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Autor: Bryan, Eric R. / Rhodes, James
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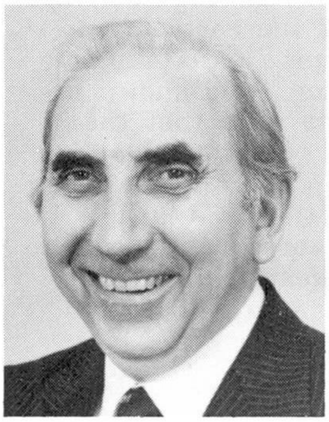
Cold-Formed Steel Structures and New British Code of Practice

Structures en acier formés à froid et nouveau règlement britannique

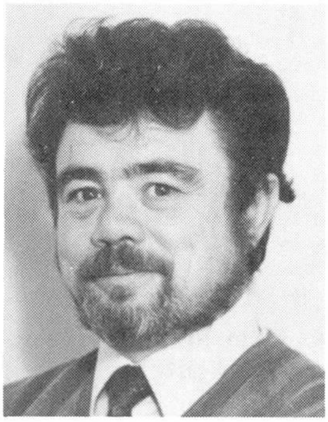
Kaltprofilkonstruktionen und neue englische «Code of Practice»

Eric R. BRYAN
Professor
Univ. of Salford
Salford, UK

James RHODES
Reader
Univ. of Strathclyde
Strathclyde, UK



Professor Bryan is Chairman of the British Standards Committee for the new code. He is the author and co-author of three books and many publications on stressed skin design, sheeting and cold formed steel structures, and acts as a consultant in these fields.



Dr. Rhodes is consultant drafter for the new code. He is editor of the international journal «Thin-Walled Structures» and has organised several major conferences on the subject. He is technical adviser to the Cold Rolled Sections Association and has published widely.

SUMMARY
Existing uses and new developments in cold-formed steel structures are reviewed. The part played by new codes of practice in stimulating and encouraging these developments is emphasized. In particular, some aspects of buckling effects in the new BS 5950 Part 5 are outlined and compared with the AISI Specification and with experiment.

RÉSUMÉ
Les utilisations actuelles et les nouveaux développements en matière de structures en acier formé à froid sont passés en revue. Le rôle joué par les nouveaux réglements stimulant et encourageant ces développements est mis en valeur. En particulier, des aspects d'effets du flambement et du voilement dans le nouveau réglement anglais «BS 5950 Part 5» sont esquissés et comparés avec les spécifications de l'AISI et avec l'expérimentation.

ZUSAMMENFASSUNG
Bereits existierende Anwendungsmethoden und Neuentwicklungen von Kaltprofilstahlkonstruktionen werden nochmals überprüft. Die neuen Normen geben Anregung und Ermunterung zu solch neuen Entwicklungen. Insbesondere werden einige Aspekte der Instabilitätserscheinungen (Knicken, Kippen, Beulen) im neuen BS 5950, Teil 5, aufgezeigt und mit den AISI-Vorschriften und mit Versuchen verglichen.

1. INTRODUCTION

1.1 Production of steel strip

Since 1975, one of the few areas of steel production which has seen an increase in Britain is the rolling of coated steel strip, as used in sheeting and galvanised cold formed members. In fact, over half the British steel output is now in the form of sheet steel in one form or another. This is a remarkable figure, and one which has crept up upon us. Although, in the structural field, we have given a lot of attention to the design of hot rolled sections and tubes, we have not given enough attention to the design of strip steel. We should remedy this situation and give strip products the design that their importance and tonnage warrants.

	%	
Main frames	27	)
Gable framing	3	) hot rolled steel 31%
Tubular bracing	1	)
Purlins and sheeting rails	6	)
Roof sheeting	23	) cold formed steel 42%
Side sheeting	13	)
Gutters and downpipes	3	
Insulation to roof and walls	24	
	100	

Fig.1. Costs of a 30 m span x 90 m long x 6 m high pitched roof building.

Of all structural steelwork produced in Britain, about 70% goes into industrial buildings, and a breakdown of the costs of a typical building - as supplied by a fabricator - is shown in Fig. 1. It is seen that the value of cold rolled steel in purlins, sheeting rails and sheeting is more than that of the hot rolled steel, though it is safe to assume that nearly all of the design effort went into the frames.

