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Cold Formed Sections in Britain
Research and its Application to Design

Les profilés à froid en Grande-Bretagne. Recherches et étude des projets

Kaltgeformte Profile in Großbritannien. Forschung und ihre Anwendung im Entwurf

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A. Introduction

The first design specification for light-gauge steel sections was published by the American Iron and Steel Institute in 1946 [1]. This was based on extensive research work carried out under the direction of Professor George Winter of Cornell University.

In Britain research on cold formed sections was sponsored by the Cold Rolled Sections Association and has been carried out by the contributors of this paper and their associates at the University of Bristol, the Royal College of Science and Technology, Glasgow, the University of Cambridge and Industry.

The work was primarily concerned with the evaluation of the maximum strength of struts and beams and provides the rational semi-empirical basis of current industrial design methods. Research work in hand and planned for the future falls broadly into two categories.

a) Investigations concerned with further refinement and consolidation of the design methods at present in use, such as the theoretical evaluation of the local buckling collapse strength of longitudinally compressed plates by the application of the "large" deflection theory in conjunction with the deformation theory of plasticity; the torsional flexural instability of struts and lateral instability of beams (with special reference to the influence of lateral bracing) and the mechanical properties of cold formed sections in comparison with that of the parent strip.

b) The examination of certain detail aspects of the behaviour of structural components such as the characteristics of welded joints and connections in frame assemblies of cold formed sections; the strength under stable conditions of unsymmetrical section beams used as purlins etc.

The work completed to date generally confirms and substantiates the findings of Professor WINTER and his Associates [1].

In the translation of the research results into the form of a British design Specification certain differences from its American counterpart manifest themselves. These are primarily due to the fact the British Specification was drafted to fit into the framework of existing British Structural design practice, cold formed design being presented as an extension of, rather than a deviation from, its accustomed form.

B. Material Properties

Cold formed sections for general structural purposes are mostly manufactured of commercial quality low-carbon steels to British Standard 1449.

Strip specifically produced for structural purposes in accordance with British Standard 1449 EN 2 C/2 has guaranteed minimum yield and ultimate stresses of 14 and 24 tons per sq. in. respectively. In fact the strip material supplied by the continuous strip mills has mechanical properties well above the figures mentioned. Available higher grade steels of tensile yield stresses 22 and 32 tons per sq. in. may also be used profitably in certain applications.

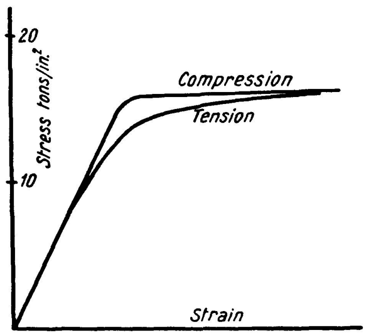

Fig. 1. Typical Stress-Strain Curves.

For most of the low carbon steels in common use Young's Modulus E has a mean value of about 12 000 tons per sq. in. The form of the tensile and the compressive stress-strain curves of the strip material may show some differences as indicated in Fig. 1. As in practice it is generally difficult to evaluate the behaviour of the strip material in compression, it is normally the tensile characteristics which are used as the basic mechanical properties in design.

The properties of the *finished section* may differ from that of the parent strip depending on the process of manufacture. The differences appear to be relatively more significant in cold *rolled* sections where considerable cold working is produced in the regions of the corners of the section than in sections manufactured by brake pressing. Depending on the shape of the section and the number of stages in the rolling process, the cold working may increase the average tensile yield stress over an appreciable area of the cross section in a significant manner [2]. At the present time no account is taken of the possible increased tensile strength which a strip may develop by virtue of being rolled into a section. It is hoped that in the future when more is known of the cold working processes involved, it may be possible to associate the permissible design stresses with the minimum mechanical properties of the material in its finished cold rolled section form and so enable the structural designer to take full advantage of the strength of his "raw material" which after all is not the parent strip but the finished section.

To cater for such potential developments and for the possible use of materials of a variety of yield stress values, particular attention has been paid to flexibility as regards this property in drafting the Specification. This has been achieved by defining design stresses throughout the Specification in terms of the yield stress of the material used.

C. Structural Behaviour and Design of Thin Walled Struts

Two types of thin walled strut failure due to instability are differentiated: *local buckling* in short and stocky struts, strength being limited by the stability of the thin walls of the section and *overall column buckling* in long and slender forms either by simple flexure or by a combination of flexure and torsion.

