONS

Flange Slenderness. For a flange with one free longitudinal edge, such as that on an I or [section, the flange slenderness is the ratio of the flange outstand b_1 to the average flange thickness T. For a flange supported on two longitudinal edges, such as that in a box section, the flange slenderness is the ratio of the flange width b_2 to the flange thickness T.

Lateral Buckling is the instability phenomenon, which may lead to collapse, because of a dramatic increase in the lateral deflections and twist of the member. Lateral buckling may also be called flexural-torsional buckling.

Lateral Slenderness is the ratio of the effective length of the member to the radius of gyration about the minor principal axis of the section.

Local Buckling is the instability phenomenon which involves a change of shape of the cross-section of a steel member over a relatively small part of the member.

Web Slenderness is the ratio of the web depth d_1 to the web thickness t.

3. INTRODUCTION

Frames are frequently designed by the strong column - weak beam philosophy. With this method and in other situations the beams are assumed to be able to absorb energy during a strong-motion earthquake.

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In order to achieve energy absorption and dissipation at selected points in a structure, adequate plastic rotation capacity of the steel members at these points, must be achieved.

It is assumed that in addition to the requirements detailed later, the beam will be designed in accordance with NZS 4203 (1) and NZS 3404 (2).

Lateral and local buckling either separately or jointly, can have a marked effect on the ability of a steel member to achieve adequate plastic rotation capacity for moment redistribution and for the maintenance of strength. They can lead to undesirable, premature or brittle-type failures of the structure under seismic loading.

It should be appreciated that slender plates, unlike columns, exhibit considerable post-buckling strength which has been relied upon in steel design for many years. The flanges and webs of beams tend to behave as plate elements, and even when distorted after local buckling are capable of safely carrying very high stresses, provided the longitudinal boundaries are stable. However, the local distortion of the section may also increase the lateral deformations elsewhere and thereby reduce the overall stability of the member. There is also a tendency for the magnitude of the buckles to grow with each successive cycle.

For a member, restrained adequately by lateral bracing and with small width-to-thickness ratios provided by the flanges and webs, severe buckling can be sufficiently delayed to achieve satisfactory behaviour under seismic loading. However NZS 4203 Clause C3.2 suggests some loss of strength of a primary member of up to 30 percent is permissible after eight reversals (4 cycles) provided the overall building ductility requirements are met.

The main purpose of this paper is to discuss local and lateral buckling of low-carbon structural steel members consisting of plate elements with relatively thin walls or outstands such as UC, UB, hollow and channel sections.

Interaction occurs between local buckling of the flange and web, and lateral buckling. So far design codes have not attempted to account for this complex interaction, even for monotonic loading. The rules giving limits to the slenderness of flanges and web and to the spacing of lateral bracing do not consider interaction. This is simpler for design, but less accurate in some situations.

Where slendernesses are discussed in the text below, they are used as defined above, unless otherwise noted.

4. BACKGROUND

For the plastic design of structures under static loads, adequate rotation capacity is required under proportional monotonic loading. NZS 3404:1977 adopted the plastic design rules of AS 1250 (3) for the slenderness of members for aseismic design. Buen (4) while acknowledging the uncertainties involved, confirmed this approach until additional information was available.

Popov and Pinkney (5) after testing a series of 8 WF 20 I-sections, which have a flange slenderness of 6.64 and a web slenderness of 29.8, observed that local flange buckling did not precipitate an immediate loss of load-carrying capacity as the buckles appeared and disappeared cyclically until failure, under equal amplitude loading.

Popov (6) suggested it would be prudent to be more conservative in assigning flange and web slendernesses for cyclic loading compared to those for monotonic loading. He noted that under cyclic loading, lateral deflections tend to magnify, and it is imperative to prevent this by bracing. Whereas the top flanges are often held by the floor system, the bottom flanges are not laterally braced and can be in compression over a considerable portion of the span. Observations in the laboratory (7) demonstrated that deep beams with unbraced bottom flanges are particularly vulnerable to this phenomenon.