1.2 Cold formed sections

Cold formed steel sections are usually formed by a series of rolls which bends them, stage by stage, into the desired shape. Once the rolls have been set up, it is almost as easy to form an intricate shape as a simple one, so that the optimum shape for a structural member may be rolled very economically.

Cold formed steel members have several additional advantages over conventional steel sections: they are usually formed from galvanised strip so that they do not need painting; they are automatically cut to length and punched for holes; they often 'nest' so that they may be easily transported; and they are light and easy to erect. Moreover, the process of cold forming enhances the yield strength, so that by using extra bends in a section, not only is the stability of the various flat elements improved, but a higher design stress may also be used.

2. PRESENT USE OF COLD ROLLED SECTIONS

2.1 Purlins and sheeting rails

By far the largest structural use of cold rolled sections in Britain has been in purlins and sheeting rails. The Zed profile was one of the major shapes used but this has now largely been superseded by the Zeta profile (Fig. 2) to meet the requirements of smaller roof pitches and to use the benefits of extra bends. Because the section is reversible, offcuts can be used as sleeves. In the design process, the actual moment-rotation characteristics of the sleeves were taken into account so that the bending moment diagram approached the optimum condition shown in Fig.3a rather than the continuous or simply supported conditions shown in Figs. 3b and 3c.

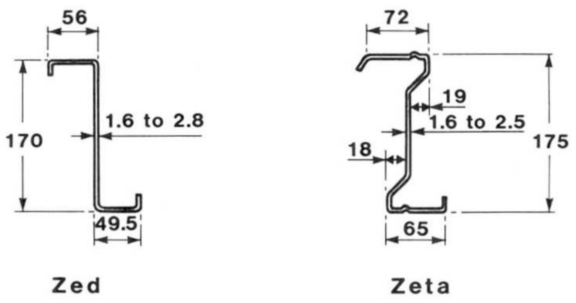


Fig.2. Zed and Zeta sections

In the design of sheeting rails, the same principles as for purlins apply but because of door and window openings there is less opportunity to have multi-span members. Also, provision must be made for carrying the vertical weight of the side cladding (Fig. 4).

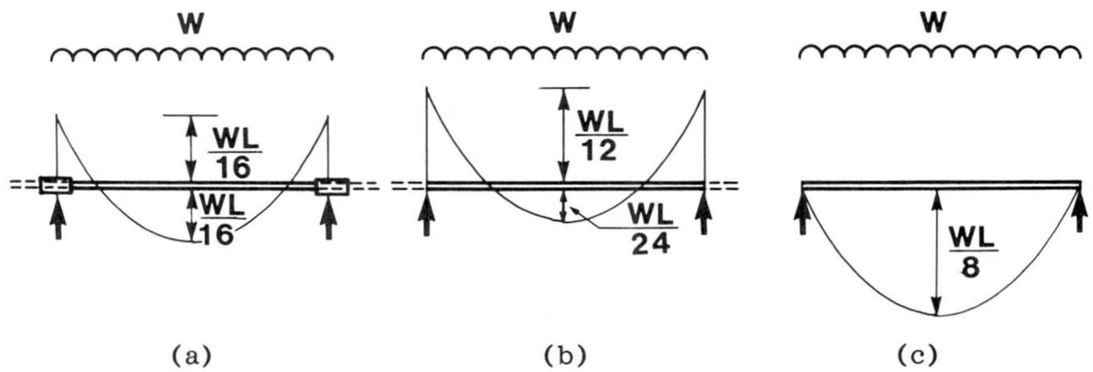


Fig. 3. Bending moment diagrams for purlins

Because of the substantial experience gained with cold rolled zed purlins and sheeting rails, simplified rules for their design and use have been drawn up, and these have been incorporated into the new British Code of Practice.