Initiation of failure due to instability always obtains at a stress which is below the yield or proof stress of the material while stable struts generally fail in direct compression at the appropriate limit stress of the material.

Maximum Strength of Compressed Plates and Sections in Local Buckling. The critical longitudinal compressive stress that *initiates* local *elastic* buckling in concentrically loaded flat plates or struts of thin walled section consisting of flat plate elements is given by:

$$\sigma_c = K \pi^2 E t^2 / 12 (1 - \nu^2) w^2, \tag{1}$$

where K is a constant depending for plates, primarily on the longitudinal edge support conditions and for sections on its dimensions; t and w is the thickness and width respectively of the plate or of a selected plate component (flange or web) of the cross-section and ν is Poisson's ratio. The values of the constant K have been obtained by a number of investigators [3, 4] for a variety of edge support conditions and for sections of different outline.

Tests have indicated [1, 5, 6] that the average compressive stress σ_m at which collapse of the plate or thin walled section occurs may be considerably greater than σ_c for large values of w/t and it has been shown [5] that the relevant parameters are (σ_m/σ_y) and (σ_c/σ_y) where σ_y is the yield or proof stress of the material.

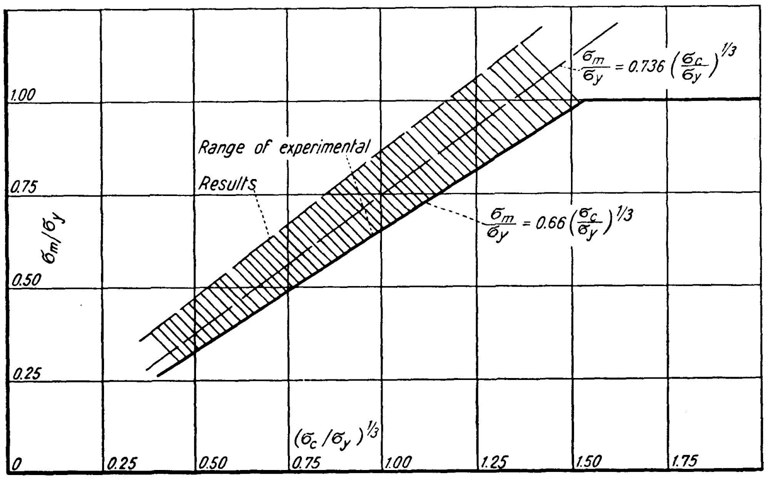

Fig. 2. Collapse Tests on Short Lengths of Concentrically Loaded Struts.

Fig. 2 shows the results of some 200 tests on short lengths of concentrically loaded steel struts. The tests cover a variety of section outlines (plain and lipped, single and double channels of various proportions) wall thicknesses of 0.128 to 0.036 in. and yield stress values form 10 to 32 tons per sq. in.

The relationship [5]

$$\sigma_m/\sigma_y = 0.736\,(\sigma_c/\sigma_y)^{1/3} \tag{2}$$

defines the average maximum strength with reasonable accuracy, while a conservative form of this relationship approximating to the lower scatter boundary can be written as

$$\sigma_m/\sigma_y = 0.66\,(\sigma_c/\sigma_y)^{1/3}. \tag{3}$$

In the region of high elastic buckling stresses, for which $\sigma_c > 3.50\,\sigma_y$ the maximum stress σ_m may be taken to be equal to the yield or proof stress σ_y of the material.

In applying this rationally conservative semi-empirical relationship to the assessment of the local buckling strength of structural sections the following considerations are relevant:

Thin walled sections in common structural use are formed into flat elements. A plain channel for example may be regarded as consisting of three flat plate elements — the web and two flanges. The longitudinal edge support given to these elements by the rest of the section is dependent on the proportions of the section outline but for practical purposes it is sufficient to distinguish only two types of edge support.

Adequate edge support equivalent to a simply supported edge is considered to be provided if the plate element under consideration is attached to an element of similar width or to a substantial reinforcing lip. In connection with the latter test results incorporated in Fig. 2 on "lipped" channels have shown that a lip of not less than $^1/_5$th of the plate width it supports may be regarded as adequate.

An edge, along which no support at all is provided or where the reinforcing lip is less than $^1/_5$th of the plate width, is considered free.

