The behaviour of thin stiffened steel plates

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The Behaviour of Thin Stiffened Steel Plates

Le comportement de plaques minces raidies en acier

Das Verhalten dünner ausgesteifter Stahlplatten

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Introduction

The maximum load-carrying capacity of a structure is often used as the sole basis for its design. This is quite satisfactory for the design of, say, a conventional steel or reinforced concrete beam but for certain structures there is a rapid deterioration of load-carrying capacity when they start to fail. A typical example is the mild-steel pin-ended strut of rectangular cross-section and Fig. 1 shows that once the maximum (failure) load P_F is obtained the load-carrying capacity diminishes. If the applied load is increased to P_F and held constant during and beyond failure (as is the case when P is a dead load) the difference between P_F and the load-carrying capacity is a force which accelerates the elements of the strut. The larger the deflection Δ the smaller

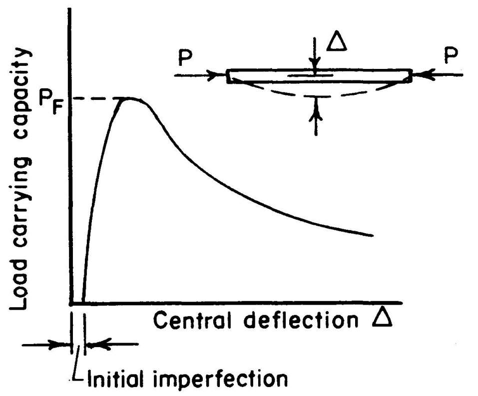


Fig. 1. Load-carrying capacity of a rectangular steel strut.

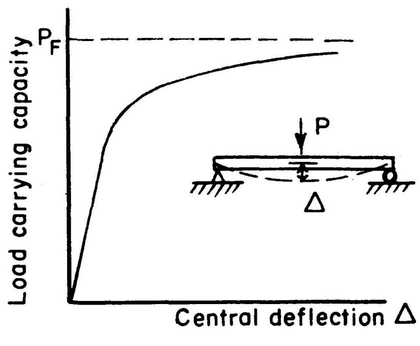


Fig. 2. Load-carrying capacity of a rectangular steel beam.

is the load-carrying capacity and the greater is the accelerating force. When the same member is used as a simply supported beam and P is applied laterally, the curve of load-carrying capacity against deflection Δ is a curve which rises, flattens out and becomes horizontal (Fig. 2). In this case failure is much slower because the acceleration force is zero or very small.

For an isolated strut once an applied dead load reaches the failure load P_F (Fig. 1) failure must occur whether the curve beyond F drops away gradually or suddenly. However, if some means is used to prevent complete collapse the strut is still capable of carrying some share of the load. This means may, in fact, be some adjacent member in a structural system. Another way of using post-buckling strength is to provide an alternative load path. Most air travellers have seen the skin of the wings of their aircraft buckling and it is comforting to those who are aware of it that there is, hidden from the eyes of the public, an alternative load path through the stringers. In this way the wings are able to carry loads quite safely even though part of the structure has buckled. The wings are a special type of box girder, the skin being the deck and the stringers are the webs. In the case of box girders used in modern bridges the webs tend to be much weaker structurally than the deck and once the deck fails almost no alternative load path is provided. The comparison in Fig. 3 illustrates this point. A designer would certainly wish to apply a higher load factor to the girder with the thinner webs.

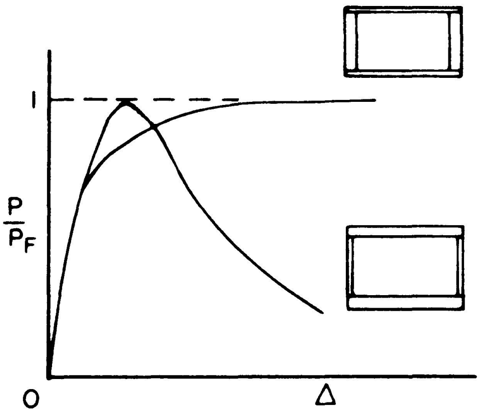


Fig. 3. Shapes of carrying capacity curves of two box-girder beams.

The rate at which the curve of load-carrying capacity of a structure drops away after the maximum load is some measure of its "toughness" *) or its ability to withstand a limited amount of overload and abuse. Design philosophy should have something to say about toughness but the concept is not a simple one. As has been discussed above it is seen that an individual component, e. g., the skin of an aircraft wing, may lack toughness but the structure as a whole is tough because alternative tough load paths are provided.