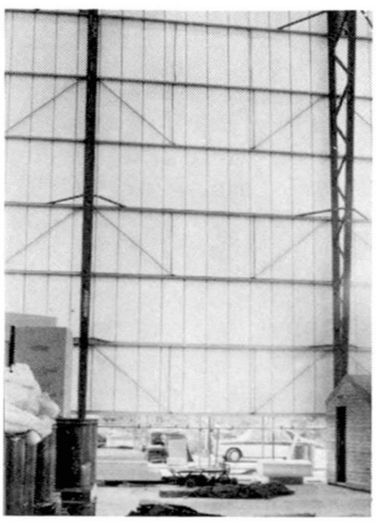


Fig.4. Bracing to sheeting rails

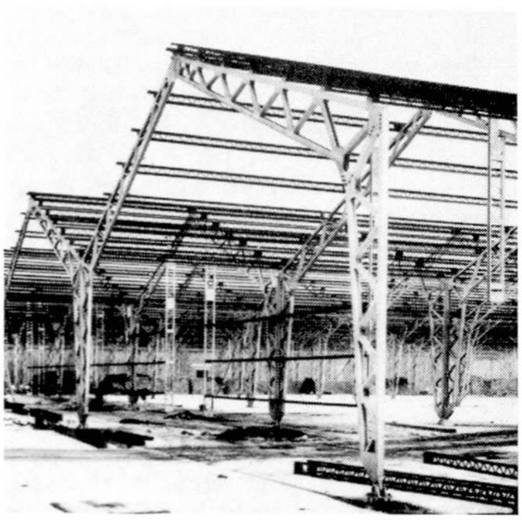


Fig.5. Latticed portal frames - c.1950

2.2 Other structural uses

Shortly after the war, due to shortage of hot rolled sections, latticed building frames were fabricated in Britain from cold rolled sections (Fig. 5). Although they were economical in terms of material they were expensive at that time in terms of fabrication and jointing, and so the method was not further exploited. Perhaps they were ahead of their time. However, lattice joists are still used.

In France, welded cold formed sections have been used in columns and in various novel forms of beams. These components have been used instead of hot rolled members in conventional school and office construction.

In the United States, cold formed steel has been used for many years as wall studs in domestic and office buildings, but the principle has not been widely used in Europe.

3. CODES OF PRACTICE

3.1 Limitation of existing codes

The uses of cold rolled sections just described have largely occurred without a great deal of dependence on codes of practice. For example, purlin and sheeting rail design in Britain, although soundly based theoretically, has been verified and refined by full scale testing. One of the reasons for this is that the present code of practice for cold formed steel sections - Addendum No. 1 to BS 449 : Part 2 : 1969 - is inadequate to deal with the problems involved because knowledge at the time it was written was limited. For instance, the only laterally unrestrained beam with which it deals is an I section, composed of two channel sections, connected back to back. Any other beam has to be dealt with on an experimental basis.

3.2 Development of new codes

In recent years, several new codes have either been published or are being written. The AISI Specification [1] was revised in 1980 and a further revision is under way. In Sweden, a comprehensive new Code [2] which deals with sheeting as well as sections was published in 1982. European Recommendations [3] have been drafted by the European Convention for Constructional Steelwork, first in Committee TC8 and subsequently in Committee TC7. The draft for comment has recently been circulated to the various National Associations.

Against this background, Britain, amongst other countries, has been preparing a new code of practice on the design of cold formed sections. It is part 5 of a comprehensive new code, BS 5950, on the "Structural use of steelwork in building" [4]. Like all parts of the code, it is written in limit state terms and deals with material and section properties, connections, simplified rules and testing, as well as the fundamental aspects of local buckling, stiffeners, beams and columns which are treated in the latter part of this paper.

A good design code must not just follow in the wake of existing practice: it must be adequate to allow for new developments. It must also be soundly based, both theoretically and experimentally, so that it has authority in the profession, and it must be practical. It is expected that the new code will satisfy these requirements and will act as a stimulus to develop more complex and efficient sections which may be used as main members in structures and not merely as secondary members. It should also signal the acceptance by the structural steelwork industry of cold rolled sections as an equal alternative to hot rolled sections.

4. SOME NEW DEVELOPMENTS IN PRACTICE

4.1 Swagebeam section

Recently a channel section has been introduced in Britain which has grooves or "swages" rolled into the web (Fig. 6a). These swages increase the stress at which web buckling occurs, so that they also permit a higher stress in the flanges. The b/t ratio for the flanges is sufficiently low for the flanges to be fully effective (or nearly so) and the double lip improves the section properties.