In consequence thin walled sections are regarded as built up of two kinds of plate elements only — "stiffened" if the element possesses adequate support as defined above along both its longitudinal edges and "unstiffened" if one of its edges is adequately supported and the other is free.

To obtain the failure stress σ_m of these two kinds of plate elements *when acting as components* of thin walled strut sections, σ_c computed from Eq. (1) corresponding to $K=4$ for stiffened and 0.456 for unstiffened elements is substituted in Eq. (3) accompanied by the appropriate value of σ_y.

In assessing the failure strength of a strut section in local buckling for design purposes, the collapse strength of the individual plate elements are added together. Thus the failure load P in local buckling becomes $P=\sum \sigma_m w t$ where σ_m, w and t are the collapse stress, width and thickness of the various plate elements composing the section.

These results have been presented in the Specification in non dimensional form obtained as follows:

The σ_m values for plate elements of mild steel ($E=12000$ tons per sq. in. $\nu=0.3$, $\sigma_y=14$ tons per sq. in.) are derived in the form of "local plate stress factors" $C_L=\sigma_m/14$ from Eq. (3) for

$$\begin{aligned} &\text{"stiffened" elements} \quad && C_L = 9.62\,(t/w)^{2/3}, \\ &\text{"unstiffened" elements} \quad && C_L = 4.65\,(t/w)^{2/3}. \end{aligned} \tag{4}$$

These variations are shown in Fig. 3.

The local plate stress factor for a thin walled section of 14 tons per sq. in. steel then becomes $C_{Lm}=\sum C_L w t/\sum w t$ giving the collapse stress of the section as a whole as $14\,C_{Lm}$ tons per sq. in.

The advantage of this non dimensional stress factor presentation is realised when considering the local buckling condition for materials of yield stress other than 14 tons per sq. in.

The "equivalent" local plate stress factor for any yield stress σ_y may be obtained from Eq. (4) by substituting for the actual w/t, the "equivalent" $(w/t)_e=(w/t)\sqrt{\sigma_y/14}$ (this is evident from Eqs. (1) and (2)) and multiplying the $(C_L)_e$ value so obtained by $\sigma_y/14$.

Since in cold formed sections all the plate elements are normally of the same material the factor $(\sigma_y/14)$ may be associated with C_{Lm} instead of $(C_L)_e$ and thus for a yield stress other than 14 tons per sq. in. the local plate stress factor of the section becomes

$$C_{Lm} = [\sum (C_L)_e w t / \sum w t] (\sigma_y / 14)$$

giving the collapse stress of the section as a whole again as 14 C_{Lm} tons per sq. in. It will be noticed that all C_{Lm} factors are referred to a basic yield stress of 14 tons per sq. in. — this, at present, is the minimum guaranteed yield stress of strip material used for cold forming in Britain.

The following numerical example illustrates the procedure described:

Obtain the local plate stress factor C_{Lm} of a concentrically loaded channel section strut of 4 in. web $\times$ 2 in. flanges $\times$ 1 in. inwardly turned lip cold formed of 0.08 in. thick material, of yield stress $\sigma_y = 20$ tons per sq. in.

Plate element				w/t	$(w/t)_e = (w/t)\sqrt{(20/14)}$	$(C_L)_e$*	$(C_L)_e w$	w
Number of	w in.	t in.	Edge support					
2	1	0.08	unstiffened	12.5	14.9	0.765	1.53	2.00
2	2	0.08	stiffened	25	29.9	1.00	4.00	4.00
1	4	0.08	stiffened	50	59.9	0.63	2.52	4.00
							$\sum 8.05$	$\sum 10.00$

* Obtained from graphs of Fig. 3.

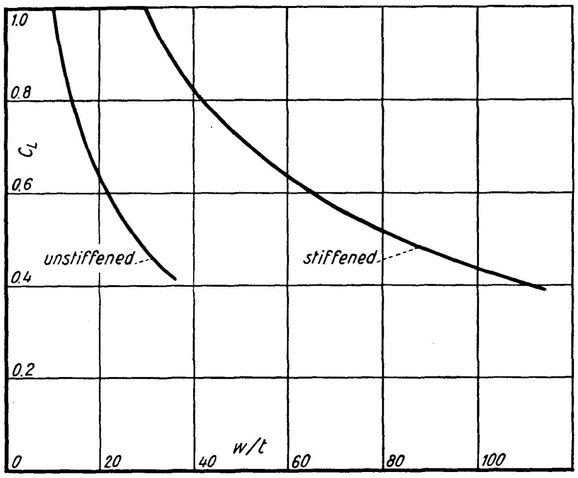

Fig. 3. Local Plate Stress Factor C_L.

