The design of the webplates of light alloy plate girders

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The design of the webplates of light alloy plate girders

Die Berechnung der Stehbleche von Leichtmetallträgern

Cálculo das almas das vigas de ligas leves

Calcul des âmes des poutres en alliages légers

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1. Introduction.

This paper presents the general conclusions obtained from extensive tests on girders constructed of H. 10. WP. (1) high strength aluminium alloy. With the aid of the new experimental information an improved procedure for the design of plate girder webplates constructed of high strength aluminium alloys has been developed.

The proposed design procedure, in fully recognising that the buckling of plate girder webplates merely results in a redistribution of the stress system, recommends that when the buckling stress is below the maximum permissible working stress, the webplate be designed to operate in the postbuckled range. The extent to which the plate may be loaded beyond the buckling load is, in many cases controlled by aesthetic requirements rather than stress requirements.

One important feature of the proposed design procedure is the new relationships between the size and spacing of intermediate vertical stiffeners and the buckling stress of the stiffened webplate.

2. The Behaviour of Stiffened Webplates Subjected to Shear Stresses.

2. 1. *Background.*

Theoretical studies [1, 2, 3] have shown that the shear buckling stress (T_{cr}) of rectangular panels is given by equation (1) ; K being a coefficient,

(1) See British Standard 1470:1948. Wrought Aluminium and Aluminium Alloy.

the value of which depends upon the superficial dimensions of the panel and the type of edge support, see figure 1.

$$T_{cr} = \frac{K \pi^2 D}{d_c^2 \, t} \tag{1}$$

where D = Flexural Rigidity of Unit Width of Plate $= \dfrac{Et^3}{12\,(1-\mu^2)}$

μ = Poisson's Ratio

d_c = Clear Depth of Webplate

b = Width of Unstiffened Plate or Stiffener Spacing for Stiffened Plate

t = Thickness of webplate

E = Young's Modulus of Material.

As will be seen from figure 1, the value of K increases with decreasing values of the aspect ratio b/d_c. It is therefore evident that by the judicious use of effective intermediate stiffeners the buckling stress of a plate can be increased to any desired value. However, it must be appreciated that the buckling stress of a plate stiffened by intermediate stiffeners is not solely dependent upon the spacing of the stiffeners, being affected by the relative value of flexural rigidity of the intermediate stiffener (EI) employed to the flexural rigidity of a strip of webplate equal in width to the stiffener spacing b.

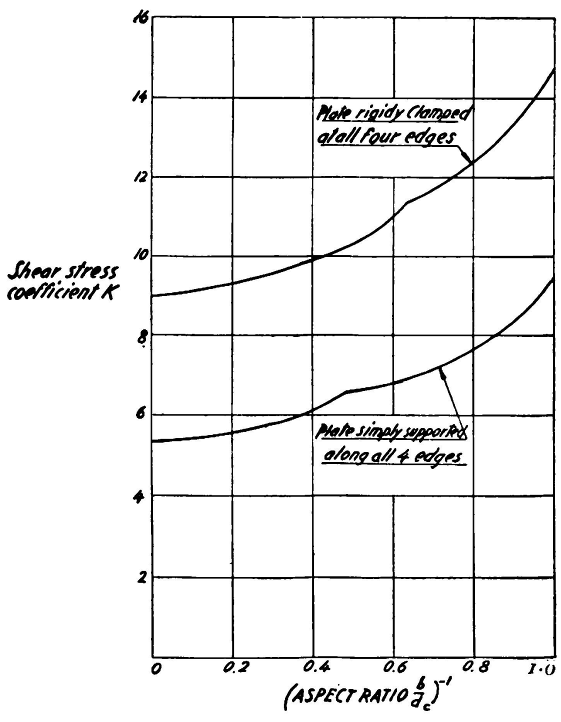

FIG. 1

Theoretical relationships [1, 4, 5] between the size and spacing of intermediate stiffeners and the buckling stress of the stiffened webplate have been obtained for different conditions of edge support and panel dimensions. However, in obtaining these relationships the investigators have made certain simplifying assumptions which are not strictly correct. Therefore, before these relationships can be used in practice, experimental confirmation is necessary. Unfortunately, however, not one of the experimental investigations [6, 7, 8] conducted to date has been successful.