A major advantage of the swages is that they can lock on to similar swages on the jointing components (Fig.6b) to give a stronger and more rigid joint. This takes advantage of a feature of cold formed sections which is not available for conventional sections. It is a challenge to designers to find other ways of maximising such opportunities.

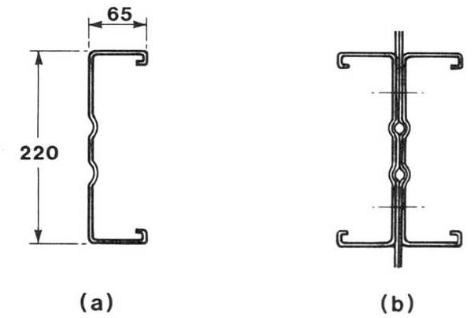


Fig.6. Swagebeam section and joint

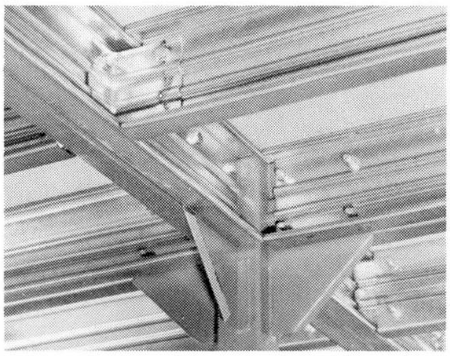


Fig. 7. Joints in storage platform

4.2 Swagebeam structures

Swagebeams have been used, both singly and in pairs, as the members for storage platforms. The rigidity at the joints (Fig. 7) has ensured a stable structure.

Another use has been in the development of a new pitched roof portal frame building system with spans between 9 and 15 metres, and with the swaged gusset plates formed from 2 - 3 mm plates with lips. The crucial factors in the design were the behaviour of the joints and the individual behaviour of the two swagebeam sections in the rafters, since they were connected together only at the purlin positions. The first point was resolved by bending tests on the apex and eaves joints which showed that the joints could be regarded as fully rigid and that they were stronger than the members which they connected. The second point - lateral buckling of the individual members between purlin points - was resolved by calculation to BS 5950 Part 5 and by bending tests.

After the design of the frames had been checked theoretically and experimentally, a pair of 12m span frames were test loaded to collapse at Salford (Fig.8). The actual behaviour confirmed the design method and showed that the frames were adequately strong.

A further recent use of the swagebeam section has been in the fabrication and testing of a 21.6m span latticed portal frame (Fig. 9). This design, using 1.5mm thick members, did not use the interlocking properties of the section, but it did show the importance of frame geometry and joint detailing. By re-arranging the internal bracing and re-designing the joints, the strength was nearly tripled.

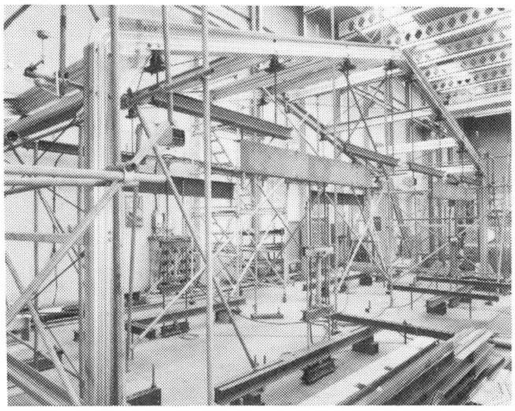


Fig. 8. Test on Swagebeam portal frames

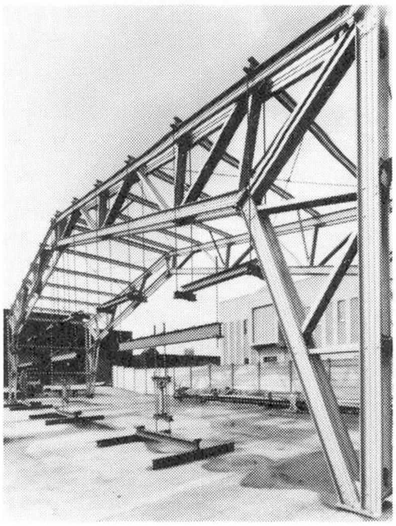


Fig. 9. Latticed test frame - 1985

5. SOME DESIGN DEVELOPMENTS AND COMPARISONS

5.1 Plastic design in cold formed sections

In the new British code, as in the AISI Specification, some utilization of plastic bending capacity is allowed in cold formed sections, but full plastic design including moment redistribution is not allowed except for compact sections. This is because the plastic rotation capacity of many sections is insufficient.