Carpenter and Lu (8) investigated the experimental behaviour of five full-sized single-bay mild steel (F_y = 248 MPa) frames subjected to constant gravity loads and to cycles of reversed and repeated lateral displacements. The frames represented parts of an eight-storey frame subjected to simulated earthquake loading. One problem studied was the effect of local flange buckling. Two essentially similar frames were tested but one frame had an I-beam flange slenderness of 5.51 and the other 9.58. The other geometric properties were similar for both beams, the web slendernesses being 31.9 and 30.4 respectively and the lateral slendernesses 32.6 and 25.7 respectively. They concluded that although local flange buckling occurred at an early stage, in the frame with the more slender flange outstand, the shape and reproducibility of the hysteresis loops were not changed significantly. The value of 9.58 would exceed the proposed limit of $120/\sqrt{F_y}$ and yet satisfactory behaviour was obtained. This is probably because the web slenderness is quite low (few UB s are that low) and hence web buckling did not occur, so that the section was still stable and could provide post-flange-buckling strength. In addition the lateral slenderness was low at 25.7.

Takanashi et al. (9) reported tests which showed that repeated cyclic result loading tests on H-shaped sections gave quite different results from those obtained with monotonically increasing loads. The rotation capacity for cyclic tests was considerably smaller than for monotonic tests.

Vann et al. (10) tested wide-flange cantilever beams and concluded that for cyclic loading of large amplitude, with slenderness ratios close to the limits prescribed for ordinary plastic design, unstable hysteresis loops may be obtained. The deterioration found was severe only when local flange buckling combined with local web buckling or with lateral buckling. Web buckling had a particularly adverse effect on load capacity, whereas lateral-torsional deformation tended to produce a loss of stiffness. The addition of an axial load tended to induce more rapid deterioration of strength.

They found that a W8 x 13 I-section, with a flange slenderness of 7.42, slightly less than $120/\sqrt{F_y}$, and with modest values of web slenderness of 32.6 and lateral slenderness of 29.6, suffered local flange buckling beginning in the second half cycle, and web buckling beginning in the fifth half-cycle. After 11 full cycles the strength had degraded to 72 percent of the plastic moment. This behaviour would satisfy current New Zealand requirements for ductile behaviour.

For a specimen with a W6 x 16 I-section and a lateral slenderness of 59.6 (which just satisfies the plastic design rule of $960\alpha r_y/\sqrt{F_y}$) and modest values of web slenderness of 20.9 and flange slenderness of 4.67 it was found that the strength had degraded to less than half the plastic moment, after 20 cycles to a ductility of 11.1. A W8 x 15 I-section suffered significant strength degradation when the unbraced flange was in compression, and was tested with a brace to one flange at mid-span and a span to radius of gyration ratio of 65.8, a web slenderness of 30.6 and a flange slenderness of 6.00. It may comply with the current requirements of NZS 4203 for ductility. These two tests indicate that the limit $720r_y/\sqrt{F_y}$ could be exceeded if the web and flange slenderness ratios are modest.

Mitani et al. (11) investigated the influence of local buckling on the behaviour of H-shaped steel beam-columns under alternating bending moment and constant axial load. Test results show the deterioration of strength because of local buckling, particularly where web buckling occurred after flange buckling. For specimens whose yield stress varied between 270-332 MPa, they found for a flange slenderness B/(2T) of 8 and zero axial load and web slendernesses D/t tested up to 54.4, that the strength decreased gradually with each cycle. A similar result was obtained with B/(2T) of 8 and axial load ratios P/P_Y of 0.3 and 0.6 provided the web slenderness D/t was less than 40. If B/(2T) were between 11 and 16 and P/P_Y either 0 or 0.3, with D/t less than 40, then a significant decrease in strength was obtained in the first few cycles (presumably because of flange buckling) then a gradual decrease in strength with each cycle followed, (presumably the low web slenderness inhibited total loss of strength). Even with a squat flange with B/(2T) of 8 when D/t was greater than 40 and the axial load ratio P/P_Y was 0.3 a significant decrease in maximum carrying capacity was obtained with each cycle of loading. A similar drastic decrease was obtained when the flange slenderness was between 11 and 16 and the axial load ratio P/P_Y was 0.6.

