Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte

Band: 49 (1986)

Artikel: New British code for the design of aluminium structures

Autor: Bulson, Philip S. / Nethercot, David A.

DOI: https://doi.org/10.5169/seals-38280

New British Code for the Design of Aluminium Structures

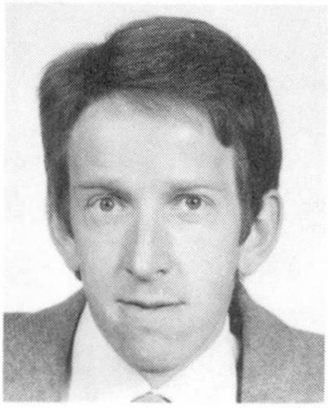
Nouveau code britannique pour le dimensionnement des structures en aluminium

Neue britische Vorschrift für die Bemessung von Aluminiumkonstruktionen

Philip S. BULSON
Director
Mott Hay & Anderson
Winchester, UK

Philip Bulson was a postgraduate at Bristol before he began his career in research, development and management of military engineering. Much of his work has dealt with military aluminium structures, and he is chairman of the codes of practice committee for structural aluminium.

David A. NETHERCOT
Senior Lecturer
Univ. of Sheffield
Sheffield, UK



David Nethercot, born 1946 obtained his degree at University College Cardiff where he was also an ICI Fellow. He has been active in research, teaching and code drafting in steel and aluminium structures at Sheffield since 1971.

SUMMARY

The procedures for the determination of the static strength of thin-walled members in BS 8118, the new UK code for the structural use of aluminium are summarised. Attention is concentrated on the various forms of instability that must be considered, with particular attention being given to those special aspects of aluminium construction which distinguish it from structural steelwork.

RÉSUMÉ

Les procédures utilisées pour déterminer la résistance statique des éléments à parois minces, qui figurent dans le nouveau règlement britannique «British Standard 8118», concernant l'emploi structurel de l'aluminium, sont résumées. Une attention particulière est accordée aux diverses formes d'instabilité qu'il faut prendre en compte, en soulignant surtout les aspects propres aux structures en aluminium qui les distinguent des constructions en acier.

ZUSAMMENFASSUNG

Die Verfahren zur Bestimmung der statischen Festigkeit von dünnwandigen Bauelementen nach BS 8118 (die neue englische Vorschrift für die konstruktive Verwendung von Aluminium) werden zusammenfassend beschrieben. Die verschiedenen zu berücksichtigenden Instabilitätsformen werden im Detail behandelt. Besonders hingewiesen wird auf diejenigen Merkmale der Aluminiumkonstruktionen, die sie von den Stahlkonstruktionen unterscheiden.

1. INTRODUCTION

The current British Code of Practice for Structural Aluminium (CP 118) was published in 1969, and although aimed at general engineering structures of most types (but not aircraft or aerospace structures), its roots lay in earlier Reports by the Institution of Structural Engineers on the structural use of aluminium alloys in building. CP 118 acknowledged that in pursuit of structural efficiency thin-walled structural members were part of a designers tool-kit, and the problems of local and torsional instability, and of lateral buckling, were pursued in depth.

The British Code is now under revision, and a new version, BS 8118, is due in late 1986 or early 1987. It has been written in limit state format, and as well as the changes that result from this, there has been a considerable extension of the treatment of static strength, fatigue life and joint analysis. The problem of heat affected zones in welded structures has received attention in the light of British research, and in addition new design rules for the ultimate strength of structural components subjected to loads that induce buckling have been established. These use the results of research and testing in many countries during the 1970's and early 1980's.

The purpose of this paper is to summarise the important developments incorporated in the new code, with particular reference to thin-walled structures. There are several areas that need discussion here: design principles; heat affected zones; the static strength of struts, ties, beams, plates, beam-columns and plate girders.

2. DESIGN PRINCIPLES

It is recognised that all structures must be checked for static strength, deformation and corrosion. In addition, certain structures will also need to be examined for overturning, fatigue and vibration.

For static strength the procedure is to check that for a component the factored resistance is not less than the Action under the factored loading × γ ,where λ_c is a partial factor to take account of the consequences of failure. The factored resistance is the calculated resistance ÷ γ_m, where γ_m is the material partial factor. This factor takes account of differences between the strength of material test specimens and the strength of the actual material in the structure as manufactured.

