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An Approximate Analysis of Stiffened Flanges

Une analyse approximative de plaques raidies

Näherungsverfahren zur Analyse von ausgesteiften Platten

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SUMMARY
An approximate method of analysing axially loaded stiffened plates is suggested. The loss of stiffness of the flange plate and of the outstand is allowed for by using the Energy Method. Empirical provision is made to account for the influence of residual stresses. Long panels are analysed as axially loaded individual singlespan struts using a modified form of the Perry-Robertson formula. The proposed method is calibrated against a large number of experimental results; design charts for various plate aspect ratios and stiffener sizes are proposed.

RÉSUMÉ
On présente une méthode approximative pour l'analyse de plaques raidies chargées axialement. La perte de rigidité de la plaque et des raidisseurs est prise en considération par l'utilisation de la méthode de l'énergie. Des mesures empiriques permettent de tenir compte de l'influence de contraintes résiduelles. A l'aide d'une version modifiée de la formule Perry-Robertson, de longs panneaux sont analysés comme des poteaux à travée unique et axialement chargés. La méthode proposée est étalonnée par rapport à un grand nombre de résultats expérimentaux; des abaques de calcul sont proposés pour différents rapports de dimensions des plaques et des raidisseurs.

ZUSSAMMENFASSUNG
Für die Analyse ausgesteifter Platten, die axial belastet sind, soll ein Näherungsverfahren vorgeschlagen werden. Dabei wird unter Anwendung der Energiemethode die Abminderung der Steifigkeit der Flansche und der Steifen in genügendem Masse berücksichtigt. Aufgrund eines empirischen Verfahrens können die Eigenspannungen ermittelt werden. Eine modifizierte Form der Perry-Robertson-Gleichung erlaubt lange Platten durch axial belastete, voneinander unabhängige Einfeldträger zu ersetzen. Das vorliegende Berechnungsverfahren stützt sich auf eine grosse Anzahl Versuchsergebnisse. Bemessungstabeln für verschiedene Platten- und Steifenverhältnisse sind in diesem Artikel vorgeschlagen.

1. INTRODUCTION

Longitudinally stiffened plates are used in the flanges of box girders and in the double bottom grillages of ships. In the past the design of such plates was based on empirical methods |1|. The collapse of a few box girders during erection in recent years triggered extensive research on the various design aspects of these structures |2-6|. In Great Britain, this research largely centred around the Merrison Committee which inquired into the collapse of Milford Haven Bridge |7|.

Compression tests have been carried out on stiffened plates of realistic sizes at Manchester University, Monash University, Polytechnic of Central London and elsewhere |8-12|. Semi-empirical methods for the prediction of collapse loads of stiffened plates based on these tests have been proposed |13-16|. These methods differ considerably in approach and it is, therefore, not surprising that significant variations exist with respect to the general tendency of the various methods either to overestimate or to underestimate the failure loads |6,16|. On the other hand, although most of the analytical models would give excellent results, the demands made on the computer time tend to be excessive in most cases.

The analytical treatment proposed by Horne and Narayanan |17| is based on the elastic post-buckling characteristics of an initially deformed plate simply supported against out of plane displacements but restrained along the edges against in-plane displacements. Only the loss of stiffness of the plate was considered and it was assumed that the stiffener was sufficiently stocky to prevent the occurrence of stiffener-initiated collapse. This is not a reasonable proposition for panels collapsing by the onset of instability in the stiffener. This paper is a natural extension and includes a provision for the buckling of the outstand.

2. MODES OF COLLAPSE OF AN AXIALLY LOADED STIFFENED PLATE

The collapse of an axially loaded stiffened plate may be initiated by the failure of the plate or by the failure of the stiffeners. In either case, there is an interaction between the local buckling of the plate or the stiffener and the buckling of the panel as a whole, i.e. the so-called "overall" or column buckling of the panel. The term "plate failure" indicates that the yielding in the plate is sufficiently extensive to trigger the failure of the panel. On the other hand, when a long stiffened plate is axially loaded, the tip of the stiffener could yield either in tension or by compressive buckling. The corresponding collapse modes are termed "stiffener-tension failure" and "stiffener compression failure" respectively. In the former case, the panel bows towards the edges of the stiffener, while in the latter, towards the plate. All these cases are sketched in Fig. 1.