2. 2. *The Buckling of Stiffened Webplates Subjected to Shear Stress.*

As mentioned above, before a correct and therefore reliable design procedure can be developed, it is essential that the relationships between the buckling stress of the stiffened panel and the size and spacing of the stiffeners be known. An experimental investigation with this objective was, therefore, conducted by the writer. Over 200 different plate-stiffener combinations were investigated, involving the use of 18 plate girders and 20 shear panels constructed of H. 10. WP. aluminium alloy.

The analysis of the results provided the following empirical relationships between the critical shear stress coefficient K and the non-dimensional parameter γ.

$$K = K_u + A\,(\gamma)^{1/3} \tag{2}$$

in which

$$\gamma = \frac{EI}{Db}$$

EI = Flexural Rigidity of Stiffeners
Db = Flexural Rigidity of Plate equal in width to stiffener spacing
K_u = Critical Shear Stress Coefficient of the Unstiffened Plate
K = Critical Shear Stress Coefficient of the Stiffened Plate, being equal to the maximum value K_L for values of γ equal to or greater than $\gamma_L \left(= \frac{EI_L}{Db}\right)$

A = A constant, the value of which depends upon the effective aspect ratio α_e and the type of stiffener employed.

For double and single sided stiffeners.

$$K_L = 7.0 + 5.6\,(\alpha_e)^{-2} \tag{3}$$

For double sided stiffeners.

$$\gamma_L = 27.75\,(\alpha_e)^{-2} - 7.5 \tag{4}$$

$$\alpha_e = \frac{\text{clear web distance between stiffeners bc}}{\text{clear web depth } d_c}$$

For single sided stiffeners.

$$\gamma_L = 21.5\,(\alpha_e)^{-2} - 7.5 \tag{5}$$

$$\alpha_0 = \frac{\text{stiffener spacing b}}{\text{clear web depth } d_c}$$

The moment of inertia of double sided stiffeners to be taken about the centre line of the webplate and the value for single sided stiffeners to be taken about the surface of the webplate in contact with the stiffener.

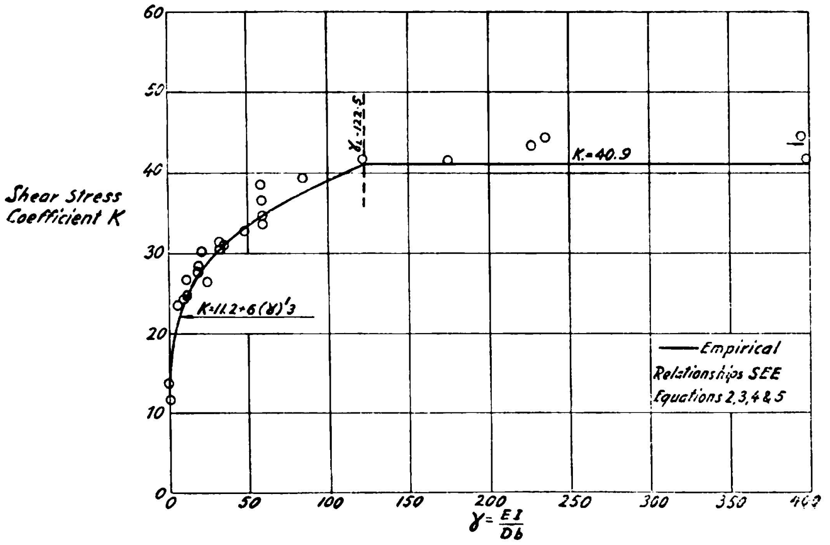

FIG. 2. Experimental values obtained when employing single sided stiffeners at $4^{7}/_{8}''$ centres on webplates having a clear depth of 12″

One restriction in the use of the above relationships is that they are only valid when the attached stiffener leg is equal to or greater than the thickness of the webplate.