Chopra and Newmark (12) suggested a flange outstand slenderness ratio of six would be required to develop a ductility factor of about six.

Mild steel portal frame specimens were tested by Matsui et al. (13) under constant vertical and either monotonic or alternating horizontal loads. A significant reduction of restoring force was noted as the values of flange and web slenderness were increased. There was a large reduction when web local buckling occurred after flange local buckling. This reduction became even more pronounced as the axial load was increased. They found that the deformation capacity, which was defined as the value of the displacement at the maximum horizontal load, does not depend so much on the web slenderness as on the flange slenderness. The energy absorption capacity was reduced as the flange and web slendernesses increased. It was also reduced by increases in the lateral slenderness, section depth to flange thickness ratio and axial load.

Figure 7 in their paper shows a rapid reduction in deformation capacity (defined above) as the flange slenderness increases. A flange slenderness b_1/T of 6.56 would give a ductility of 8 before strength degradation occurred, whereas a slenderness of 7.59 ($120/\sqrt{F_Y}$) would give a ductility of 7. (Higher ductilities can be obtained with these flange slendernesses, but only with strength degradation.) Figure 6.2 in their paper shows significant degradation with a flange slenderness B/(2T) of 8.3, a moderate web slenderness D/t of 31.3 and an axial load ratio P/P_Y of 0.3.

Thurston (14) expressed concern about the early occurrence of local flange buckling in some beam testing, and proposed that the allowable flange width to thickness ratio be reduced.

Johnstone and Walpole (15) tested some beam-column specimens where the beams were 310 UB 46, which have a flange slenderness ratio of 6.73 and a web slenderness of 42.1, and it was found that local and lateral buckling occurred, although there was no significant strength degradation because of these effects. The lateral slenderness was 54 compared to 45.5 for $720/\sqrt{F_Y}$. When the lateral slenderness was reduced to 25 the twisting of the beam was greatly reduced.

Whittaker and Walpole (16) tested two haunched beams built up from 410 UB 54 sections which have a flange slenderness of 7.82 compared to 7.59 for the rule $120/\sqrt{F_Y}$. The web slenderness was 50.2 and the lateral slenderness about 19. It was found with one beam that the strength of the beam was reduced to less than half of its maximum strength on the third loading cycle because of the occurrence of local buckling. This may have been unusually severe, because of the haunching giving a longer length of yielded flange and because of the presence of a web stiffener, which may have concentrated the local buckling in one area. A third haunched beam built without web stiffeners from 410 UB 60 sections with a flange slenderness of 6.65 and web slenderness of 48.8 was subjected to nine cycles at ductility four and two cycles at ductility six without suffering significant strength degradation. Local buckling was noticeable after six cycles and the magnitude of buckles increased with further cycles. Overall buckling was also evident although the slenderness ratio was only 29.

They also tested three plain beams using 310 UB 46 sections. The lateral slenderness was 29 and although local buckling was detected during the first cycle to ductility four, several cycles at high ductilities were applied, during which the buckles grew in size, but there was no significant strength degradation.

Redwood et al. (17) tested under cyclic loading two full-scale frame sub-assemblages consisting of steel column and composite I-beams joined with moment -resisting connections. The bottom flange was unbraced for 1.871 m and the top flange was bonded to the slab by Nelson studs. The Rule $720r_Y/\sqrt{F_Y}$ requires a brace at 1.25 m to both flanges for a bare steel frame. The flange slenderness was modest at 5.68 and the web slenderness was 51.0 (common for UB s) but this was reduced to about 40 by horizontal stiffeners placed near the bottom flange over the peak moment region. Lateral buckling occurred when the maximum deflections were about twice the measured yield deflection. This was associated with local flange buckling located near the end of the horizontal web stiffeners. These tests indicate interaction between lateral and local flange buckling. Further testing is required to repeat this test with slightly longer horizontal stiffeners. The flange slenderness of 5.68 was not high and the local flange buckling may have been influenced by the web slenderness of 51 where the stiffener terminated.