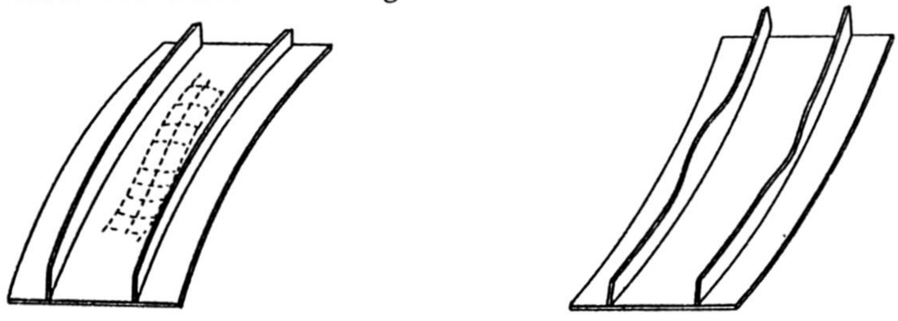

Plate failure and stiffener-tension failure

Stiffener compression failure

FIG. 1

3. DEFINITION OF TERMS USED

3.1. Plate Imperfection (Δ_x) is the maximum out-of-plane imperfection of the plate measured over a gauge length equal to the spacing of the stiffeners (b).

3.2. Stiffener Imperfection (Δ_{sy}) is the maximum torsional imperfection of the stiffener parallel to the plate measured, at the tip of the outstand in accordance with the Merrison Rules and the draft British Standards |18|.

3.3. Slenderness Ratio (ℓ/r) refers to the ratio of the free bending length of the panel to the radius of gyration of a single stiffener and its associated gross width of plating about its centroidal axis.

3.4. Squash Load (P_{sq}) is computed by adding the yield stress of the stiffener (σ_{yso}) times the area of the stiffener (A_s) to the yield stress of the plate (σ_{ysp}) times the area of the plate (b.t).

3.5. Ultimate Load (P_u) refers to the maximum load sustained by the panel.

3.6. Crushing Load (P_{as}) refers to the load at collapse of a stiffened plate having a very low slenderness (ℓ/r) value.

4. ASSUMPTIONS

The material of the plate is taken to be homogeneous, isotropic, elastic and then perfectly plastic and the effect of strain hardening is negligible. All the edges of the plate are taken to be held straight in plane and out of plane but are free from restraining or applied moments. Membrane shear stresses on planes parallel to the edges of the plate panel are assumed to be zero. The number and length of half waves is assumed to be the same in post-buckled stage as at incipient buckling. The buckling shape in the post-buckled stage is sinusoidal. The stiffened panel is assumed to be wide enough for it to be treated as a pin-ended column and the orthotropic plate action of the stiffened panel is not considered. The analysis of stiffened panel is carried out on a single longitudinal stiffener and its associated plate. The panel is assumed to be stiffened only by plain flat stiffeners. The analysis is not valid for other types of stiffeners.

5. AXIALLY LOADED PLATES WITH INITIAL IMPERFECTIONS

The elastic load-shortening behaviour of a simply supported plate with initial imperfections under uniform axial load was investigated by Horne and Narayanan |17| (Fig. 2). If the longitudinal edges of such a plate are kept straight, the load-shortening behaviour was shown to be described by the following equations :

$$\sigma_m = E\left|\varepsilon_x - \left(\frac{m^2-1}{2}\right)\frac{\pi^2 A_o}{4b^2}\right| \qquad (1)$$

and

$$\varepsilon_x = \frac{m-1}{m} \cdot \frac{\sigma_{cr}}{E} + (m^2-1)\frac{\pi^2 A_o}{4b^2} \qquad (2)$$

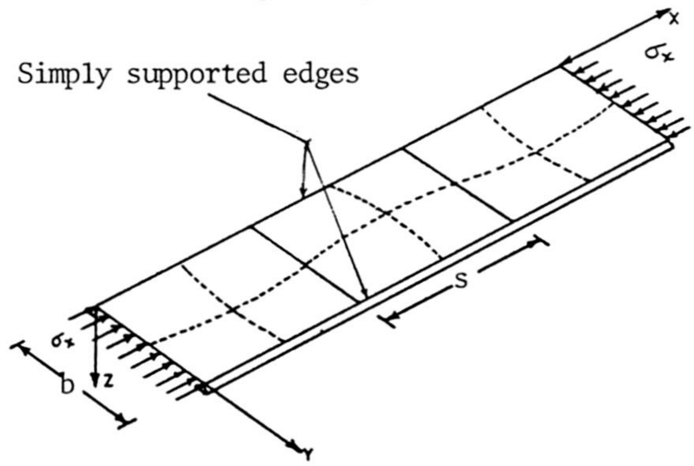

FIG. 2: Simply supported plate with axial loading

where

σ_m = mean longitudinal compressive stress in the x direction due to an axial strain of ε_x
m = maximum plate deflection under a mean stress of σ_m / A_o
A_o = amplitude of the buckling wave
b = half wave length of the buckle
E = Modulus of Elasticity
σ_{cr} = Elastic critical stress for the plate

The theoretical elastic critical stress of an ideally flat plate simply supported along the edges and loaded uniformly in one direction is given by |19|

$$\sigma_{cr} = \frac{k\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b}\right)^2 \qquad (3)$$

where

t = thickness of the plate
k = a constant depending upon the support conditions and the aspect ratio (a/b)
a = length of the plate in the direction of applied stress
ν = Poisson's ratio