The load factor, γ_f, is the product of two components γ_{f1} and γ_{f2}. Factor γ_{f1} will be given by the relevant British structural loading standard, where one exists, but to check the strength of a structure where no guidance is available, BS 8118 suggests a set of factors to take account of dead loading, imposed loads (other than wind loads), wind load, and forces due to temperature effects. The load factor γ_f, to take account of the low probability that the severest loading actions will occur simultaneously, can be found analytically if enough statistical information exists, but for preliminary designs the code gives simple values (1.0, 0.8, 0.6 etc.) that are known to give reasonable agreement with more exact probability analysis. In certain codes γ_c is incorporated in γ_f or γ_m, but because of the wide-ranging use of BS 8118 separate values are given in this Code.

3. HEAT-AFFECTED ZONES (HAZ)

The treatment of Heat-affected zones in BS 8118 represents a notable step forward from earlier codes. Recent research at Cambridge University (1) has thrown a new light on the extent and strength of these zones, and much of this work has been incorporated in the design rules.

Thus detailed guidance is given for a variety of weld types, welding procedures and alloy classes. This will frequently lead to smaller strength reductions than those obtained with the old "one inch" rule.

4. STATIC STRENGTH OF STRUCTURAL MEMBERS

Almost one half of the Code is devoted to the detailed procedures required for the determination of the resistance of ties, struts, beams etc. The form of these may best be appreciated by considering the clauses for one particular type of element in some detail; the material covering the design of laterally unrestrained beams has been selected as representative of the general style.

4.1 Unrestrained Beams

The resistance moment M_{max} is determined from:

$$M_{max} = M_S \, C_{LT}/\gamma_m \qquad (1)$$

in which M_S = basic moment capacity
C_{LT} = reduction factor for lateral buckling
γ_m = material factor (1.2 for extruded beams, 1.25 for welded beams)

Presentation of the design expression in this way provides explicit recognition of the various phenomena that influence beam strength. Thus M_S, which does, of course, represent the resistance for a beam not susceptible to lateral-torsional instability such as a box-section or a closely braced or continuously restrained I-section, is obtained as:

$$\begin{array}{lll} M_S = f_{0.2}\,Z_p & \text{for a compact section} & \\ f_{0.2}\,Z_p > M_S \geqslant f_{0.2}\,Z_e & \text{for a semi-compact section} & (2) \\ M_S = f_{0.2}\,Z_{eff} & \text{for a slender section} & \end{array}$$

Since section classification depends upon the extent to which local buckling affects its capacity, the class of a particular section will depend principally upon the width/thickness ratio of its most slender plate element. Account is also taken of alloy strength, stiffeners and the presence of any weakened HAZ material through the use of a plate slenderness parameter defined as:

$$\frac{hb}{t}\sqrt{sf_{0.2}/250} \qquad (3)$$

in which b = plate width
t = plate thickness
h = factor to allow for the effect of stiffeners
s = reduction factor for HAZ effects
$f_{0.2}$ = 0.2 percent proof stress in N/mm^2

Fig. 1 illustrates the dependence of M_S on b/t for two particular alloys for an unstiffened box section. This also shows how the practice in CP 118 of requiring separate design curves for each alloy has been simplified to consideration of two groups, the division between which is made solely on the basis of the ratio $f_u/f_{0.2}$, in which f_u is the ultimate tensile strength of the material. Since this is related to the parameter n used in the Ramberg-Osgood representation of the stress-strain curve, it distinguishes between material with a very rounded stress-strain curve for which $f_u/f_{0.2} \geqslant 1.4$ and more sharply yielding material for which $f_u/f_{0.2} < 1.4$. For the former Z_p is not used because of the large strains and thus excessive deformations needed to reach 'M_p'. In determining Z (plastic, elastic or effective) it is, of course, necessary to allow for the presence of HAZ material whose strength is less than $f_{0.2}$. Inclusion of when calculating plate slenderness recognises the lower stresses present in the unaffected zones of welded members that results from the use of these lower Z values.