There are many other structures whose toughness is of interest. Most thin-

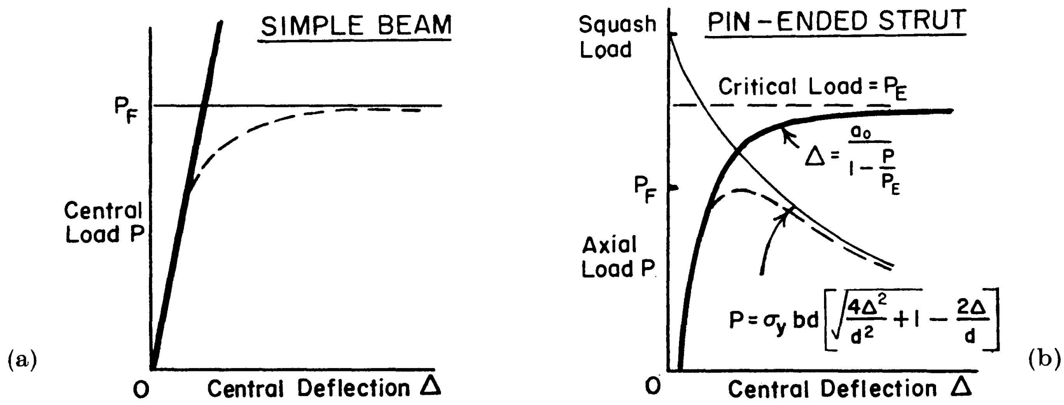
*) The Author is indebted to Professor L. K. Stevens for suggesting this term.

walled sections rapidly unload once their maximum capacity is reached and one example of this is a thin plate to which is attached thin stiffeners.

Following the collapse of West Gate Bridge in Melbourne a number of its components were tested and analysed theoretically. Because of their thin-walled nature some components exhibited a rapid drop off in load-carrying capacity. This report briefly describes the theoretical work and the tests carried out and discusses the importance of post-failure behaviour and initial imperfections especially in the case of thin-walled structures.

Theoretical Analysis of Elasto-Plastic Buckling

There are several ways of analysing the buckling behaviour of a metal structure when some plastic regions exist. The work of BAKER, HORNE, RODERICK and others [1] traced out the plastic zones as they developed in mild steel structures. For aluminium structures the tangent-modulus and double-modulus theories were developed. Another way of analysing the buckling of mild steel structures is described in MATHESON's book [2]. It is this last method which is used here. Even though it is only approximate the analysis of quite complicated structures can be carried out with reasonable accuracy and it allows one to gain an impression of the toughness of a structure. The behaviour of a mild steel structure can be described by first assuming that it behaves entirely in an elastic manner and then by assuming that it behaves in a rigid-plastic manner. Fig. 4a illustrates the concept applied to a simply supported beam of rectangular cross-section and Fig. 4b shows how it can be used to predict the behaviour of a pin-ended strut of rectangular cross section.