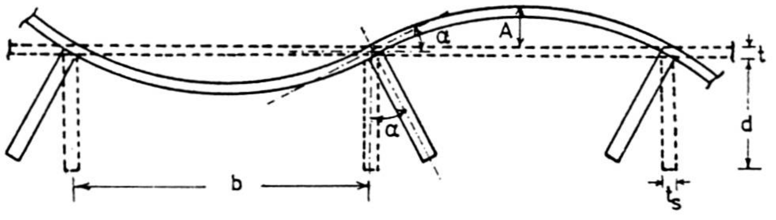

FIG.3: Cross-section of stiffened panel

The bed plates of the stiffened panels (Fig.3) have boundary conditions as described above. However, the stiffeners are also axially loaded plates having one of the edges held straight and the other edge free to wave. These outstands have initial imperfections, which also increase due to the application of an axial load. This growth is partly contributed by the rotation of the stiffener when the plate imperfections grow, as the panel is compressed. The analytical treatment given below of an initially imperfect outstand follows the same general approach as that of Horne and Narayanan |17|.

The outstand ABCD in Fig.4 is initially flat and the transverse edges AB and CD in the co-ordinate direction z, kept straight in plane by cross girders, are induced by the applied load to approach by a uniform displacement of ε_x. The longitudinal edge BC in the co-ordinate direction x is simply supported against out of plane displacement but restrained in plane; it is also free to rotate about BC and corresponds to the plate/stiffener junction line. The edge AD is free to wave. Had the outstand ABCD been an initially flat plate, it would have remained straight under axial loading until buckling took place. For a given value of ε_x, the induced longitudinal stress σ_x would remain uniform until

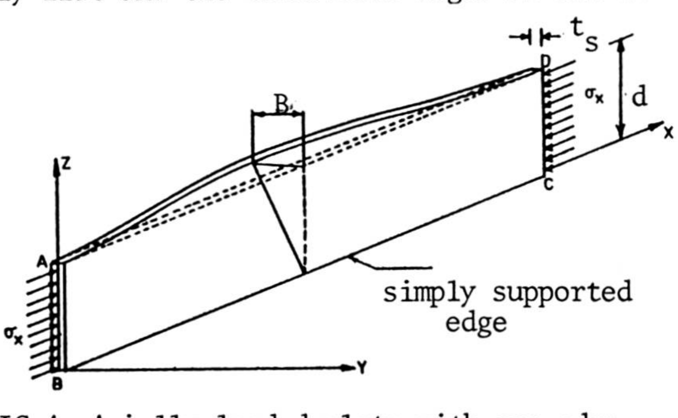

FIG.4: Axially loaded plate with one edge free to wave

it reaches the elastic critical stress σ_{cs}, when the outstand buckles in a series of half wave lengths, s. The theoretical analysis of an initially flat plate can, however, give only an upper bound solution. It will be assumed that the initial surface of the outstand before any loads are applied can be described by

$$v_o = \left(\frac{B_o}{d}\right) \text{Sin } (\pi x/s) \cdot z \qquad (4)$$

where, B_o = initial out of plane imperfections of the outstand
s = half wave length of the buckle
v_o = initial deflection function
d = width of the outstand in z direction

The growth of out-of-plane imperfection of the stiffener is influenced by the loading on AB and DC as well as the rotation of BC induced by the growth of (Δ_x) imperfections in the bed plate. The buckled shape of the free edge AD can be described by

$$v = \left(\frac{B}{d} + \alpha\right) \text{Sin } (\pi x/s) \cdot z \qquad (5)$$

where, B = amplitude of buckling
α = angle of rotation of the stiffener when the (Δ_x) imperfection of the bed plate increases

The bending energy stored in a half wave length of buckle for a flat plate is

$$U_b = \frac{t_s}{2} \int_o^d \int_o^s \sigma_{cs} \left(\frac{dv}{dx}\right)^2 dx\, dz$$

$$= \left(\frac{B}{d} + \alpha\right)^2 \kappa \qquad (6)$$

$$\text{where } \kappa = \frac{\pi^2 t_s}{4s} \sigma_{cs} \cdot \frac{d^3}{3} \qquad (7)$$

t_s = thickness of the outstand

The work done by a small increment in B from B to (B + δB) for a plate with an initial imperfection of B_o is obtained from

$$\delta U_b = 2 \left(\frac{B - B_o}{d} + \alpha\right) \frac{\delta B}{d} \cdot \kappa$$