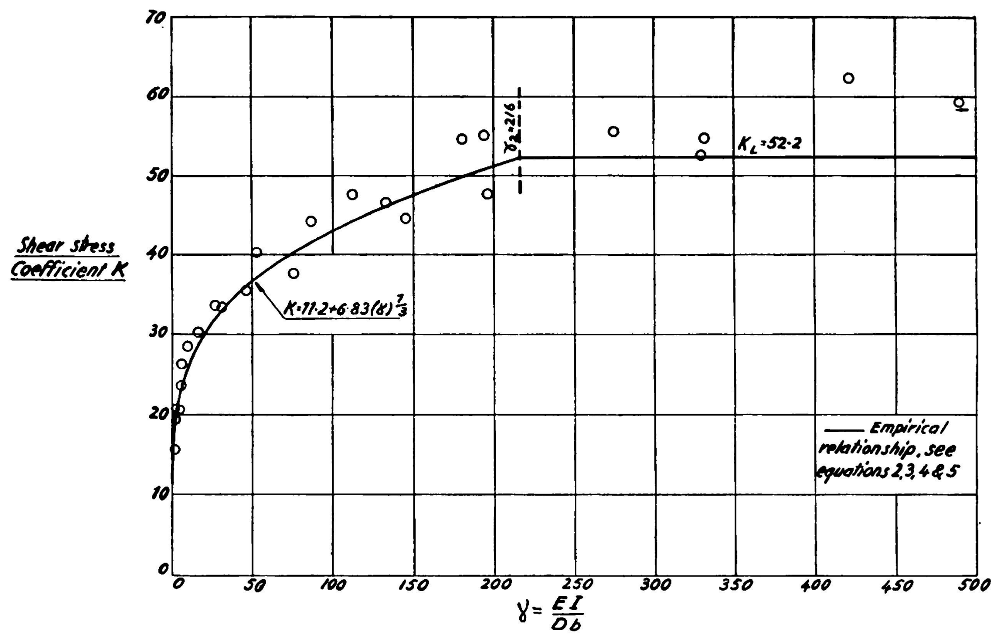

FIG. 3. Experimental values obtained when employing double sided stiffeners at 4⅞″ centres on webplates having a clear depth of 12″

To illustrate the above laws, the experimental values obtained from tests involving single and double sided stiffeners spaced at $4\,^{7}/_{8}''$ centres on girders having a clear webplate depth of 12″ are plotted in figures 2 and 3. Also plotted are the empirical relationships given above. In calculating the value of K_u, it has been assumed that the webplate, which had an aspect ratio of 1.625:1, received a completely clamped support along all 4 edges.

2. 3. *The Post-Buckled Behaviour of Stiffened Webplates Subjected to Shear Stresses.*

2. 3. 1. *The Behaviour of the Stiffeners.*

In addition to its function of increasing the buckling stress of a webplate, it is essential that an intermediate stiffener shall operate as a fully effective member when the webplate is loaded beyond its buckling load. Tests were therefore carried out on stiffened webplates to examine the post-buckled behaviour of webplates reinforced by relatively flexible stiffeners.

In order to have a standard against which the behaviour of other stiffened webplates could be compared, one girder, BTG/1, was fitted with a fairly rigid set of intermediate stiffeners, see Table 1. The tests established that stiffeners having a flexural rigidity at least equal to EI_L operated as effective members when the webplate was loaded beyond the buckling load; this being clearly indicated by the values given in Table 1

However, during those tests involving single sided stiffeners, it was noted that the distortions of the stiffeners, were greater than those occurring in double sided stiffeners at corresponding values of the load ratio w/w_{cr}. For this reason it is recomended that the depth of the outstanding leg shall not be greater than 12 times its thickness unless it be reinforced by some form of lip.