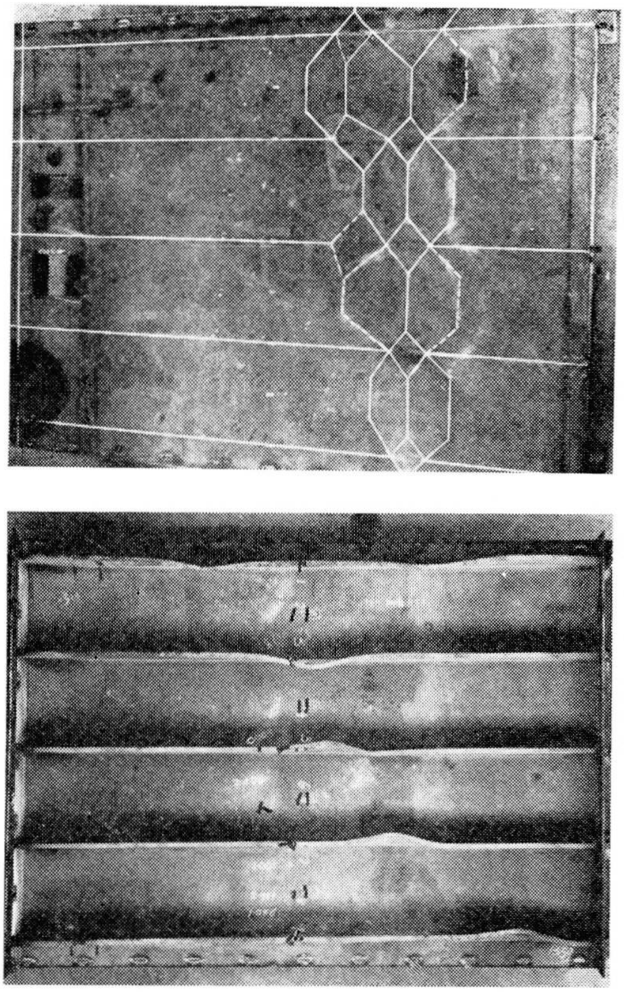


Legend: ——— Theoretical $P-\Delta$ curve assuming elastic steel only
——— Theoretical $P-\Delta$ curve assuming rigid-plastic steel only
– – – Actual $P-\Delta$ curve

Fig. 4. Elastic theory and rigid-plastic theory used to establish bound curves (a) for a beam and (b) for a strut.

Rigid-Plastic Analysis of Stiffened Plates

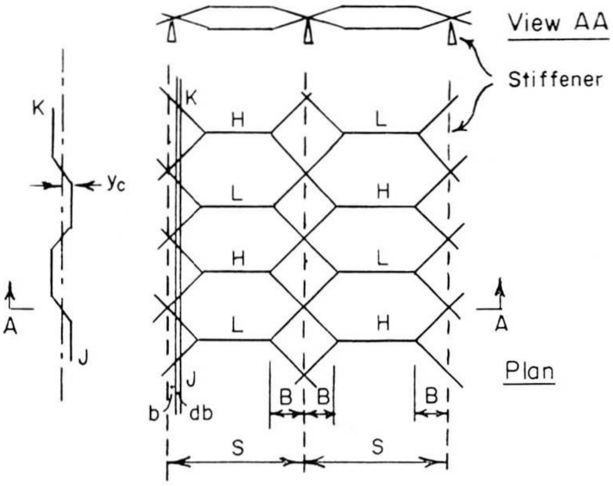
Experiments described later on stiffened plates in axial compression resulted in two collapse mechanisms illustrated in Fig. 5. Mode I was compressive failure of the plate followed (at a higher load) by a bending failure of the stiffeners and Mode II was initiated by a lateral buckling of the stiffener.



(a) Panel H failed in Mode I (plate compression – see Fig. 6).

(b) Panel J failed in Mode II (stiffener buckling – see Fig. 7).

Fig. 5. Photographs of identical panels which buckled in different modes.



H signifies high point
L signifies low point

Fig. 6. Mode I plastic collapse mechanism – plate compression.

Mode I mechanism is analysed as follows by the method set out in MATHESON [2]. A strip KJ of width db and thickness t (Fig. 6) can carry an axial load

$$dF_1 = \sigma_y t \left[\sqrt{\frac{4y_c^2}{t^2}+1} - 2\frac{y_c}{t}\right] db.$$

Let $y_c = \delta \frac{b}{B}$ and integrate from $b=0$ to B.

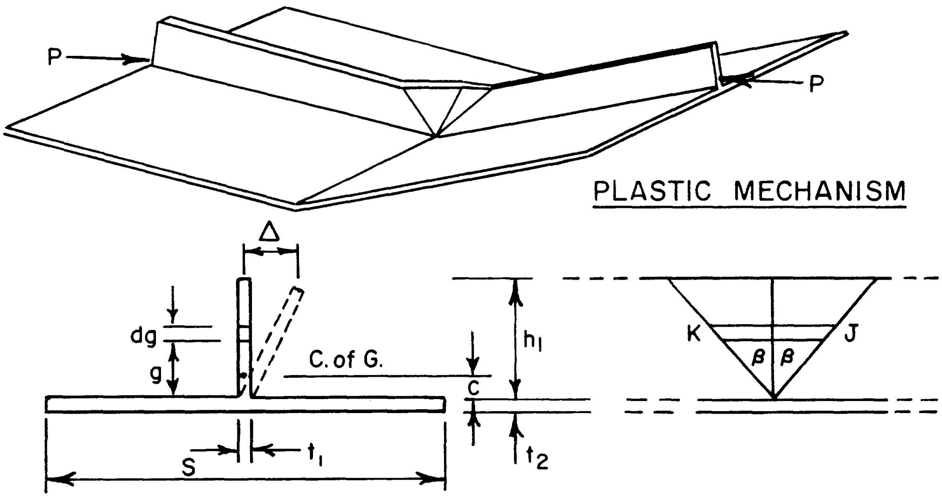
$$F_1 = \sigma_y t \left[\frac{B}{2}\sqrt{\frac{4\delta^2}{t^2}+1} - \frac{\delta B}{t} + \frac{tB}{4\delta}\log\left(\frac{2\delta}{t} + \sqrt{\frac{4\delta^2}{t^2}+1}\right)\right].$$