The lateral buckling reduction factor C_{LT} is given in terms of a beam slenderness λ_{LT} which is based directly on the fundamental parameter $\sqrt{M_S/M_{cr}}$, in which M_{cr} is the elastic critical moment. This follows the practice of several recently published steel codes - notably EC3 - but in line with recent U.K. practice considerable assistance is given with the calculation of λ_{LT} for the more common structural shapes. Thus for I's, channels and tees λ_{LT} is given by

$$\lambda_{LT} = u\ v\ (\ell/\rho_y)\sqrt{f_{0.2}/250} \qquad (4)$$

in which u = 0.85 for symmetrical I's, 0.75 for symmetrical channels and 1.0 for tees

$v = f\left(\frac{\ell}{\rho_y} \times \frac{t}{D_b}, N\right)$ is obtained from a graph

ℓ = effective length
ρ_y = radius of gyration about the weak axis
t = mean flange thickness
D_b = overall depth
$N = I_c/(I_c + I_t)$ is a measure of the degree of monosymmetry

Explicit expressions for λ_{LT} are also provided for solid and hollow rectangular sections. All of these assume $M_S = f_{0.2} Z_p$.

Positioning of the actual $C_{LT} - \lambda_{LT}$ design curve was based largely on test data (2) since no complete theoretical solution embracing the combined effects of inelastic material behaviour, initial geometrical imperfections etc. is presently available. Tests from 4 types of cross-section under 5 different patterns of loading were employed. Fig. 2 compares the resulting design curve with these data. Following the approach taken with the strut curves, as well as that employed generally for buckling problems in recent U.K. codes, this may be represented by a modified Perry equation of the form

$$(M_{cr}/M_S - C_{LT})(1 - C_{LT}) = \eta\ C_{LT}\ M_{cr} \qquad (5)$$

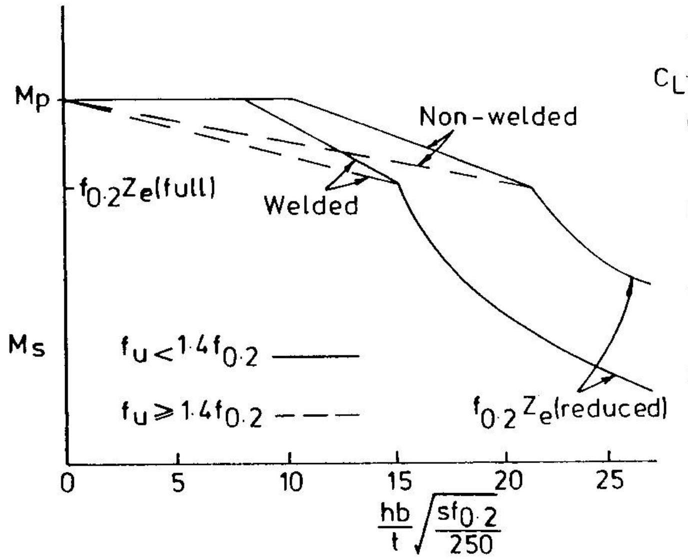


Fig. 1 Moment Capacity M_s - plotted for a beam with $f_u = 1.2f_{0.2}$ or $f_u = 1.4f_{0.2}$ with capacity controlled by an internal element

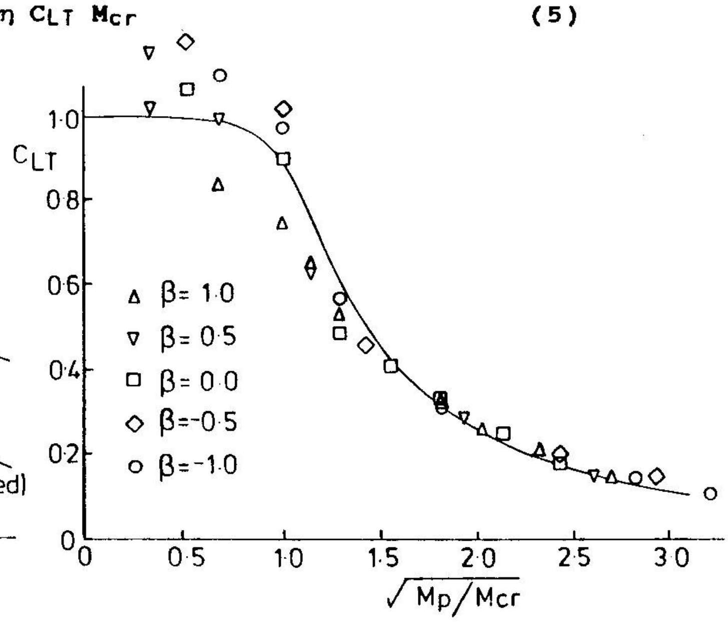


Fig. 2 Values of C_{LT} for lateral-torsional buckling, β = ratio (smaller end moment)/(larger end moment)