2. 3. 2. *The Behaviour of the Webplate.*

It is now generally accepted that of all the various theories of failure which have been proposed, the Hencky-von Mises theory of constant shear strain energy of distortion is the one which satisfies most loading conditions. According to this theory yielding will occur when the maximum comparison stress σ_{mc} given by equation (6), reaches the yield stress of the material as obtained by a simple tensile test.

$$\sigma_{mc}^2 = \sigma_x^2 + \sigma_y^2 + 3\tau_{xy}^2 - \sigma_x \sigma_y \tag{6}$$

where σ_x, σ_y and τ_{xy} are the stress conditions occurring in any rectangular system of co-ordinates.

With the aid of the strain gauge readings taken during the tests, it has been possible to determine the loads at which the webplates first yielded, the values obtained being given in Table 1.

The mean of the experimental relationships between the ratio of the comparison stress σ_{mc} to the idealised comparison stress $\tau\sqrt{3}$ T and the loading ratio T/T_{cr} is plotted in figure 4.

TABLE 1

Girder Reference No.	Buckling Load Tons W_{cr}	Corresponding Shear Stress Tons/sq. in. τ_{cr}	Ultimate Load Tons W_{ult}	Load at which yielding occurred in the web plates. Tons W_{yield}	$\frac{W_{ult}}{W_{cr}}$	$\frac{W_{yield}}{W_{cr}}$	$\frac{W_{ult}}{W_{yield}}$	Load at which initial yielding occurred in stiffeners Tons W_s	$\frac{W_s}{W_{cr}}$	$\frac{\gamma}{\gamma_L}$
BTG/1	4.6	2.64	25.4	8.2	5.54	1.78	3.1	24.0	5.2	3.68
BTG/2	4.5	2.58	24.96	N.D	5.55	—	—	22.0	4.88	1.12
BTG/3	3.8	2.22	22.6	8.25	5.95	2.17	2.74	14.0	3.68	0.65
BTG/4	4.7	2.65	23.2	7.0	4.94	1.49	3.32	12.7	2.7	0.80
BTG/5	4.4	2.53	23.65	8.5	5.38	1.93	2.78	11.7	2.67	1.37
BTG/8	7.83	4.66	25.0	N.D	3.20	—	—	N.D	—	1.47
BTG/10	1.67	2.30	13.35	N.D	8.17	—	—	N.D	—	5.1
ATG/1	4.62	2.31	25.5	9.1	5.5	1.97	2.8	N.A	N.A	Unstiffened plate

N. D = Not Determined

N. A = Not Applicable.

Using figure 4 in conjunction with equations (1) to (5), the applied shear stress at which any panel will start to yield can be readily determined.

Now since a factor of safety of 2.4 against general yielding is used with high strength alloys at the present time [9], and the maximum stress in a buckled plate is only approximately 1.85 times the mean stress, the mean stress at which localised yielding will occur in the webplate is greater than the maximum permissible stress, as indicated in figure 5.

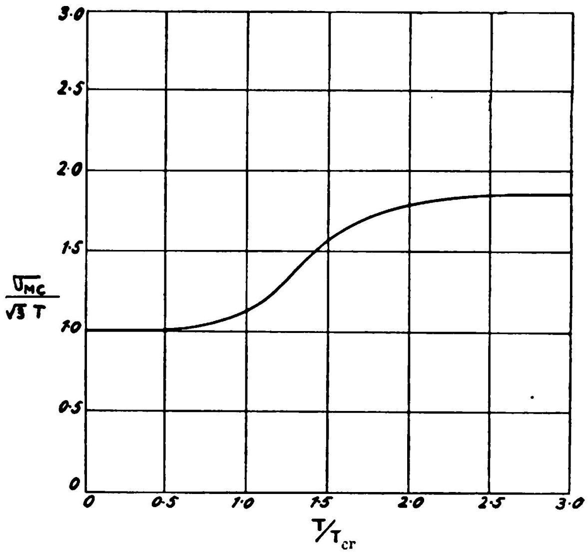

FIG. 4

Therefore, when designing webplates to operate in the immediate post-buckled range, the only additional requirement will be that of ensuring that the buckle patterns are not too prominent at working loads.