Since the thickness t is the same for all the plate elements in this case, $C_{Lm} = [\sum (C_L)_e w / \sum w] (\sigma_y/14) = (8.05/10)(20/14) \doteqdot 1.15$ (this corresponds to a collapse stress for the section of $\sigma_m = 1.15 \times 14 = 16.1$ tons per sq. in.).

Overall Column Buckling and its Interaction with Local Buckling. The critical longitudinal compressive stress that initiates elastic overall instability by flexure or by a combination of flexure and torsion may be expressed for purposes of design as

$$\sigma_s = \pi^2 E r^2 / (\alpha L)^2, \tag{5}$$

where L/r is the slenderness ratio of the column appropriate to the end conditions and α is the torsional-flexural buckling factor.

Generally torsional flexural buckling is only important in angle sections and certain types of channel sections in which the flanges are of the same order of width as the web. Practical values of α for such sections have been derived [7] and have been incorporated in the specification. Their significance was discussed in detail elsewhere [6].

Failure by local buckling influences to some extent the overall buckling behaviour of columns, particularly at intermediate slenderness ratios. This is taken account of in design by allowing some interaction between the two effects.

To cater for this interaction consistent with current British structural design practice the strut failure stress σ is expressed by a Perry-Robertson type of transition between local and overall buckling as follows:

for $\alpha L/r > 80$

$$\sigma = [\sigma_m + (1 + 0.003\,\alpha L/r)\,\sigma_s]/2 - \sqrt{[\sigma_m + (1 + 0.003\,\alpha L/r)\sigma_s]^2/4 - \sigma_m \sigma_s}, \quad (6)$$

for $\alpha L/r < 80$ a linear variation from the value of σ at $\alpha L/r = 80$ given by Eq. (6) to $\sigma = 1.18\,\sigma_m$ at $\alpha L/r = 0$.

Comparison of this type of interaction with experimentally obtained average failure stresses of axially loaded mild steel struts of varying length is shown in Fig. 4. The range bounded by the curves drawn for $C_{Lm} = 0.25$ and $C_{Lm} = 1$ contains the practical range of structural sections in mild steel.

A safety factor of 2 is employed on this transition and the permissible working stress for struts σ_w is obtained as $\sigma/2$.

Typical values of σ_w derived on this basis plotted against the equivalent slenderness ratio are shown in Fig. 5.

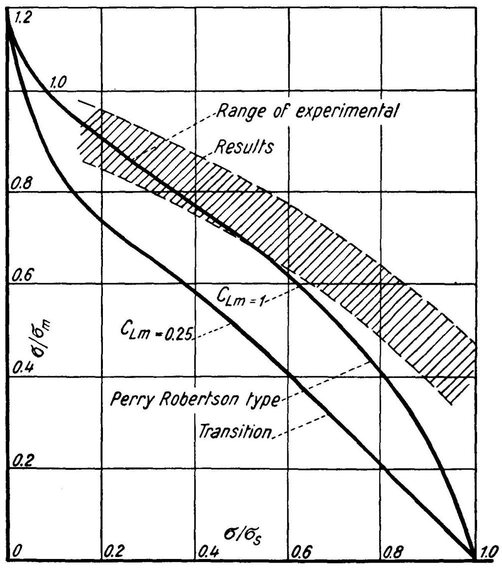

Fig. 4. Interaction of Local and Overall Collapse.

Continuing the illustrative numerical example, if the equivalent slenderness ratio $\alpha L/r$ (for the section considered $\alpha = 1$) is 110, the permissible working stress for $C_{Lm} = 1.15$ is obtained by interpolation from Fig. 5 as 3.78 tons per sq. in.