The middle strip in Fig. 6 consists of a series of pin-ended struts placed end to end with plastic hinges at their centres. Its axial load is

$$F_2 = \sigma_y (S-2B)\, t \left[\sqrt{\frac{4\delta^2}{t^2}+1} - \frac{2\delta}{t}\right].$$

Hence the total axial load carried by the plate during collapse is

$$F = F_2 + 2F_1 = \sigma_y t (S-B)\left[\sqrt{\frac{4\delta^2}{t^2}+1} - \frac{2\delta}{t}\right] + \frac{\sigma_y t^2 B}{2\delta}\log\left(\frac{2\delta}{t} + \sqrt{\frac{4\delta^2}{t^2}+1}\right). \quad (1)$$



Section at hinge   Side view of hinge

Fig. 7. Mode II plastic collapse mechanism – stiffener buckling.

A similar analysis of the Mode II mechanism shown in Fig. 7 is now carried out. A strip KJ in the stiffener carries a load

$$dF_S = \sigma_y t_1 \left[\sqrt{\left(\frac{\Delta g}{t_1 h_1}\right)^2 + 1} - \frac{\Delta g}{t_1 h_1}\right] dg.$$

By integrating from $g=0$ to h, the force in the stiffener is

$$F_S = \sigma_y t_1 \left[\frac{h_1}{2}\sqrt{\frac{\Delta^2}{t_1^2}+1} + \frac{t_1 h_1}{2\Delta}\log\left(\frac{\Delta}{t_1} + \sqrt{\frac{\Delta^2}{t_1^2}+1}\right) - \frac{\Delta h_1}{2 t_1}\right]$$

and its moment about $g=0$ is

$$M_S = \frac{\sigma_y t_1^3 h_1^2}{3\Delta^2}\left[\left(\frac{\Delta^2}{t_1^2}+1\right)^{3/2} - 1 - \frac{\Delta^2}{t_1^3}\right].$$

The resultant force in the plate and the plastic moment of the plate are

$$F_{pl} = \sigma_y S (t_2 - 2d_2),$$
$$M_{pl} = \sigma_y S d_2 (t_2 - d_2),$$

where d_2 is the depth of the tensile plastic zone in the plate.

$$F = F_S + F_{pl},$$
$$F\left(\frac{l\Delta^2}{h_1^2 \tan\beta} + c\right) = M_S - F_{pl}\frac{t_2}{2} + M_{pl}.$$

By eliminating d_2 from these equations the equation of the plastic collapse line (Mode II) is obtained.

$$F = 2\sigma_y S\left[\sqrt{\left(\frac{F_s}{2\sigma_y S} - \frac{t_2}{2} - \frac{l\Delta}{h_1^2\tan\beta} - c\right) - \left(\frac{t_2}{2} + \frac{F_s}{2\sigma_y S}\right)^2 + \frac{t_2^2}{2} + \frac{F_s t_2}{\sigma_y S} + \frac{M_s}{\sigma_y S}} + \frac{F_s}{2\sigma_y S} - \frac{t_2}{2} - \frac{l\Delta^2}{h_1^2\tan\beta} - c\right]. \quad (2)$$

The right hand sides of Eqs. (1) and (2) are functions of δ and Δ, respectively, and by letting them have different values the plastic collapse lines of the two modes can be plotted. However, for comparison of the two modes it is more convenient to convert them to the lateral deflection at the mid-point of the strut by considering the geometry of the mechanisms. In Fig. 8 this has been done for the stiffened plates used in the tests. In the theories outlined above a number of assumptions have been used as a basis. For example it is found that the value of β assumed has only a small influence upon the results so in