A small survey was, therefore, conducted in which the reaction of students to various depths of buckle formations was obtained. The survey showed that for values of the ratio w/w_{cr} up to 1.5:1, the stutends, who were not aware of the loading conditions, either failed to detect by visual examination the presence of buckles or thought that the depth of the buckles was less than the thickness of the webplate. It is therefore considered that there could be no objections on aesthetic grounds

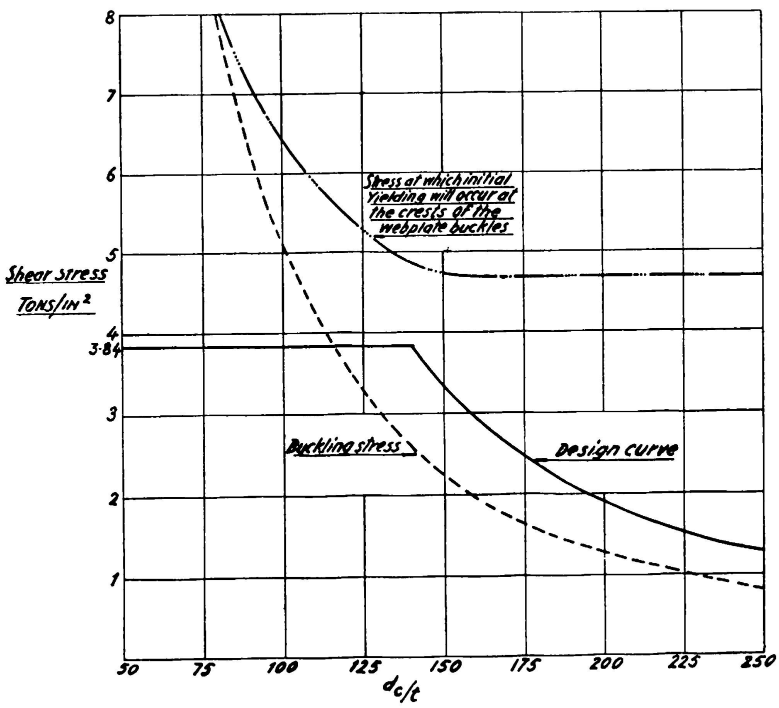

FIG. 5. Relationship for webplates reinforced by intermediate vertical stiffeners possessing a flexural rigidity EI equal to or greater than EI_L and spaced so that an effective aspect Ratio of one is obtained

to the use of webplates which had been loaded up to 1.5 times the buckling load.

3. The Behaviour of Webplates Subjected to Bending Stresses.

3. 1. *Background.*

The buckling and subsequent behaviour of webplates loaded by bending moments in the plane of the plate has not been studied to the same extent as buckling due to shear. Nevertheless, theoretical values for the buckling stress of initially plane rectangular plates have been obtained for a wide range of edge conditions and plate dimensions [1, 10, 11, 12].

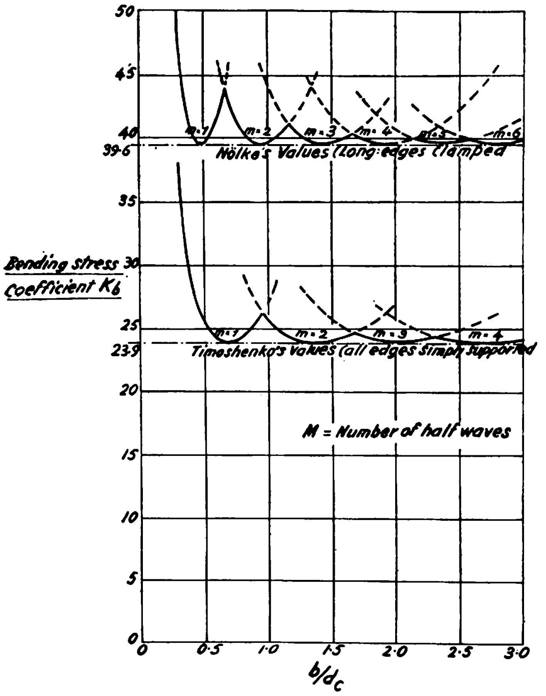

FIG. 6

The buckling stress σ_{cr} is given by equation (7) in which K_b is a coefficient, the value of which depends upon the superficial dimensions of the plate and the type of edge restraint, see figure 6.