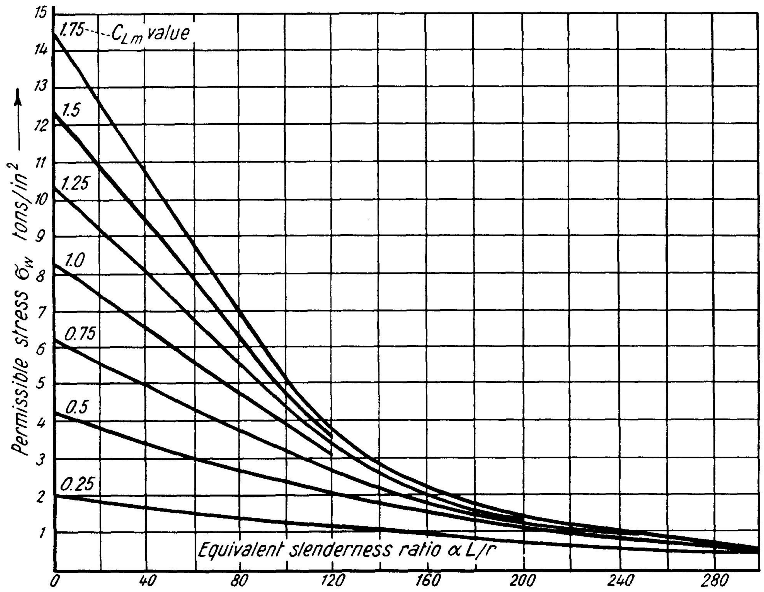

Fig. 5. Permissible Stress for Concentrically Loaded Struts.

D. Structural Behaviour and Design of Thin Walled Beams

The modes of beam failure due to instability can be classified in a manner similar to that of struts into *local* buckling of compression flanges, shear webs and *lateral* buckling involving both flexure and torsion.

The failure of locally and laterally stable beams may be assessed in the conventional way and is governed by the appropriate limit stress of the material.

Maximum Strength of Beams in Local Buckling. The compression flange of a beam bent in a plane perpendicular to the flange can be considered as subjected to uniform compressive stress and behaves substantially in the same manner as the plate element of a uniformly compressed strut. The critical compressive stress which initiates elastic buckling is again given by σ_c from Eq. (1) the values of K in this instance depending on the cross-sectional proportions of the beam section as a whole [4].

When the compressive stress reaches a certain average value of $\sigma_m > \sigma_c$ the compression flange ceases to take any further load. This however does not imply collapse of the beam which will not take place until the tension flange reaches the yield stress σ_y, the whole beam failing in a manner not dissimilar to the concept of "plastic" failure. On this simplified basis the limitation of the compression flange stress to σ_m is equivalent to assuming that certain regions of the compression flange have become "ineffective" and that the operative effective width is w_e (Fig. 6a) given by

$$w_e = (\sigma_m/\sigma_y)\,w\,. \tag{7}$$

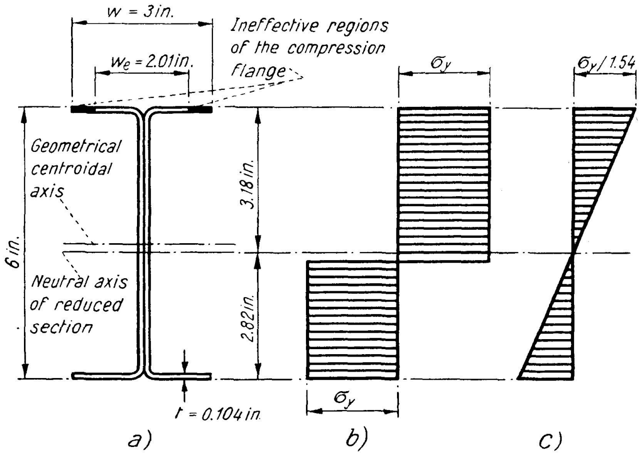

Fig. 6. Local Buckling Strength of Laterally Stable Beams.

Failure of the beam as a whole is then assumed to take place in a "plastic" manner the stress distribution at failure applied to the reduced section being as shown in Fig. 6b.

From the results of some 70 beam tests carried out [4, 6] it can be shown that the relationship given by Eq. (2) obtained from tests on struts is equally applicable to compression flanges of beams.

Fig. 7 gives a comparison of failure moments for a range of beam sections subjected to uniform bending, determined experimentally [8] with those predicted on the basis of the failure concept described above. The beams tested were laterally stable with the compression flange width/thickness ratio ranging from 10 to 128.

In the calculations the value of σ_c for "stiffened" and "unstiffened" compression flanges was again assumed to correspond to simple support at the connected edges.

It is seen that good agreement obtains between experiment and theory, confirming that the "effective width" concept is a rational basis for beam design.