However, in many structures, as in continuous purlins, the maximum moments occur in the joints. If the joints are bolted, as is usually the case in cold formed members, there is a "plastic plateau" in the moment-rotation relationship which allows redistribution of moment. This aspect of design has not yet been investigated although considerable test data on joints have been accumulated by TNO in Holland [5] and by Corcoran [6] and Geha [7] at Salford.

5.2 Comparison of BS 5950 Part 5 and AISI Specification

In the process of writing and monitoring the new British code, comparisons were made with the 1980 AISI Specification. Allowance was made for the fact that the British code was in limit state terms while the American specification was in terms of permissible stress.

The comparisons of effective width of elements, moments of resistance of beams and ultimate axial compressive load for various sections are given in Fig.10. It is seen that the results of the two codes are very similar.

6. FEATURES OF THE NEW CODE

Some of the requirements of a new code have already been listed; in particular, Part 5 was required to give safe yet economical design rules while stimulating the growth of the product and encouraging new development. It also had to be compatible with BS 5950 Part 1, design in hot rolled steel.

In writing the code, the results of research carried out in other countries was used where appropriate, and the provisions of other codes - particularly the AISI Specification and the European Recommendations - were taken into account.

Nevertheless, the approach to some of the buckling effects in cold formed sections is different from that used elsewhere and so the rest of the paper is devoted to dealing with these aspects in greater depth.

Case	Section	Yield stress Y_s N/mm²	Required condition		BS 5950	AISI
Effective widths	Stiffened element b = 130 mm t = 2 mm	(1) 280	Effective widths mm	(1)	82.9	85.1
		(2) 350		(2)	74.1	77.8
	132, B, A, 66, t = 1.2	280		Element (A)	51.1	47.7
				Element (B)	59.0	44.0
Beams	229, t = 1.52, 102, 19, 76, 76, 19	345	Moment of resistance kNm		6.125	5.876
	50 50, t = 3, 180, 20	250			15.39	16.31
Columns	50, t = 2, 50, 100	280	Effective length 3.0 m	Ultimate compressive load kN	18.97	19·15
	75, 20, t = 2.7, 75, 20	228	Effective length 2.75 m		29.69	28.17

Fig. 10. Comparison of BS 5950 Part 5 and AISI Specification (by R. Plater)

7. TREATMENT OF LOCAL BUCKLING

In the new code [4], local buckling of both stiffened and unstiffened elements is treated using the effective width approach. The code stipulates that the effective portions of each element be positioned at the supported edges. In this way effective section properties for beams and columns may be derived, which result in changes in the neutral axis position and other data.

7.1 Stiffened elements

The effective width of a stiffened element under uniform compression is taken as:

$$\frac{b_{eff}}{b} = [1 + 14 (\sqrt{f_c/p_{cr}} - 0.35)^4]^{-1/5} \qquad (1)$$

where f_c is the applied stress and p_{cr} is the critical buckling stress given by $p_{cr} = 185000\ K\ (t/b)^2$ N/mm², K being the buckling coefficient.

The effective widths so given are slightly lower than those of the AISI Specification [1] if the minimum value of K is used.

7.2 Unstiffened elements

Theory shows, and experiments verify, that unstiffened elements under uniform compression have a greater effective width ratio at a given f_c/p_{cr} ratio than stiffened elements, although their much lower p_{cr} and the strong tendency to behave asymmetrically necessitate the use of care in dealing with these elements. In the new code, unstiffened effective widths are obtained on the basis of equation (1) and thereafter increased using the expression:

$$b_{eu} = 0.89\ b_{eff} + 0.11b \qquad (2)$$

The increased effective widths have been shown in reference [8] to be in good agreement with experimental results.