5. DESIGN RULES

5.1 Maximum Width to Thickness Ratios

The limits given for category three, where parts of members are required to perform elastically, are taken from the exisitng New Zealand codes, i.e. NZS 3404 incorporating AS 1250. The limits are taken from Clause 4.3 of AS 1250, except that the limit for webs in compression is similar to that given in earlier editions of AS 1250 and in BS 449 (18). The ratios allow the yield stress to be reached without prior elastic local buckling. The limits for flanges or webs in uniform compression allow for some residual weld stresses and imperfections. Lay (19) discusses the background to the limits for categories two and three. It will be nessessary to use lower flange slendernesses, than the maximum values permitted here, if it is desired to use the maximum allowable stress values of AS 1250. These allow the formation of one plastic hinge in the beam.

Category two limits for members required to provide limited ductility have been taken as the ratios required for plastic design under non-seismic loads from Clause 10.8 of AS 1250:1981. There it was expected that the strain at the end of the yield plateau would be about ten times the yield strain. It was thought that the strains to be reached, without premature local buckling by parts of members requiring limited ductility, would be of this order or less. No attempt has been made to remove anomalies or to round off these figures as they are a current code requirement, but they may be adjusted with the next code revision. The limit of $136/\sqrt{F_Y}$ seems low compared to the values required by the Canadian code (20) of 145 for plastic design and 170 to permit the attainment of the yield moment without redistribution of moment.

Category one limits are for parts of members required to provide full ductility in a major earthquake. Here the member may be required to suffer repeated straining; well into the strain hardening range, without premature local buckling. The limits have been chosen to be a little more restrictive than those commonly used for plastic design based on the material reviewed above in section 4. Little research has been reported in English and in many cases the research was not specifically directed at the local or lateral buckling problem under seismic loading.

The maximum width to thickness ratios for the various categories of required member performance are given in Table 1.

5.2 Spacing of Lateral Restraints

Members or parts of members in category three requiring elastic behaviour are required to conform to the allowable stress rules, Clauses 5.4 and 5.9 of AS 1250:1981. The stresses used should be those derived from the loading, multiplied by the strength method load factors specified in NZS 4203 divided by 0.6. Otherwise the stresses derived by the Alternative Method of NZS 4203 may be used. These actual stresses must be less than the allowable stresses, which depend on the effective length. This is a function of the spacing and stiffness of the lateral restraints.

Those parts of members required to provide limited ductility shall comply with Clause 10.9 of AS 1250 assuming that a plastic hinge is formed at a position of peak moment on the bending moment diagram derived from code loading. These limits are set out in Table 2 and have been derived assuming a ratio R, of the rotation at the plastic hinge point to the relative elastic rotation, of 10; giving a value of 1.0 for α using the AS 1250 formula $1.5/\sqrt{(1 + R/8)}$.

Category one includes the parts of members required to provide full ductility. There a rotation ratio R of 24 was assumed, giving a value of 0.75 for α. This gives the limits set out in Table 2 using the AS 1250 formula $640\alpha r_Y/\sqrt{F_Y}$.

As part of the design philosophy for structures with parts of members in categories one and two, it is required that plastic hinges form at selected points in the structure under a major earthquake. The bending moment diagram obtained from code loading must be adjusted so that the peak moments equal the magnitude of the plastic moment M_{PC} of the section provided. This adjusted diagram should be used to find the length of yielded flange, taken to be where $M > 0.85M_{PC}$. Table 2 gives the maximum spacing of the lateral restraints, within and adjacent to the plastic hinge, depending on the length of yielded flange and the category of ductility.

5.3 Design of Lateral Restraints

The strength and stiffness of lateral restraints shall comply with Cl. 3.3.4 and Cl. 5.9.2.1 of AS 1250-1981

6. FURTHER RESEARCH

Time history analyses of steel frames designed to N.Z. code rules are required to assess the ductility requirements in each of the three categories of design currently allowed, viz. full ductility, limited ductility, and elastic response.

From a knowledge of the ductility requirements it will be possible with further theoretical and experimental work to establish the accuracy of the rules for each of the three categories of design set out in this paper. There would appear to be anomolies within the current non-seismic rules and at present it is not certain how much more conservative the rules for aseismic design should be.