In the draft specification the linear stress distribution shown in Fig. 6c is used as the basis of design. This is applied to the reduced section having an effective compression flange width of $C_L w$ where C_L is the local plate stress factor as used for plate elements of struts (Fig. 3). The permissible moment calculated in this manner incorporates a "stress factor of safety" of 1.54 on the yield in the fibres furthest from the neutral axis of the reduced section which is equivalent to a load factor in excess of 1.75 corresponding to beam collapse.

The following numerical example illustrates the procedure described:

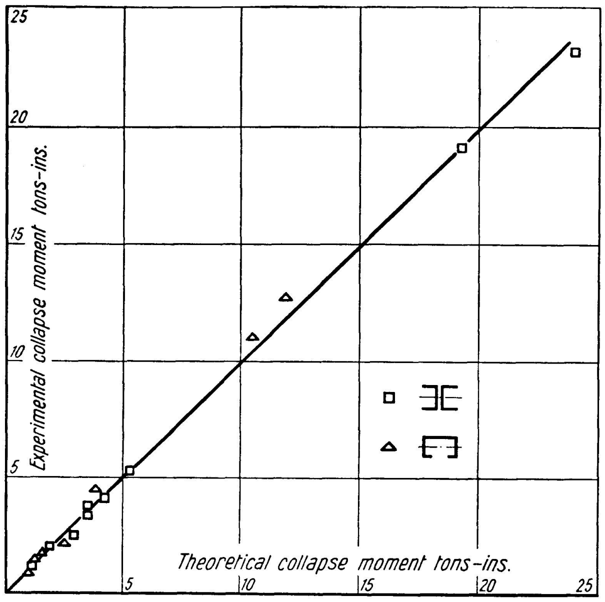

Fig. 7. Collapse Tests on Beams Subjected to Uniform Bending Moment.

Obtain the maximum permissible bending moment for a 6 in. × 3 in. double channel beam section (Fig. 6) fabricated of 0.104 in. thick material of yield stress $\sigma_y = 23$ tons per sq. in.

Equivalent w/t for flange (unstiffened) $= (w/t)\sqrt{(\sigma_y/14)} =$ $(1.5/0.104)\sqrt{(23/14)} = 18.5$.

Value of C_L from Fig. 3 = 0.67.

Effective width of compression flange $w_e = 0.67 \times 1.5 \times 2 = 2.01$ in.

The position of the neutral axis of the "reduced" section is shown in Fig. 6a giving the moment of inertia and minimum modulus of the section (referred to the compression fibre) as 7.600 in^4 and 2.39 in^3 respectively.

The permissible extreme fibre stress $= 0.65\,\sigma_y = 14.95$ tons per sq. in. giving the maximum permissible bending moment as $14.95 \times 2.39 = 35.7$ tons in.

Shear buckling of the webs in cold formed sections is encountered in hot rolled deep web girders and presents no new features. The permissible stresses are determined in the conventional way using a safety factor of 2 on the shear stress initiating elastic instability. Webs if required are reinforced by attaching vertical stiffeners or alternatively forming reinforcing ribs in the web plate components during the manufacturing process.

Local crippling of bent or compressed plate elements is present where heavy concentrated loads are applied through the flanges into the web of the beam. The treatment of this problem in the draft specification is based on the pioneer work of Professor WINTER and his Associates [1], no systematic research work on this aspect having been undertaken in Britain. Generally the design attitude, to local crippling, is directed to avoid it becoming a critical factor, through careful detail design.

Lateral Buckling of Beams. In practice cold formed steel beams are frequently continuously supported along their compression flanges. Where beams are not so supported, lateral buckling involving both flexure and torsion must be considered. For a long beam under transverse load and with no intermediate supports between the ends of the beam, the critical value of the transverse load initiating elastic buckling is a function of $\sqrt{(E I_y G J)}$ where $E I_y$ is the lateral bending stiffness and $G J$ the torsional stiffness of the beam.

For the practical range of beam sections symmetrical about the plane of bending the nominal extreme fibre stress at which buckling is initiated may be represented with reasonable accuracy by

$$\sigma_f = \text{constant} / (L/r_y)^2 \tag{8}$$

where L/r_y is the slenderness ratio corresponding to the lateral bending stiffness. This is in line with current American design practice and in the draft British Specification a form of this type is used, incorporating a safety factor of 2.