7.3 K factors

For stiffened elements, K may be taken as 4 and for unstiffened elements K may be taken as 0.425. The new code permits the use of higher K values if these can be justified, and gives details of higher values for a range of different sections and loading conditions.

Figure 11 illustrates the effective widths of various stiffened and unstiffened plate elements of sections. For unstiffened elements, increases in effective width of the order of 60% are obtained if the supported edges are highly restrained in comparison to the simply supported-free condition. In the case of stiffened elements the AISI effective width variation is shown for comparison purposes.

8. STIFFENERS

In the new code, the coverage of stiffeners is increased from that of its predecessor [9] to come more into line with the AISI Specification and the new European Recommendations [3].

8.1 Edge stiffeners

The minimum required second moment of area for an edge stiffener is given as:

$$I_{min} = \frac{b^3 t}{375} \qquad (3)$$

where I_{min} is determined with respect to the plate middle surface and b is the full plate width.

For b/t less than 60, a simple 90° lip of full width equal to b/5 satisfies this requirement, but for b/t greater than 60 a compound lip is required. Edge stiffened elements of b/t ratio greater than 90 are not allowed.

8.2 Intermediate stiffeners

The required rigidity for adequate intermediate stiffeners is obtained from:

$$I_{min} = 0.2 \left(\frac{w}{t}\right)^2 \frac{Y_s}{280} \qquad (4)$$

where w is the flat width of the sub-element between stiffeners, and Y_s is the material yield stress.

This equation is an approximation to the linear equations covering three different ranges as specified in the European Recommendations, and based on the work of Desmond, Pekoz and Winter [10]. It should be mentioned that in the new code, I_{min} is determined with respect to the plate middle surface, rather than with respect to the stiffener neutral axis as in other codes.

9. TREATMENT OF BEAMS

9.1 Laterally stable beams

The moment capacity of laterally stable beams is determined on the basis of a limiting stress, obtained by considering the web, which is then used to evaluate the effective width of the compression elements. Failure is assumed to occur when the limiting stress, p_c, is reached in compression. If the section geometry is such that tension yield occurs before p_c is attained, then this can be taken into account using an elasto-plastic stress distribution, and yield in tension is allowed to occur in any section.

The limiting stress, p_c, is given by

$$p_c = (1.13 - 0.0019 \frac{D}{t} \sqrt{\frac{Y_s}{280}}\ ) Y_s < Y_s \qquad (5)$$

where D is the web depth. For intermediately stiffened webs, modified values of p_c can be used.

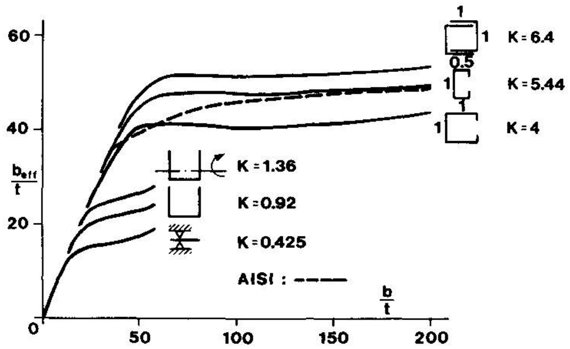


Fig. 11. Effective widths

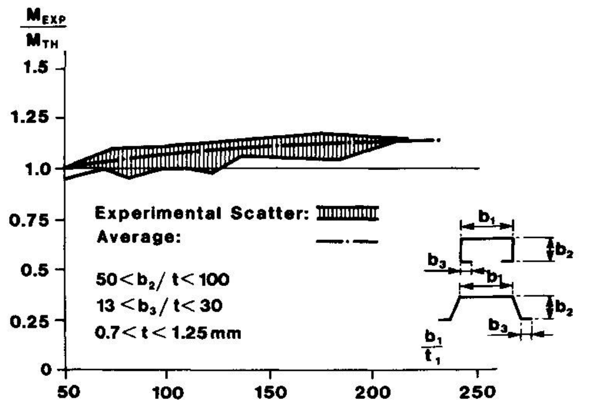


Fig. 12. Comparison of calculated and experimental failure moments

Figure 12 shows a comparison of the moment capacities obtained on the above basis for channel and trapezoidal section beams with the results of a series of 48 tests on these beams detailed in reference [11]. The comparisons indicate that the approach is accurate, although the results become slightly conservative at b/t ratios around 200.