$$\sigma_{cr} = \frac{K_b \pi^2 D}{d_c^2 t} \tag{7}$$

A number of investigators [13, 14, 15, 16] have conducted tests on webplates subjected to pure bending stresses, and although each investigation has provided interesting and sometimes new information, no conclusive results have been obtained to date.

3. 2. *The Buckling and Post-Buckled Behaviour of Webplates Subjected to Bending Stresses.*

Extensive tests on thin webplates subjected to pure bending stresses have been conducted by Jenkins [2] and the writer, in order to obtain further information on the buckling and post-buckled behaviour of such plates.

With reference to the buckling of the webplates, with those webplates which were initially plane, well defined buckling loads which were in reasonably close agreement with Nolke's [10] theoretical values were obtained.

In addition, it was found that when the initial webplate deformations were less than half the thickness of the webplate, the webplate would

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[2] One time Post-graduate student at the University College of Swansea.

buckle into a wave pattern similar to that predicted by theory [12], but that when the deformations were large, the buckle pattern was governed by the form of initial deformations.

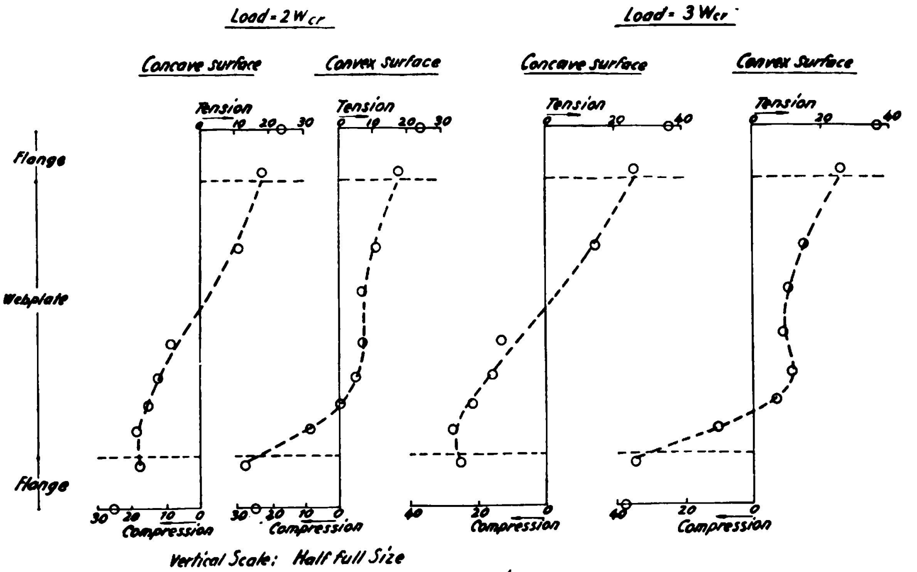

FIG. 7. Experimental results showing the distribution of strain along the vertical centreline of a panel

The surface strains of the webplates in the postbuckled range did not generally show agreement with the theoretical form [12]. In all cases, the stress in the webplate adjacent to the flanges exceeded the values at any other part of the webplate, as illustrated in figure 7.

4. The design of the webplates of plate girders constructed of high strength aluminium alloys possessing properties similar to H. 10. WP alloy (3).

In addition to determining new fundamental knowledge, one of the ultimate aims of engineering research is the improvement of design regulations. In what follows, a design procedure is developed using the evidence obtained from the investigations referred to in the previous sections.