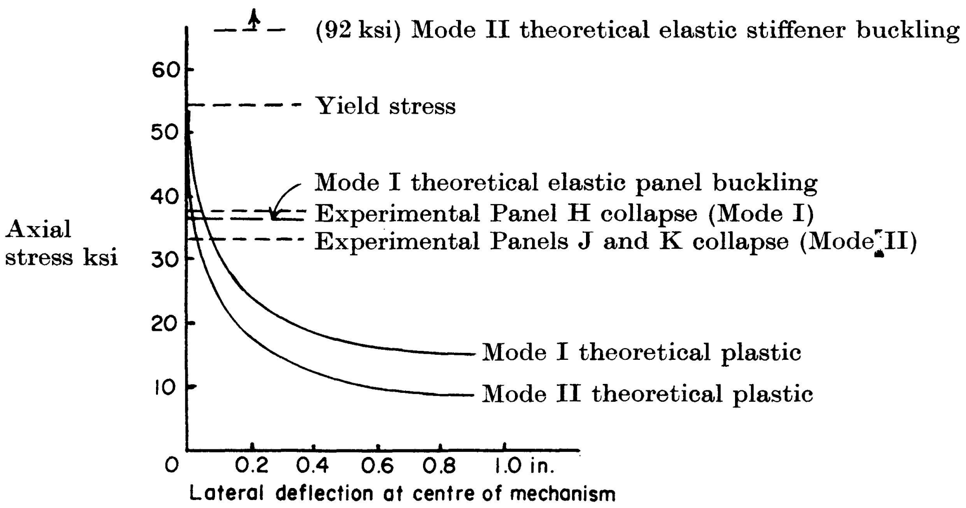


Fig. 8. Theoretical and experimental results of panel tests.

Fig. 8 a value of 45° was used. Also it has been assumed that at the slanting hinges the fibres are strained only in the direction of the applied load F. There are some doubts about the validity of this and ancillary tests have so far been inconclusive. The theory has also been developed to allow for slanting plastic hinges in each strip but it appears to have little effect when the deflections are small.

Elastic Analysis of Stiffened Plates

The stiffeners used in the panels were 6″ bulb flats which have very little torsional rigidity. When the plate buckles their main influence is to provide a nodal line. The classical stress for such a plate buckling as a square panel is [3]

$$\sigma_c = \frac{3\,\pi^2\,E}{(1-\nu^2)}\left(\frac{t_2}{S}\right)^2.$$

The measured imperfections in the plate were small so their influence on the critical load [14] has been ignored. The critical stress of the bulb flats was calculated using the Merrison Report [5] (Appendix A, Fig. 4.2.8). Each of these values is shown in Fig. 8.

Tests on Stiffened Panels

To date three full scale and one half scale compression tests have been conducted and one bending test. Space does not permit a full description of the tests and results. The large compression panels were loosely attached to a bed to allow for longitudinal movement but not vertical movement. A yoke consisting of $1\frac{1}{4}$″ dia. Macalloy bars and cross beams at each end enclosed the

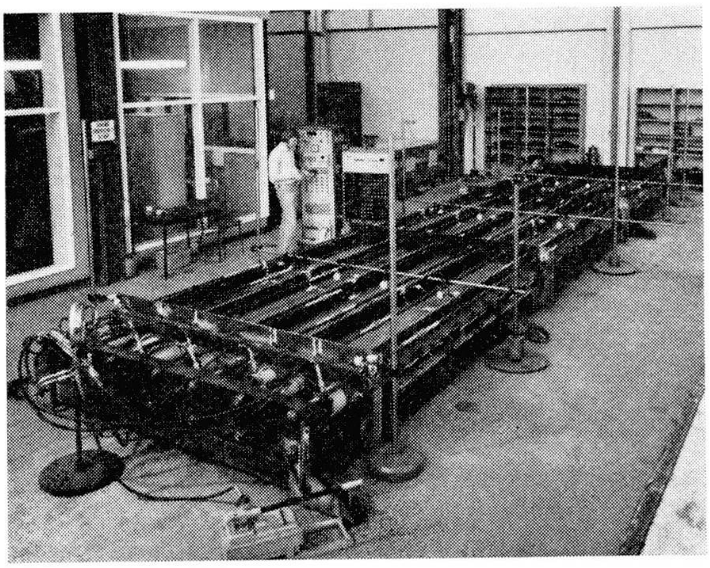


Fig. 9. Photograph of Panel K ready for testing.

specimen and the load was applied through a roller at each end of the specimen by 9/100 tons jacks manually operated (Fig. 9). Strain readings were used mainly as a statical check and dial gauges enabled the deflection pattern to be studied. Chronologically the tests were carried out as follows.