Beams which are not symmetrical about the plane of bending may twist appreciably when subjected to load and may require continuous lateral supports to eliminate this.

As indicated in the Introduction the research programmes at present in hand include the investigation of lateral instability of beams and in this connection the characteristic behaviour of unsymmetrical beams in particular is being examined.

E. Comments on the Utilisation of Design Data

Under this heading it is proposed to comment briefly on three general aspects of design considered of importance:

Presentation of Design Data. The role of applied research in development is firstly to elucidate the *basic* characteristics of behaviour and secondly to show how these basic research results may be translated into design data in a rational and duly conservative manner. The more *basic* structural design becomes, that is the more it takes realistic and rational account of all the relevant factors, the more frequently it has to consider *simultaneously* a number of different features. Cold formed design comes into this category in that even in its simplest aspects it requires the simultaneous consideration of "local" and "overall" failure. In such a field tabular or formular presentation of design data tends to become clumsy and prone to misinterpretation. In consequence a considerable amount of attention has been paid to the presentation of design data in chart form. Most of the basic relations incorporated in the draft specification such as the evaluation of C_{Lm} for struts or that of the reduced section characteristics for beams are amenable to alignment or intercept chart presentation. Such charts if properly laid out and drawn to a reasonably large scale provide the sensitivity and accuracy necessary for structural design computations and preclude misuse even by personnel unacquainted with the complete technical background. Charts of this type have been published [6, 7] and their inclusion in the draft specification is under active consideration at present.

Economic Design. The general aim of the structural designer should always be the utilisation of the full potential strength of the material used in fabrication. The cold forming process because of its flexibility lends itself to the production of a variety of outlines and provides the designer with the opportunity not only of producing structurally sound but rather structurally efficient and economically sound designs. Applied research in Britain includes these aspects and a number of studies have been published [2, 6, 7, 9, 10] concerning the efficient use of material in the design of struts and beams and of the economic aspects of using high strength steels. It is to be hoped that the opportunities in this respect that exist in the cold-formed field will be fully exploited unfettered by conventions current in related fields of structural engineering.

Experimental Basis of Design. It was indicated in the text that the Specification discussed was drafted so as to fit in with current British Structural practice and to have a reasonable degree of flexibility thereby encouraging rather than limiting cold formed development. Consistent with these aims, constructions where the methods of design given in the Specification are inapplicable or are inappropriate are admissible under the Specification provided they are experimentally proved. The load tests required for this purpose, as laid down in British Standard 449 — 1959 "The Use of Structural Steel in Building" have been accepted as applying to cold formed construction also. These are planned to ensure that the construction has a) adequate strength to support a load equal to twice the sum of the dead load and the specified superimposed load and b) adequate stiffness to support without excessive deflection a total load consisting of the dead load and $1^1/_2$ times the specified superimposed load.

References

1. Publications of the American Iron and Steel Institute as follows: Specification for the Design of Light Gage Steel Structural Members. 1946. Light Gage Cold-Formed Steel Design Manual, 1951.
   Commentary on the Light Gage Cold-Formed Steel Design Manual by George Winter, 1958.
2. Applications of Cold-Formed Sections by R. M. Kenedi and W. Shearer Smith. West of Scotland Iron and Steel Institute, Paper No. 509, May 1958.
3. Theory of Elastic Stability by S. Timoshenko. McGraw-Hill Book Co. New York, 1936.
   Buckling Stresses for Flat Plates and Sections by E. Z. Stowell, G. J. Heimerl, C. Libone and E. E. Lundquist. Transactions American Society of Civil Engineers Separate No. 77, 1951.

Behaviour of Thin-Walled Structural Members in Compression by A. H. CHILVER, p. 281, Engineering 1951.
Structural Strength of Thin Walled Sections by J. M. HARVEY, ibid. p. 291, 1953.
Thin-Walled Structural Members by A. H. CHILVER. Thesis for the degree of Ph. D., University of Bristol 1950.