The energy U_s due to strain in the mid plane of the plate is obtained from

$$U_s = \int_o^d \int_o^s \frac{\sigma_x^2}{2E} t_s \cdot dx \cdot dz = \frac{t_s \cdot s}{2E} \int_o^d \sigma_x^2\, dz$$

$$\delta U_s = \frac{t_s \cdot s}{E} \int_o^d \sigma_x \cdot d\sigma_x\, dz$$

The total work done by a small increment in B is given by

$$\delta U = \delta U_b + \delta U_s = 2 \left(\frac{B-B_o}{d} + \alpha\right) \frac{\delta B}{d} \cdot \kappa + \frac{t_s s}{E} \int_o^d \sigma_x \cdot d\sigma_x \cdot dz \qquad (8)$$

The change in the applied strain may be expressed as the sum of change due to $\delta\sigma_x$ and that due to flexural shortening.

$$\delta\varepsilon_x = \frac{\delta\sigma_x}{E} + \frac{\delta u}{s}$$

where the flexural shortening is obtained from

$$u = \frac{1}{2}\int_0^s \left(\frac{dv}{dx}\right)^2 dx ,$$

$$\delta\varepsilon_x = \frac{d\sigma_x}{E} + \frac{\pi^2}{2s^2}\,(B + \alpha d)\;\;\delta B \,.\, \frac{z^2}{d^2} \qquad (9)$$

Over the length s of the outstand, the shortening is $s.d\varepsilon_x$ and the external work done by the applied load is given by

$$\delta T = \frac{s.t_s}{E}\int_0^d \sigma_x \,.\, d\sigma_x \,.\, dz + \frac{(B + \alpha d)\;\delta B\;\pi^2\;t_s}{2s\;d^2}\int_0^d \sigma_x\; z^2\; dz \qquad (10)$$

Since $\delta U = \delta T$, we obtain from (8) and (10) ,

$$(B + \alpha d)\int_0^d (\sigma_{cs} - \sigma_x)\; z^2\; dz = B_o \,.\, \sigma_{cs} \,.\, \frac{d^3}{3} ,$$

$$\left(m_s + \frac{\alpha d}{B_o}\right)\int_0^d (\sigma_{cs} - \sigma_x)\; z^2\; dz = \sigma_{cs}\,\frac{d^3}{3} \qquad (11)$$

where, $m_s = B/B_o$

Differentiating (11),

$$dm_s\int_0^d (\sigma_{cs} - \sigma_x)\; z^2\; dz = \left(m_s + \frac{\alpha d}{B_o}\right)\int_0^d d\sigma_x\;\; z^2\; dz \qquad (12)$$

Substituting for $d\sigma_x$ from (9) and after performing the integrations we obtain

$$d\varepsilon_x = \frac{B_o^2 \, dm_s \cdot \sigma_{cs}}{E(m_s B_o + \alpha d)^2} + (m_s B_o + \alpha d) \frac{3\pi^2}{10 s^2} dm_s B_o \qquad (13)$$

ε_x is evaluated by integrating (13) and substituting $m_s=1$ when $\varepsilon_x=0$ and $\alpha=0$.

$$\varepsilon_x = \frac{\sigma_{cs}}{E} \cdot \frac{(m_s-1)B_o+\alpha d}{m_s B_o + \alpha d} + \frac{3\pi^2}{20 s^2} |(m_s+1)B_o+\alpha d|\,|(m_s-1)B_o+\alpha d| \qquad (14)$$

σ_x is obtained by integrating equation (9)

$$\sigma_x = E\{\varepsilon_x - \frac{\pi^2}{4s^2} |(m_s+1)B_o+\alpha d| . |(m_s-1)B_o+\alpha d| \, |\tfrac{z}{d}|^2 \} \qquad (15)$$

These equations are similar in form to those derived by Horne and Narayanan |17| for plates simply supported at the edges.

Since the outstands (free at one edge) have no reserve strength, collapse could be expected to occur as soon as yield sets in. The ultimate load of such a plate is, therefore, the load causing the onset of yield at its middle surface at the supported edge.

6. APPLICATION TO STRENGTHS OF PLAIN OUTSTANDS

In order to test the validity of the above failure criterion, the method was applied to predict the strengths of a number of flat plate oustands tested by Rogers |20| at the University of Cambridge. He carried out tests on 58 such plates under axial loading. Each test specimen consisted of a single plate supported along its centre-line, effectively producing two hinged plain outstands; the load was applied axially at the two ends, using a specially fabricated rig. The aspect ratio d/t_s varied from 8.8 to 28.4. All the panels tested were short, so that no column buckling took place. Full details of the tests are given in Reference |20|.

In computing the predicted values using the above treatment, the amplitude and half wave lengths measured by Rogers were used. The mean value of the ratio of predicted to observed load was found to be 1.00, with a standard deviation of 0.052. In 21 cases the ratio of predicted to observed load exceeded 1.00. All predictions were within +6%. This seems to show that the failure criterion used is a reasonable one.