In the absence of either a comprehensive theoretical treatment or a sufficiently large and well structured body of test data, differentiation between classes of section in the manner of multiple column curves (which are used in this Code) was not possible. A particular difficulty was caused by the absence of test data for welded beams, where evidence for steel (3) demonstrates the lower lateral buckling strength of such members. Although the reduction in basic cross-sectional strength is allowed for in the determination of M_S, the existance of a further deleterious interaction of HAZ and residual stresses on overall buckling is a possibility. A single design curve has therefore been given corresponding to

$$\eta = 0.001\,(\lambda_{LT} - 10.6)/\sqrt{f_{0.2}/250} \qquad (6)$$

Whether or not this gives lower safety margins in the case of welded beams will have to await the production of the necessary data. For the 80 test results on extruded members the mean of the ratio (predicted strength)/(test strength) is 1.013 with a standard deviation of 0.11. In making these comparisons actual measured cross-sectional dimensions and material properties (including the compressive 0.2 per cent proof stress where possible) were used.

Fortunately 27 of these tests (4) were conducted under unequal end moment loading, thereby providing an opportunity to check the suitability of the equivalent uniform moment concept as a means of recognising the generally less severe effects of a non-uniform moment on lateral stability. Fig. 2 shows how the Code's provision for using

$$\bar{M} = mM_1 \qquad (7)$$

in which $m = (0.6 + 0.4\,M_2/M_1) \nless 0.40$
M_1, M_2 = larger and smaller end moments respectively

in place of M_1 causes these results to plot with those for uniform bending. The use of m-values of less than unity does, of course, mean that an additional check that M_1 does not exceed M_S is also necessary.

Guidance is also provided, through the use of effective length factors, on the approximate effects of end fixity and destabilising loads i.e. those applied above the level of the shear centre in such a way that they are free to move sideways with the beam as it buckles. In the case of cantilevers, factors previously included in recent U.K. steel codes recognise the importance of restraint of the tip and, in the case of cantilevers formed by overhanging spans, at the vertical support.

The enterprising designer is given the opportunity to use research data for M_{cr} by giving the basic expression for λ_{LT} as

$$\lambda_{LT} = 53\sqrt{M_S/M_{cr}} \qquad (8)$$

This approach should also prove advantageous for beams containing slender plate elements for which M_S should be determined using Z_{eff}. Using some of the test data obtained by Cherry (5), Fig. 3 shows how the use of the simplified Eq. 4 for the full section leads to very conservative allowances for the interaction of local and overall buckling. Cherry's tests were conducted on specimens with extremely thin compression flanges for which the Code gave t_e/t values of around one fifth, leading to values of M_S of approximately one half of $f_{0.2}\,Z_p$. For sections just requiring the use of Z_{eff} the difference would, of course, be substantially less. An improved, but still conservative, result may be obtained if the approximate λ_{LT} is reduced in the ratio $\sqrt{Z_{eff}/Z_p}$.

4.2 Other Types of Member

Strut design is based on an expression analogous to Eq. 1 with the important difference that C_c, the reduction factor for flexural buckling, is defined by the 5 column curves shown in Fig. 4 with the allocation of a particular member being on the basis of cross-section, alloy and method of manufacture as given in Table 1. Because of the multiplicity of shapes possible with the extrusion process, torsional buckling is more likely than is the case for conventional steel sections. Accordingly a separate check that the reduction factor for torsional buckling C_T, the value of which is obtained using $\lambda_t \sqrt{f_{0.2}/250}$ in which $\lambda_t = 830/\sqrt{f_{cr}}$ and f_{cr} is the elastic critical stress for torsional buckling, is not less than C_c, will often be required. Procedures to assist with the rapid determination of λ_t (or f_{cr}) are provided for several structural shapes. The particular values of C_T given are largely based on 2 large series of tests (6, 7).