4. Studies on the Interaction of Plate Components of Structural Sections under Selected Load Conditions by J. M. HARVEY. Thesis for the degree of Ph. D., University of Glasgow 1952.
5. Maximum Strength of the Thin Walled Channel Strut by A. H. CHILVER. Civil Engineering and Public Works Review, p. 1143, 1953.
6. Light Structures — Research and its Application to Economic Design by R. M. KENEDI, W. SHEARER SMITH and F. O. FAHMY. Paper No. 1203, Transaction of the Institution of Engineers and Shipbuilders in Scotland 1956.
7. The Contribution of Research in Developing the Use of Cold-Rolled Sections in Buildings by R. M. KENEDI and A. H. CHILVER. Sheet Metal Industries, August-September 1959.
8. Technical Report No. 3. Results of Experiments on Struts and Beams by A. H. CHILVER. Cold Rolled Sections Association 1950 (not published).
9. The Use of Equal Strength Sections in Structural Design by R. M. KENEDI and J. M. HARVEY, Paper No. 1136, Transactions of the Institution of Engineers and Shipbuilders in Scotland 1951.
10. Problems of Design Associated with the Use of Cold-Rolled Sections for Structural Purposes by E. GRIFFIN. Sheet Metal Industries, August-September 1958.

Supplementary Note

Since the submission of the original manuscript results of the research work on the lateral buckling of beams (referred to on page 147) carried out by CHILVER at the University of Cambridge have become available. These will shortly be published in full. Meantime it is relevant to point out that the results of this work indicate that σ_f can be represented by simple expressions of the type

$$\sigma_f = A/(L/r_y)\,, \tag{9}$$

where A is a numerical factor dependent on the thickness of the section.

In consequence the adoption of equation (9) in place of equation (8) as the basis for assessing the lateral stability of symmetrical beam sections in the draft British Specification is now under active consideration.

Summary

Many of the structural problems of cold formed sections are similar to those encountered in the design of conventional steel structures. Certain additional considerations — not associated with the design of hot rolled steel structures for example — arise because of the relatively thinner section outlines employed in cold formed construction. These introduce in particular the problems of local and of flexural-torsional instability in the case of both columns and beams.

The paper summarises briefly the relevant research work carried out in Britain under the Sponsorship of the Cold Rolled Sections Association and indicates the translation of its results into design data. This provides the background for a new British Standard Specification, dealing with "The Use of Cold Formed Sections in Building" at present in draft form and shortly to be issued as an Addendum to the main British Standard 449: 1959 — "The Use of Structural Steel in Building".

Résumé

Dans l'emploi des profilés à froid, l'étude des projets pose très fréquemment des problèmes analogues à ceux que l'on rencontre dans la construction métallique classique. Quelques considérations complémentaires interviennent qui ne se présentent pas dans l'étude des projets avec profilés laminés à chaud et qui résultent du fait que dans l'utilisation pratique des profilés à froid, les sections des profils sont relativement minces. Ceci pose en particulier des problèmes de stabilité locale et de flambage latéral, qu'il s'agisse de poteaux ou de poutres.

Les auteurs exposent brièvement les importantes investigations qui ont été effectuées en Grande-Bretagne sous l'égide de la Cold Rolled Sections Association et ils exposent les résultats obtenus en ce qui concerne les bases des projets. Ainsi se trouve posé le fondement d'une nouvelle norme britannique, concernant l'emploi des profilés laminés à froid dans la construction. Cette norme est actuellement à l'étude et sera prochainement publiée à titre d'appendice à la norme principale britannique 449 (1959) concernant l'emploi de l'acier dans la construction.

Zusammenfassung

Viele Entwurfsprobleme bei Verwendung kaltgeformter Profile sind ähnlich denen, die beim Entwurf von konventionellen Stahlbauten begegnet werden. Einige zusätzliche Betrachtungen — die sich beim Entwurf mit warmgewalzten Profilen nicht ergeben — entstehen wegen der relativ dünnen Querschnittsprofilierung bei der praktischen Verwendung von kaltgeformten Profilen. Dies führt im besonderen auf die Probleme der lokalen und der Kippinstabilität, sei es bei den Stützen wie bei den Trägern.

Diese Veröffentlichung faßt kurz die bedeutende Forschungsarbeit in Großbritannien unter der Ägide der Cold Rolled Sections Association zusammen und zeigt die Übersetzung der erhaltenen Resultate in Entwurfsgrundlagen. Damit ergibt sich der Hintergrund für eine neue britische Norm, die von «der Verwendung von kaltgewalzten Profilen im Bauwesen» handelt. Sie ist momentan im Entwurf und wird bald als Anhang zur britischen Hauptnorm 449 (1959) «die Verwendung von Baustahl im Bauwesen» veröffentlicht.