7. THE INFLUENCE OF WELDING SHRINKAGE STRESSES

The experiments carried out by Horne and Narayanan |12,21| have shown that residual stresses due to welding have no influence on the ultimate strengths of practical panels having plate aspect ratios (b/t) of 30 or less. As these tests were carried out on plates having yield stress of 350 N/mm^2, the corresponding aspect ratios for other steels may be taken as 30 $\sqrt{\sigma_{ys}/350}$. A comparison of ultimate strengths of similar panels has shown that panels with high residual stresses did not necessarily exhibit lower strengths compared with similar panels with low residual stresses. Thus even for larger values of $^b/t$, the influence of residual stresses can be adequately accounted for in panels of realistic proportions and realistic imperfections if the applied load stresses everywhere are restricted to 90% of the yield stress of the material. While computing the peak load on a stiffened plate it will be assumed that the ultimate load is reached when the maximum stress in the plate/stiffener junction reaches 90% of the yield stress, when $^b/t > 30\sqrt{\sigma_{ys}/350}$. For lower values of $(^b/t)$ no reduction in the yield stress will be made.

8. APPLICATION TO SHORT STIFFENED PANELS TESTED IN A FIXED ENDED CONDITION

The analysis described in the foregoing pages was applied to the results of 21 fixed ended tests on stiffened panels carried out at Manchester University. The tests covered a wide range of values in $^b/t$, $^d/t_s$, $^\ell/r$ and residual stresses. The detailed description of these tests will be found in references |11| and |12|. A set of typical predicted results using the above treatment and the corresponding experimentally observed values are tabulated in Table I. The measured values for the plate and stiffener imperfections were used in the comparison. The predicted values are shown to be sufficiently accurate in all cases.

TABLE I : Fixed ended Tests

Values in Columns (8) and (9) are in terms of ultimate load/squash load

b mm (1)	t mm (2)	d mm (3)	t_s mm (4)	$^b/_t$ (5)	$\frac{d}{t_s}$ (6)	$\frac{\ell}{r}$ (7)	observed strength (8)	predicted strength (9)	$\frac{\text{predicted}}{\text{observed}}$ (10)
Plate Failure Tests:									
457	9.5	152.5	16.0	48	9.5	19	0.92	0.87	0.95
457	6.5	152.5	9.5	70	16.0	20	0.62	0.66	1.07
200	9.7	150.0	15.2	21	10.0	18	1.04	0.87	0.87
300	10.0	150.0	15.2	30	10.0	18	1.00	0.95	0.95
350	9.8	150.0	15.2	36	10.0	19	0.93	0.87	0.93
480	9.7	150.0	15.1	49	10.0	20	0.75	0.71	0.95
Stiffener Compression Failure Tests:									
200	10.1	148.5	9.8	20	15.0	25	0.85	0.84	0.99
200	10.0	148.5	9.7	20	15.0	25	0.98	0.91	0.92
457	9.5	152.5	9.5	48	16.0	22	0.95	0.91	0.96

Total number of panels tested = 21
Mean value of predicted/observed load = 0.96
Standard deviation = 0.059
Number of cases where predicted to observed load exceeded 1.00 = 4
The maximum value of unconservative prediction = 1.07

9. COLLAPSE LOAD OF LONG PANELS

The collapse load of an axially loaded long column, having an initial imperfection of Δ can be computed by using the criterion of first yield occurring at the outer faces of the column :

$$\sigma_{ys} = \sigma_a + \frac{\sigma_a \cdot \sigma_e}{\sigma_e - \sigma_a} \cdot \frac{a\,\Delta\,.\,A}{I} \qquad (16)$$

where σ_{ys} = yield stress
σ_a = mean stress causing failure
σ_e = Euler stress
A = Area of the column
I = Second moment of area of the column
a = distance of the extreme fibre from the centroid of the column

On the basis of a large number of tests on struts, Robertson |22| suggested that the term $a\Delta A/I = \eta$ can be effectively approximated by $(0.003\ \ell/r)$. With the above form of the imperfection constant, η, the mean failure stress σ_a falls below the yield stress even for very small values of slenderness ratios (ℓ/r). This is, however, not observed in practice. To overcome this objection, Dwight |23| has suggested that the imperfection factor be reduced by a correction factor $(\ell-\ell_o)/\ell$ in which $\ell_o = 0.2\ \pi\ r\ \sqrt{(E/\sigma_{ys})}$ when $\ell > \ell_o$ and $\ell_o = \ell$ when $\ell < \ell_o$.