In cases where the lips of these beams are small,tension yield occurs long before failure, and the benefits of permitting tension yield become apparent. This is illustrated in Fig.13 which compares the new code predictions for such sections with predictions based on tensile yield indicating failure. Test results show reasonable accuracy for the new code predictions, whereas those of the AISI Specification (and of the new code if tensile yield were taken as the failure criterion) are very conservative under these conditions. In plotting the AISI ultimate moments the design moments obtained using this specification were divided by 0.6 to remove the safety factor.

9.2 Post yield capacity of compact sections

The new code recognises the fact that if a section is sufficiently compact, post compression yield may be tolerated, and perhaps even plastic limit analysis, involving redistribution of moments, may be used. To take this into account, then subject to a number of conditions regarding section geometry, web dimensions, loading and material ductility, similar to those of the AISI Specification, sections with compression element b/t ratios less than certain limits are designated 'plastic cross sections'. The required limits are $25\sqrt{280/Y_s}$ for stiffened elements and $8\sqrt{280/Y_s}$ for unstiffened elements.

For plastic cross sections, full plasticity may be assumed and limit analysis involving moment redistribution may be used. For non plastic cross sections, design is based on elastic considerations. However, for compression element width to thickness ratios less than 1.6 times the plastic cross section limits, some limited increase in moment capacity due to post compression yield plasticity may be utilised.

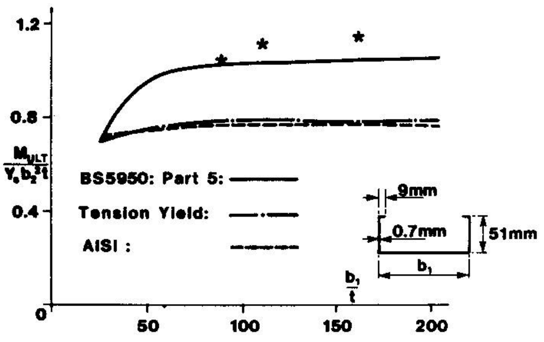


Fig.13. Failure moments for channels with small lips

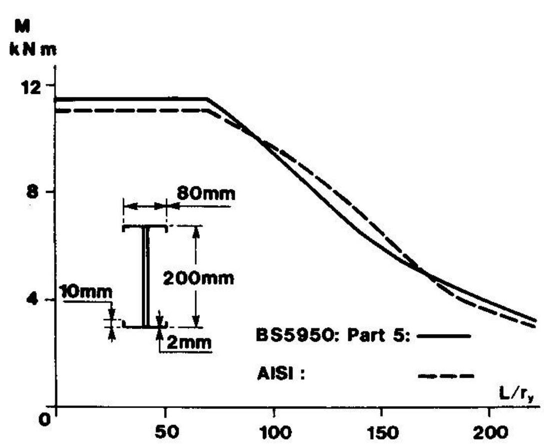


Fig. 14. Lateral buckling capacity of I section beam

9.3 Lateral buckling

In the new code, I, C, Z and T beams, which cover the bulk of lateral-buckling-prone beams used in practice, are considered. Determination of the lateral buckling capacity of these beams is made on the basis of a Perry-Robertson formula which uses the fully plastic moment M_p, and the elastic critical moment M_E. In determination of M_E, effective lengths are used. The effective length depends on the support conditions and varies from 0.7L to 1.1L, where L is the span between supports. For most circumstances a value of 0.9L is applicable if the support restrains twisting but permits rotation about major and minor axes.

The moment capacity of the section is taken as the lower of its lateral buckling capacity, determined as above, and its capacity evaluated as for a laterally stable beam.

Figure 14 shows a comparison of the working moments for an I section under pure bending with those of the AISI Specification. The working moments used in connection with the new code were the ultimate moments divided by 1.6, which is the maximum load factor used in this code. As can be seen, the working moments given by both codes are similar.