In section 2.2, the relationships between the critical shear stress coefficient K of the stiffened plates and the non-dimensional parameter γ were discussed. With these empirical relationships it is possible to design a stiffened webplate so that it will buckle at a given stress. Now, tests have shown that stiffeners which possess a flexural rigidity EI equal to or greater than EI_L, will operate effectively when the webplate is loaded beyond the buckling load, providing the outstanding legs are not

(3) See British Standard (as front page).

too thin. It is therefore recommended that intermediate stiffeners shall possess an effective inertia I of not less than I_L, see equations (8) and (9).

Single Sided Stiffeners.

$$I_L = \left[1.97 \left(\frac{d_c}{b} \right)^2 - 0.7 \right] bt^3 \tag{8}$$

Double Sided Stiffeners.

$$I_L = \left[2.54 \left(\frac{d_c}{b_c} \right)^2 - 0.7 \right] bt^3 \tag{9}$$

For webplates reinforced by stiffeners having the above properties, the buckling stress of the stiffened panel is given by equation (3).

$$K = 7.0 + 5.6 \, (\alpha_e)^{-2} \tag{3}$$

Now, as stated in 2. 3. 2. it is considered that there are no objections on aesthetic grounds to the use of webplates loaded up to 1.5 times the buckling load. Therefore, the permissible design shear stress (T_{perm}) is given by equation (10).

$$T_{perm} = \frac{1.5 \, (7.0 + 5.6 \, (\alpha_e)^{-2}) \, \pi^2 E}{12 \, (1 - \mu^2)} \left(\frac{t}{d_c} \right)^2 < 3.84 \text{ tons/in}^2 \tag{10}$$

With regard to the design of panels subjected to pure bending, as as stated in section 3. 2, the maximum stresses in the unsupported webplate occur adjacent to the flanges. Therefore, since the flange stress is always greater than the webplate stress, the design of the flanges will ensure that at all times the stresses in the webplate do not exceed the safe permissible values. Therefore, when designing webplates subjected to pure bending stresses to operate beyond the buckling load, the only additional consideration, as was the case for shear buckling, will be that of ensuring that the buckle formations are not too prominent at working loads.

In determining the buckling stress, it will be assumed that the flanges provide a partial (50 %) clamped edge support to the webplate. Therefore, the permissible design stress (σ_{perm}) is given by equation (11)

$$\sigma_{perm} = \frac{1.5 \, (31.75) \, \pi^2 \, E}{12 \, (1 - \mu^2)} \left(\frac{t}{d_c} \right)^2 < 6.7 \text{ tons/in}^2 \tag{11}$$

When the maximum permissible bending stress of 6.7 tons/in² in the flange and the maximum permissible shear stress of 3.84 tons/in² in the webplate occur at the same section, a minimum factor of safety of 1.6 is obtained against yielding of the webplate adjacent to the flanges. This is considered quite satisfactory.

The stresses at which a plate subjected to combined shear and bending would buckle may be calculated by employing equation (12), where T_{cr} and σ_{cr} are the stresses at which the plate would buckle when subjected to pure shear and pure bending respectively

$$\left(\frac{\sigma}{\sigma_{cr}}\right)^2 + \left(\frac{T}{T_{cr}}\right)^2 = 1 \tag{12}$$

If, as was assumed in the cases of pure shear and pure bending the webplate can be loaded up to 1.5 times the buckling load, then the maximum permissible design stresses can be determined from equation (13)

$$\left(\frac{\sigma_{perm}}{\sigma_{cr}}\right)^2 + \left(\frac{T_{perm}}{T_{cr}}\right)^2 < 2.25 \tag{13}$$

where

$$\sigma_{cr} = \frac{31.75\,\pi^2 E}{12\,(1-\mu^2)} \left(\frac{t}{d_c}\right)^2$$

$$T_{cr} = \frac{(7.0 + 5.6\,(\alpha_c)^{-2})\,\pi^2 E}{12\,(1-\mu^2)} \left(\frac{t}{d_c}\right)^2$$

Limits

$$\sigma_{perm} < 6.7 \text{ tons/in}^2$$
$$T_{perm} < 3.84 \text{ tons/in}^2$$

On comparing the design stresses proposed above with the existing design stresses for structures constructed of high strength aluminium alloys [9, 17], it will be found that much higher working stresses are proposed. Since, in many forms of construction, the self weight of the structure is a considerable proportion of the load carried, it will be appreciated that by employing these higher working stresses considerable savings in materials and construction costs will be effected.