Panel H. The end rollers were fixed to the theoretical neutral axis and failure occurred in Mode I (i.e., plate failure, see Fig. 6). Collapse at 17.2 tons/in^2 was reasonably gradual until the mechanism was fully developed at which stage the load dropped off suddenly.

Panel J. Theoretical considerations outlined above suggested that if Panel *H* had failed in Mode II (i.e. stiffener failure, see Fig. 7) collapse would have occurred suddenly and at a lower load. Panel *J* was a duplicate of Panel *H* but the end rollers were raised only 1/10 in. and this was sufficient to induce Mode II collapse at 15.2 tons/in^2. Failure occurred with almost no visible warning.

It is interesting to note that if this panel had an initial central deflection of 0.16 in. the tip of the stiffener yields when the average axial stress is 15.2 tons/in^2. In this case first yield appears to be a good basis of design.

Panel M. A half scale model of Panel *H* was constructed and tested in a conventional testing machine to see whether it is possible to model stiffened panels. There appear to be two main difficulties, firstly the yield stress of model and prototype should be identical and secondly the imperfections should be scaled linearly. For the prototype and model the yield stresses were 23.2 tons/in^2 and 20.5 tons/in^2, respectively, and although the absolute values of the model imperfections were less than those of the prototype they were relatively greater even after using a preset camber. This panel failed at 14.8 tons/in^2 in Mode I.

Panel K. When three panels such as Panels *H* and *J* are joined end to end the panels will tend to buckle in one mode and the centre panel in the other mode. Panel *K* was identical in all respects to Panels *H* and *J* except for slight variations in initial imperfections and yield stress. The end rollers were located at the neutral axis. Failure occurred suddenly in Mode II in one of the end panels at 15.2 tons/in^2 (Fig. 10).

The results of these compression tests are indicated on Fig. 8.

Panel L. When a plate with thin stiffeners attached to one side is bent so that the free edge of the stiffeners carries compression stresses there is a tendency for the stiffeners to buckle laterally. If this occurs the load-carrying capacity reduces very rapidly and collapse is sudden. Panel *L* was a duplicate of Panels *H* and *J* but a uniform bending moment was applied over the central one-third of its length. The panel failed very gradually. The load-deflection

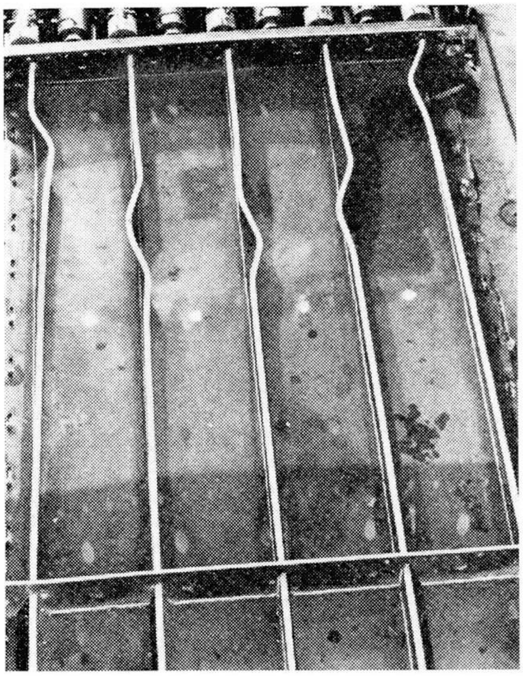


Fig. 10. End panel of K after Mode II failures.

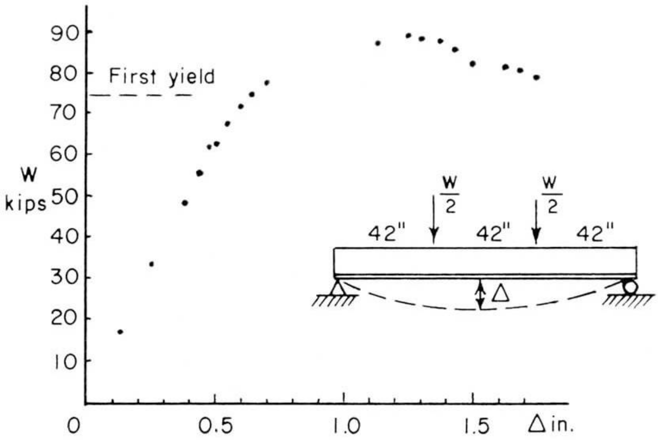


Fig. 11. Load-deflection curve of Panel L.

curve is shown in Fig. 11 where it is seen that a yield plateau of considerable magnitude exists even though this is a Mode II failure. As in the case of Panel *J* first yield appears to be a good basis of design.