10. TREATMENT OF COLUMNS

10.1 Singly symmetric columns - beam-columns

In the new code all single symmetric columns are treated as beam-columns to take account of the well known 'wandering centroid' phenomenon due to local buckling. For a column under centroidal loading the effective area, and the Q factor is determined using the effective widths of individual elements. The effective neutral axis position is also determined and the magnitude and direction of the neutral axis movement, e_s, is calculated.

The ultimate load, Pc, neglecting neutral axis movement is evaluated on the basis of a Perry-Robertson curve which uses the squash load for the effective section and the Euler load for the full section. The effects of neutral axis movement are then taken into account using the expression

$$P_{ULT} = \frac{P_c}{\left(1 + \frac{P_c}{M_y} e_s\right)} \qquad (6)$$

where M_y is the moment capacity of the section, considering bending in the direction dictated by e_s.

Figure 15 shows the design loads evaluated for plain channel columns, as a ratio of the fully effective squash load, using a load factor of 1.6. The AISI design loads are shown for comparison, indicating significant differences in some cases. Comparisons of failure predictions using the above method for plain and lipped channels are shown to agree well with experimental results in reference [8].

10.2 Torsional-flexural buckling

The old U.K. specification [9] uses effective length multiplication factors, α , to deal with torsional-flexural buckling. These α factors allow the designer to treat torsional-flexural buckling using the tables and formulae set up for flexural buckling, but considering different effective column lengths. This method of approach is continued in the new code, because of its simplicity, although the more general treatment is also detailed.

Reference [9] differs from other specifications with regard to torsional-flexural buckling inasmuch as this code assumed warping restraint at the supports, whereas the AISI and other specifications assumed zero warping restraint. To obtain a balance between the over-optimistic approach of reference [9] and the over-pessimistic views of other codes, the new code assumes partial warping restraint.

Figure 16 shows the variation of α factors given in the new code for a typical cross section. Multiplying the effective column length by the relevant α factor permits evlauation of the torsional-flexural buckling capacity using the Perry-Robertson curve for flexural buckling.

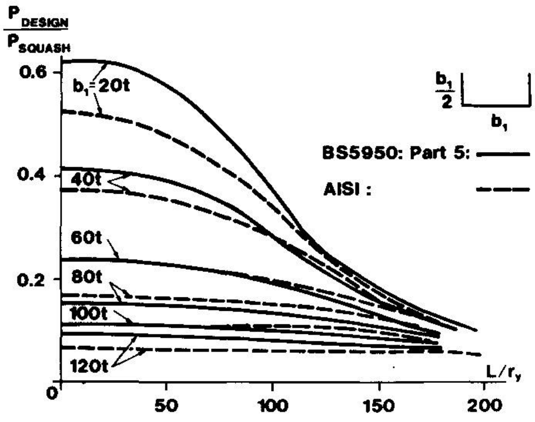


Fig. 15. Failure loads for plain channel columns

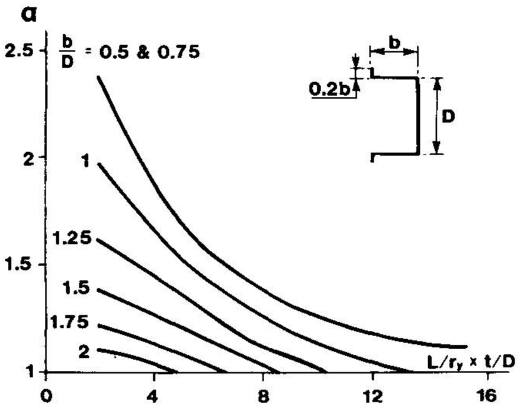


Fig. 16. α factors

11. CONCLUSIONS

This paper shows the importance of cold formed members in steel design. It reviews the existing structural uses and notes the new developments which are occurring in theory and practice. In particular it observes that new codes of practice should stimulate the product and encourage these developments.

In particular, some of the more prominent aspects of the treatment of buckling effects in the new British code BS 5950 Part 5 are outlined and comparisons with the AISI Specification and with experiment have been used to illustrate the predictions of the approaches used.

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