5. Conclusion.

Although the design procedure proposed in the previous section can only be used when designing plate girder webplates of high strength aluminium alloy, the basic experimental information on which the design procedure is based can be used to develop design procedures for structures to be constructed in the more ductile but less strong aluminium alloys.

ACKNOWLEDGEMENT

The investigations conducted by the writer into the behaviour of the webplates of light alloy plate girders were kindly sponsored by the Aluminium Development Association.

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SUMMARY

The paper summarises the results obtained from several hundred tests on webplates of model light alloy plate girders. The test programme involved tests on webplates subjected to pure shear, pure bending and combinations of shear and bending.

Electrical resistance strain gauges and dial gauges were used to determine the buckling load of the webplates and to study in detail the post-buckled behaviour of the webplates.

As a result of the tests conducted, new formulae for the design of intermediate vertical stiffeners and the economical design of light alloy webplates have been developed. The proposed design formulae recommend that the webplates of light alloy plate girders should, in certain instances, be designed to operate in the postbuckled range.

ZUSAMMENFASSUNG

Die Arbeit fasst die Ergebnisse zusammen, welche aus mehreren hundert Prüfungen an Stehblechen von Leichtmetallträgern ermittelt wurden. Das Prüfungsprogramm umfasste Versuche an Stehblechen, welche reinem Schub, reiner Biegung sowie Kombinationen von Schub und Biegung unterworfen waren.

Elektrische und mechanische Dehnungsmesser wurden benützt, um die Beullast der Stehbleche zu ermitteln und im einzelnen das überkritische Verhalten der Stehbleche zu studieren.

Als Ergebnis der durchgeführten Versuche wurden neue Formeln für die Berechnung der vertikalen Zwischenaussteifungen und für die wirtschaftliche Bemessug von Leichtmetallstehblechen entwickelt. Die vorgeschlagenen Berechnungsformeln empfehlen, Stehbleche von Leichtmetallträgern in gewissen Fällen für den überkritischen Bereich zu dimensionieren.

RESUMO

O autor resume os resultados obtidos em várias centenas de ensaios de almas de modelos de vigas em chapa de liga leve. O programa de ensaios incluía ensaios de almas submetidas ao corte simples, à flexão simples e ao corte e flexão combinados.

Empregaram-se flexómetros eléctricos e de mostrador para determinar a carga de encurvadura das almas e para estudar em pormenor o seu comportamento depois de encurvadas.

Com os resultados destes ensaios estabeleceram-se novas fórmulas para o cálculo de reforços verticais intermédios e para o cálculo económico de almas de ligas leves. Estas fórmulas indicam que, em certos casos, as almas de vigas leves devem-se calcular para actuarem encurvadas.

RÉSUMÉ

L'auteur résume les résultats obtenus au cours de plusieurs centaines d'essais sur les âmes de modèles de poutres en tôle d'alliage léger. Le programme comprenait des essais sur des âmes soumises au cisaillement simple, à la flexion simple et au cisaillement et flexion combinés.

Des flexomètres electriques et à cadran furent utilisés pour la détermination de la charge de voilement des âmes et pour l'étude détaillée de leur comportement après voilement.

Tirées des résultats de ces essais, l'auteur a établi de nouvelles formules pour le calcul de raidissements verticaux intermédiaires et pour le calcul économique des âmes en alliage léger. Ces formules conseillent dans certains cas de calculer les âmes des poutres en alliage léger après voilement.

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