The above procedure can be extended to the computation of collapse loads of stiffened panels. First, the maximum load that can be applied to one buckle length of the plate - the so-called "crushing load" - is computed. Obviously this is the maximum load that can be sustained by a panel having a very low slenderness (ℓ/r) value. Let the corresponding mean stress be σ_{as}. The term $(a\Delta A/I)$ can be replaced by $0.003\ (\ell-\ell_o)/r$ for all practical cases of stiffened plating, ship grillages etc. Thus equation (16) may be modified as

$$\sigma_{as} = \sigma_a + 0.003\ \left(\frac{\ell - \ell_o}{r}\right) \left(\frac{\sigma_a \cdot \sigma_e}{\sigma_e - \sigma_a}\right) \qquad (17)$$

For a panel of given dimensions and values of imperfections of plate and stiffener, the mean stress at collapse for a corresponding short panel (σ_{as}) can be evaluated by using the above theory. For a long panel of known length the global mean stress failure (σ_a) can then be calculated by solving equation (17).

The validity of this treatment was checked using the Manchester tests |11,12|. Typical predictions using the above approach and the experimentally observed values are listed in Table II. In obtaining these predictions the measured values of plate and stiffener imperfections were used. It will be seen that in all the cases the predictions are sufficiently accurate.

10. DESIGN AIDS

The treatment described in the foregoing pages is seen to be adequate and sufficiently quick in computational effort. For the use of designers, charts have been prepared for predicting the value of σ_{as}, the mean stress at failure for a very short panel for various values of A_s/bt and for $d/t_s < 12.0$. These are given in Fig. 5. The imperfections used in preparing these charts are the same as the Δ_x and Δ_{sy} tolerances specified in the new B.S. Bridge Code. |18|.

For long panels, the mean stress at collapse can be computed using equation (17) or can be read off from Fig. 6.