The Behaviour of Wide Stiffened Panels

A wide plate stiffened in the direction of axial load by thin stiffeners should, under ideal conditions, carry uniform stress. However, it is inevitable that one stiffener will be less effective than the others. Once it buckles and starts unloading the others may be forced to buckle. Fig. 12 shows an analogy which explains the combined behaviour of the stiffeners. Thin stiffeners

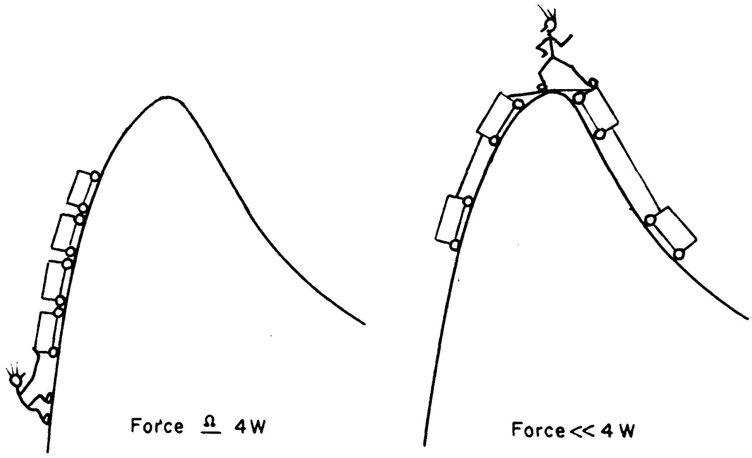


Fig. 12. Analogy of strength of axially loaded plate with four stiffeners. Simultaneous failure requires large load but when failures are not simultaneous collapse load may be much less.

exhibit a catastrophic reduction in load-carrying capacity once they buckle and the failure of one results in a very sudden redistribution of load. Quite *) often circumstances will arise where the strength of a wide stiffened plate is equal to the load at which the weakest stiffener reaches its maximum load-carrying capacity.

Conclusion

Most research on stiffened plates has been focussed on their elastic critical loads. Examination of the post-elastic region is just as fruitful and must certainly be considered in conjunction with elastic buckling. The rate at which a buckled structure unloads is most important when load factors and the effects of initial imperfections are being considered. A panel which fails slowly in a bend test may buckle suddenly without warning when it is axially loaded.

Acknowledgements

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*) The Author is indebted to his colleague, Dr. P. Grundy, for making this last point.

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Summary

This paper first draws attention to the failure of codes to take into account that some structures fail with little warning and have no alternative load paths whereas others fail slowly and collapse can be partial. For the first kind of structures higher load factors should be applied. Post-buckling behaviour is important and can be studied by simple rigid-plastic theory. The behaviour of large stiffened plates is discussed in the light of preliminary test results.

Résumé

L'article traite d'abord des prescriptions concernant le calcul des charges. Il existe des structures qui ne résistent pas, et ceci sans avertissement préalable, et qui n'offrent pas la possibilité d'une démolition alternative; d'autres échouent lentement et s'écroulent en éléments. Aux structures de la première catégorie des facteurs de charge supérieurs devraient être fixés. Le comportement du voilement post-critique est important et peut être examiné moyennant une simple théorie d'articulations entièrement plastiques. Le comportement de grandes plaques raidies est discuté sous l'aspect d'essais préliminaires.

Zusammenfassung

In dem Beitrag wird zunächst auf die Vorschriften zur Ermittlung der Traglasten eingegangen; es gibt Bauwerke, die ohne besondere Warnung versagen und keine alternative Abtragungsmöglichkeit besitzen, wogegen andere langsam versagen und in Teilen zusammenbrechen. Bei Bauwerken der erstgenannten Art sollten höhere Lastfaktoren angesetzt werden. Das Nachbeulverhalten ist wichtig und kann mittels einer einfachen Theorie vollplastischer Gelenke untersucht werden. Das Verhalten grosser ausgesteifter Platten wird im Lichte erster Teilversuche diskutiert.

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