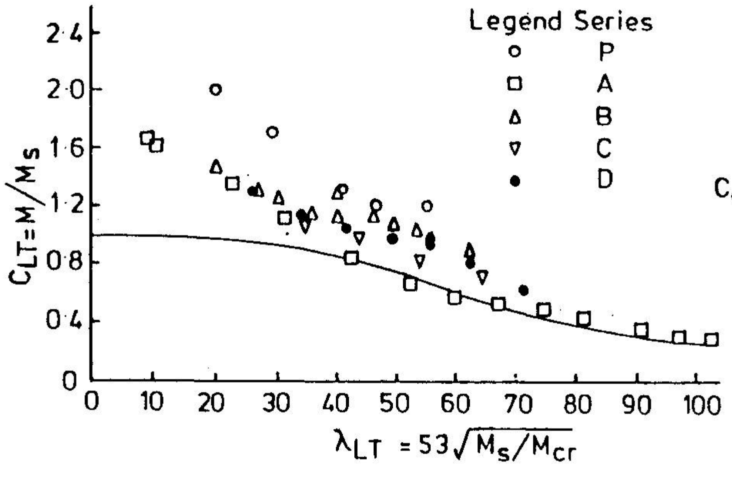


Fig. 3 Lateral-torsional buckling of beams containing slender plate elements

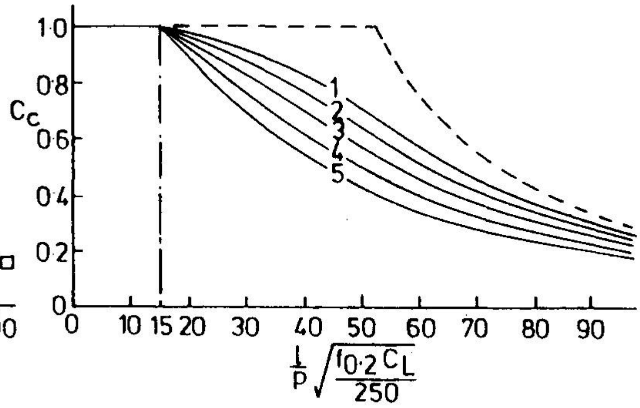


Fig. 4 Values of C_c for flexural buckling

Cross-Section		$f_u/f_{0.2}$ < 1.2	$f_u/f_{0.2}$ > 1.2
symmetric	non-welded	1	3
	welded	2	4
asymmetric	non-welded	2	4
	welded	3	5

symmetric $Y_1 / Y_2 < 1.2$
asymmetric $Y_1 / Y_2 > 1.2$
where Y_1 and Y_2 are extreme fibre distances

Table 1 Column curve selection

Members subject to combined bending and compression (or tension) are treated by means of interaction formulae. These are similar to those of the new U.K. steelwork code careful checking against several series of test data (2) having confirmed their suitability. Thus for the most general case of biaxial bending, buckling failure is checked using

$$\frac{\bar{M}_x}{M_{ax}} + \frac{\bar{M}_y}{M_{ay}} \ngtr 1 \qquad (9)$$

in which M_{ax} is the lesser of

$$M_{ax} = M_{max}\,(1 - P/P_{cy}) \qquad \text{for weak axis failure}$$

$$M_{ax} = \frac{(1 - P/P_{cx})}{(1 + \quad P/P_{cx})}\,\frac{M_{sx}}{\gamma_m} \qquad \text{for strong axis failure}$$

and M_y is obtained from the second expression by replacing x with y.

Treating the biaxial problem in this two stage fashion enables potential difficulties over factors such as different effective lengths for strong and weak axis column buckling, uncertainty over the governing mode under strong axis bending etc. to be avoided.