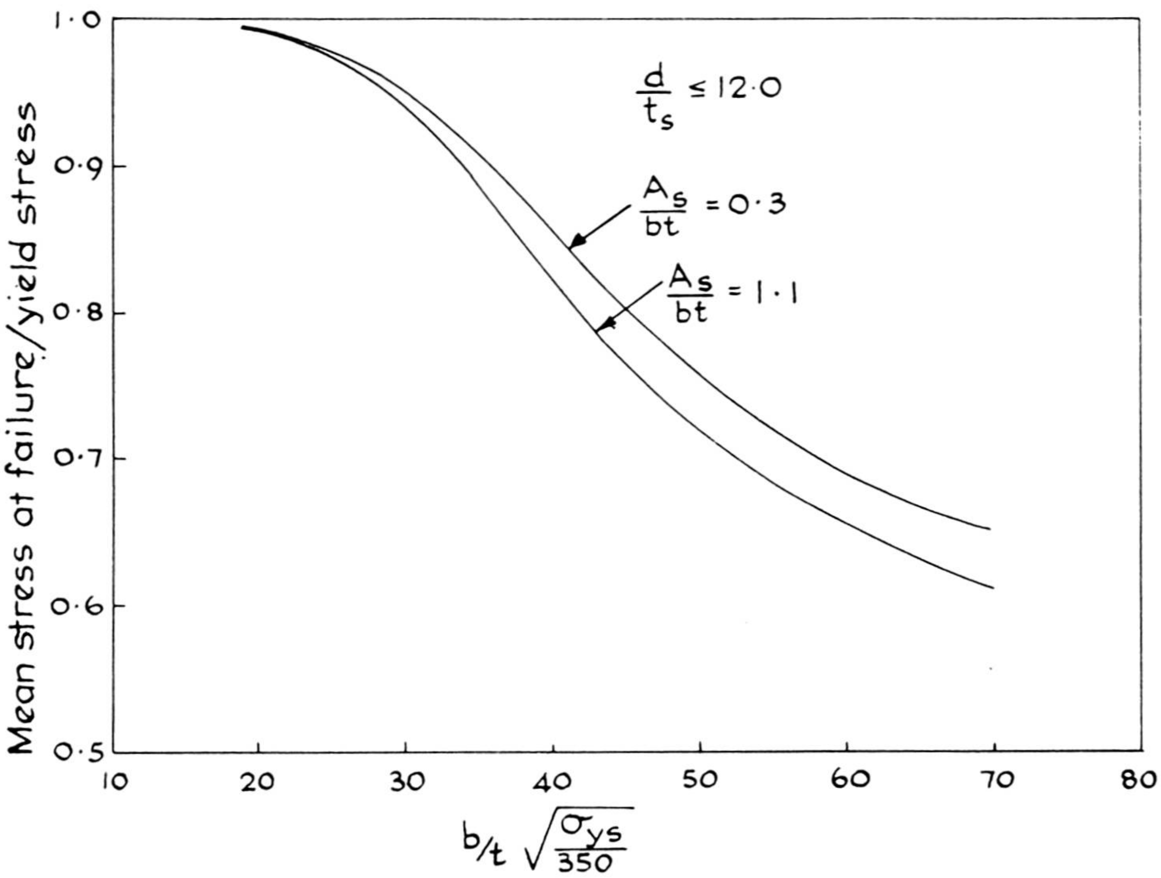

FIG. 5 : Mean stress at failure for a short panel

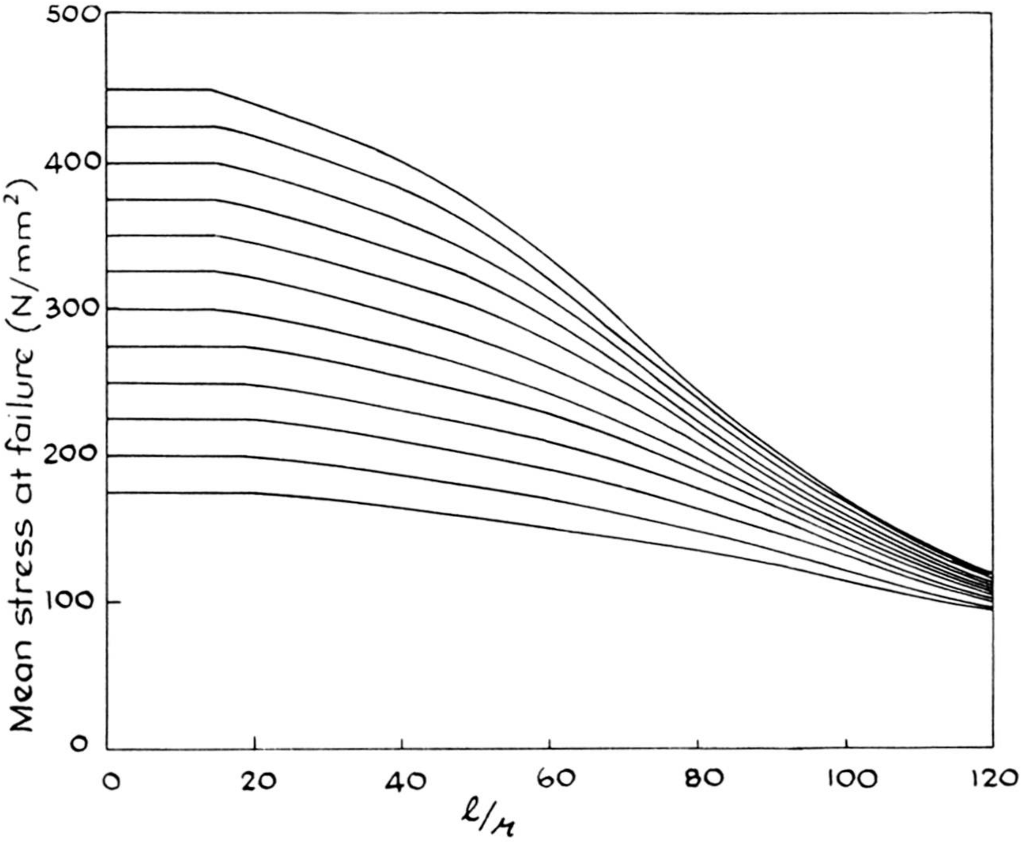

FIG. 6 : Slenderness Ratio vs. mean stress at failure

TABLE II: Pin-ended Tests

(Values in Columns (8) and (9) are in terms of ultimate load/squash load)

b mm	t mm	d mm	t_s mm	$\frac{b}{t}$	$\frac{d}{t_s}$	$\frac{\ell}{r}$	observed strength	predicted strength	$\frac{\text{predicted}}{\text{observed}}$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
Plate failure tests:									
457	9.5	152.5	16.0	48	9.5	39	0.79	0.77	0.98
457	6.5	152.5	9.5	70	16	41	0.61	0.57	0.94
350	9.8	150.0	15.2	36	10	57	0.72	0.68	0.95
300	10.0	150.0	15.2	30	10	55	0.79	0.78	0.99
200	9.7	150.0	15.2	21	10	53	0.78	0.77	0.97
Stiffener Compression failure tests:									
457	9.5	152.5	9.5	48	16	44	0.78	0.80	1.02
200	9.9	148.5	9.8	20	15	62	0.75	0.67	0.90
200	9.9	148.5	9.8	20	15	62	0.67	0.69	1.03
Stiffener Tension failure tests:									
457	10.0	80.0	12.0	46	7	93	0.60	0.55	0.92
457	10.0	80.0	12.0	46	7	93	0.65	0.54	0.83
457	6.5	76.0	12.5	70	6	88	0.45	0.43	0.97

Total number of panels tested = 32
Mean value of predicted/observed load = 0.94
Standard Deviation = 0.074
The number of cases where predicted/observed strength exceeded 1.00 = 5
The maximum value of unconservative prediction = 1.06

11. CONCLUSIONS

The elastic load-shortening behaviour of an axially loaded stiffened plate has been investigated. Allowance has been made for plate imperfections and stiffener imperfections in the study of post-buckling behaviour. Residual stresses have been allowed for by a 10% reduction in yield stress of plates having $(^b/t) > 30\sqrt{\sigma_{ys}/350})$. A modified form of Perry Robertson formula with an imperfection constant of $\eta = 0.003$ is shown to be adequate to predict the ultimate loads of stiffened plates. Charts for the use of designers have been proposed.