Table 1. Maximum width to thickness ratios

		Category 1 Parts of members requiring full ductility	Category 2 Parts of members requiring limited ductility	Category 3 Parts of members requiring elastic behaviour
Flanges and plates in compression with one unstiffened edge (eg I or [flanges)	$\frac{b_1\sqrt{F_Y}}{T}$	120	136	256
Flanges of welded box sections in compression	$\frac{b_2\sqrt{F_Y}}{T}$	500	512	560
Flanges of rectangular hollow sections	$\frac{b_2\sqrt{F_Y}}{T}$	350	420	635
Webs under flexural compression	$\frac{d_1\sqrt{F_Y}}{t}$	1000	1120	1340
Webs under uniform compression	$\frac{d_1\sqrt{F_Y}}{t}$	500	512	560

Table 2. Spacing of lateral restraints

	Category 1 Parts of members requiring full ductility		Category 2 Parts of members requiring limited ductility	
Flange length where $M > 0.85\ M_{PC}$	$> \frac{480\ r_Y}{\sqrt{F_Y}}$	$\leq \frac{480\ r_Y}{\sqrt{F_Y}}$	$> \frac{640\ r_Y}{\sqrt{F_Y}}$	$\leq \frac{640\ r_Y}{\sqrt{F_Y}}$
Spacing of braces within length where $M > 0.85\ M_{PC}$	$\leq \frac{480\ r_Y}{\sqrt{F_Y}}$	one brace required	$\leq \frac{640\ r_Y}{\sqrt{F_Y}}$	one brace required
Spacing to brace adjacent to length where $M > 0.85\ M_{PC}$	$\leq \frac{720\ r_Y}{\sqrt{F_Y}}$	$\leq \frac{720\ r_Y}{\sqrt{F_Y}}$	$\leq \frac{960\ r_Y}{\sqrt{F_Y}}$	$\leq \frac{960\ r_Y}{\sqrt{F_Y}}$

Note: Parts of members in category 3 should be braced in accordance with allowable stress rules.

The category of limited ductility was introduced for steel structures by the 1984 edition of NZS 4203 while this study group was in progress and designers have little experience with this type of design.

Research may establish formulae which consider the interaction between the various modes of buckling, enabling more accurate design rules to be established.

Testing may reveal the ability of longitudinal stiffeners to restrain local buckling in plastic hinge regions.

Further research is required to check whether the rules for the strength and stiffness of lateral restraint systems, based on research using monotonic loading are also satisfactory for seismic loading.

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8. NOTATION

B = the overall width of the flange of a section

b_1 = the outstand of a flange is the projection of the flange or plate beyond its line of support. For an I shaped section $b_1 = (B-t)/2$ as shown in Figure 1.

b_2 = the unsupported width of a flange or element is the distance between two adjacent faces of support or between two adjacent lines of connection to other elements of the member.

D = the overall depth of a section, measured parallel to the web

d_1 = the clear depth of a section between flanges, measured parallel to the web

F_Y = the specified yield stress of the steel in MPa

M = the bending moment

M_{PC} = the plastic moment of a section, being the strength in bending after yielding has just spread throughout the section, allowing for the presence of any axial load

P = the axial force in a member

P_Y = the area of a member times the specified yield stress

R = the ratio of rotation at the plastic hinge to the relative elastic rotation of the far ends of the beam segment containing the plastic hinge

r_Y = the radius of gyration of the cross -section of the member about the minor principal axis

T = the mean thickness of the flange

t = the thickness of the web of a section

UB = Universal Beam Section

UC = Universal Column Section

WF = Wide Flange Section

α = $1.5/\sqrt{(1+R/8)}$

[= a hot-rolled channel section

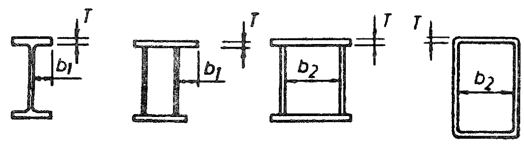

Figure 1.