Plate girder design (8) is based on the type of interaction diagram for combined moment and shear shown as Fig. 5, which is similar to that used in the U.K. steel bridge code. Since V_{CF} and M_{CF} represent the shear capacity of the web alone and the moment capacity of the flanges alone, girders may be designed relatively simply for V_{CF} and M_{CF} in situations where M and V are both significant or for full moment and $V_{CF}/2$ or full shear and $M_{CF}/2$ where one type is dominant. Tension field action is utilised for vertically stiffened girders when determining the reduction factor C_{ST} on full shear capacity. When combined with longitudinal stiffeners the latter are assumed to affect only the initial buckling of the web. Thus C_{ST} is given by

$$C_{ST} = v_1 + v_2 + m^* v_3 \qquad (10)$$

in which v_1 = represents initial buckling resistance
v_2 = contribution due to tension field action anchoring on the transverse stiffness
v_3 = additional contribution due to flanges
m^* = measure of flange strength

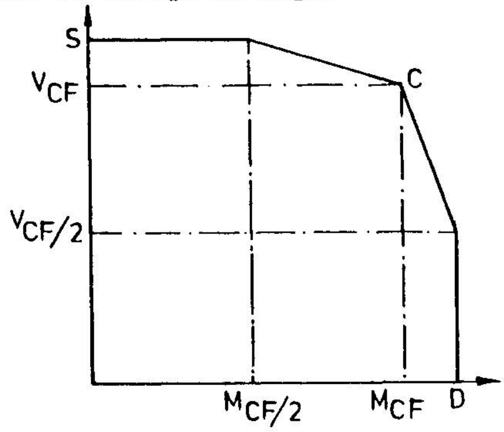


Fig. 5 Moment/shear interaction diagram

Graphs for determining each of these 4 quantities in terms of web slenderness d/t and panel aspect ratio a/d are provided.

The topics of joint design and fatigue have been treated in broadly the same fashion as in recent U.K. steel codes but noting any special features present in aluminium. Interested readers are referred to the appropriate papers (9, 10) of the draft code symposium for a brief account of the most important developments and to the forthcoming book (11) on the background to the Code for a lengthier treatment.

4.3 Topics Identified as Requiring Further Study

Although the Code incorporates the findings of much recent research, in several areas the drafting was hampered by insufficient data. On this basis the following are noted as requiring some attention:

1. Ultimate strength of members containing HAZ, particularly when these are due to transverse welds.
2. Ultimate strength of sections containing slender plate elements, particularly when these elements are subject to non-uniform stress.
3. Ultimate strength of members subject to biaxial bending and torsion.

Of course no code can cover every considerable situation adequately and it is for this reason that an important section of the Code is that covering the conduct of physical testing as a basis for design.

5. CONCLUSIONS

Certain aspects of the new British Code for the structural use of aluminium, BS 8118, have been summarised. Calibration studies against the previous document, CP 118, suggest that the new Code is more economic in terms of material usage and more rational in its coverage.

6. ACKNOWLEDGEMENTS

The authors are grateful to the many people who have assisted with the development of the Code and whose work forms the basis for the material contained therein The contribution of Mr J.B. Dwight to the formulation of design clauses is particularly acknowledged.

REFERENCES

1. Robertson, I. and Dwight, J.B., "HAZ Softening in Welded Aluminium", 3rd International Conference on Aluminium Weldments, Munich, 1985.

2. Mazzolani, F.M., "Aluminium Alloy Structures", Pitman, London, 1985.

3. Fukumoto, Y., and Kubo, M., "A Survey of Tests on Lateral Buckling Strength of Beams". Preliminary Report, Second International Colloquium on Stability of Steel Structures, Liege, 1977.

4. Clark, J.W. and Jombock, J.R., "Lateral Buckling of I-beams Subjected to Unequal End Moments", J. Eng. Mech. Div., ASCE, EM3, 1957.

5. Cherry, S., "The Stability of Beams with Buckled Compression Flanges", The Structural Engineer, Sept. 1960.

6. Baker, J.F. & Roderick, J.W., "The Strength of Light Alloy Struts", Aluminium Development Association Report No. 3, 1948.

7. Smith, R.E., "Column Tests on Some Proposed Aluminium standard Structural Sections", Report B-IR-241-54-30/2, Aluminium Laboratories, Banbury, 1954.

8. Evans, H.R., "Design of Plates and Plate Girders", Symposium on The Structural Use of Aluminium, Institution of Structural Engineers, October 1985.

9. Cullimore, M.S.G., "Joints", Symposium on the Structural Use of Aluminium, Institution of Structural Engineers, October 1985.

10. Ogle, M., "Fatigue", Symposium on the Structural Use of Aluminium, Institution of Structural Engineers, October 1985.

11. Bulson, P.S. ed., "Aluminium Structures", Elsevier Applied Science Publishers, 1987.