REFERENCES

1. Specification for Steel Girder Bridges, British Standard 153, Parts 3 and 4, British Standards Institution, London, 1958.
2. MASSONNET, C. and MAQUOI, R.: New Theory and tests, on the Ultimate Strength of stiffened box girders, Proceedings of the International Conference on Steel box girder bridges, Institution of Civil Engineers, London, 1973, pages 131-143.
3. WALKER, A.C. and MURRAY, N.W.: Analysis for Stiffened plate panel buckling Civil Engineering Research Report No. 2, Monash University, Australia, Sept, 1974.
4. CRISFIELD, M.A.: Full range analysis of steel plates and stiffened plating under uniaxial compression, Proceedings of the Institution of Civil Engineers, London, Vol. 59, December 1975, pp. 595-624.

5. LITTLE, G.H.: Stiffened Steel Compression panels - Theoretical Failure Analysis, The Structural Engineer, Vol. 54, No. 12, December, 1976, pages 489-500.
6. ECCS Task Group 8/3: Design of box girder flanges, Manual on Stability of Steel Structures, European Convention for Constructional Steelwork, 1976.
7. MERRISON COMMITTEE: Interim Design and Workmanship Rules, Committee of Inquiry into the Basis of Design and Method of Erection of Steel Box Girders, Her Majesty's Stationery Office, London 1973.
8. DORMAN, A. and DWIGHT, J.B.: Tests on Stiffened Compression panels and plate panels, Proceedings of the International Conference on Steel Box Girder Bridges, Institution of Civil Engineers, London 1973, pages 63-75.
9. MURRAY, N.W.: Buckling of Stiffened panels loaded axially and in bending. The Structural Engineer, London, Vol. 51, August 1973, pages 285-300.
10. WOLFHART, A. et al: Buckling Tests with bottom plates of steel box girders - Der Bauingenieur, 1974, 49 (10), pages 373-382.
11. HORNE, M.R. and NARAYANAN, R.: Ultimate Capacity of Stiffened Plates used in box girders, Proceedings of the Institution of Civil Engineers. London, Vol. 61, June, 1976, pages 253-280.
12. HORNE, M.R., MONTAGUE, P. and NARAYANAN, R.: "Influence on Strength of Compression panels of stiffener section, spacing and welded connection. Proceedings of the Institution of Civil Engineers, London. Vol. 63, No. 2, March, 1977, pages 1-20.
13. MURRAY, N.W.: Analysis and Design of Stiffened Plates for Collapse Load, The Structural Engineer, London, Volume 53, March 1975, pages 153-158.
14. CHATTERJEE, S. and DOWLING, P.J.: The design of Box Girder Compression Flanges, paper presented at the International Conference on Steel Plated Structures held at Imperial College, London, July, 1976.
15. DWIGHT, J.B. and LITTLE, G.H.: Stiffened Steel Compression Flanges - a simpler approach, The Structural Engineer, London, Vol. 54, No. 12, December 1976, pages 501-509.
16. HORNE, M.R. and NARAYANAN, R.: Design of axially loaded stiffened plates, Journal of the Structural Division, American Society of Civil Engineers, Vol. 103, No. ST 11, Nov., 1977, pages 2243-2257.
17. HORNE, M.R. and NARAYANAN, R.: Strength of axially loaded stiffened panels, Memoires of the International Association for Bridge and Structural Engineering, Zurich, Vol. 36-I, March, 1976, pages 125-157.
18. BRITISH STANDARDS INSTITUTION: Draft Code of Practice on Steel Concrete and Composite Bridges, Part 6, British Standards Institution, London, 1978.
19. TIMOSHENKO, S. and GERE, J.: Theory of Elastic Stability, McGraw Hill, 2nd Edition, 1961.
20. ROGERS, N.A.: Compression tests on plain flat outstands, Report No. CUED/C-Struct/TR 52 (1976), University of Cambridge, 1976.
21. NARAYANAN, R.: The influence of residual shrinkage stresses on the ultimate strengths of stiffened steel flanges, paper presented at the International Conference on Residual Stresses in Welded Construction and their effects, Welding Institute, London, Nov. 1977.
22. ROBERTSON, A.: The Strength of Struts, Selected Engineering paper No. 28, Institution of Civil Engineers, London, 1925.
23. DWIGHT, J.B.: Use of Perry Formula to represent the new European Strut curves Report No. CUED/C-Struct/TR 30 (1972), University of Cambridge, 1972.