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Simply Supported Long Thin Plate I-Girders without Web Stiffeners Subjected to Distributed Transverse Load

Poutres longues à âme mince sans raidisseurs, simplement appuyées et soumises à une charge uniforme

Einfach gelagerte, lange, dünnwandige Blechträger ohne Stegaussteifungen, unter gleichmässiger Belastung

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1. INTRODUCTION

During the last ten years there has been a marked increase in the use of welded thin plate I-girders especially in roof constructions. This has been made possible by the use of rational methods of fabrication and design. One essential point is - in spite of the thin web - to avoid web stiffeners, which have to be manually fitted and welded and thus cause considerable costs.

The simply supported girder is a common element in roof constructions. When the load is sufficiently distributed along the girder no other vertical web stiffeners than at the supports are needed. When the girder is subjected to a few concentrated loads vertical web stiffeners are required to prevent web crippling.

This investigation deals with long simply supported plate girders with web--stiffeners at the supports loaded with distributed transverse loads. The web is then subjected to varying shear forces, distributed edge loading and bending stresses simultaneously. Tne simple cases of loading, constant shear, bending moment or edge loading have been studied but not this combination of all three.

Granholm [9] has made tests on web crippling and he has given the empirical formula $P = 85\ 000\ d^2$ (d in cm gives P in kp) for the buckling load. This formula has been confirmed by Bergfeldt and Hövik [5]. See fig. 1.

Shear loaded welded girders with large distance between web stiffeners ($a/h > 2.6$) have been tested by Wästlund and Bergman [17] Basler et al [1], Granholm [9], Cooper [7], Nishino-Okumara [12], Fuiji [8]. The test girders have been loaded with constant shear forces, in some tests combined with bending moment, see fig. 2 and 3.

Theoretical solutions for an infinitely long plate subjected to the action of shearing forces along the edges have been obtained by Kromm and Marguerre [10], Bergman [6] and Skaloud [15]. These theories which are based on the differential equations for large deflections gives informations of bending and membrane stresses in the elastic postbuckling range.

The research work at Lehigh University [1,2,3,4,7] on plate girders with transverse stiffeners with a spacing less than three times the girder depth (a/h < 3) has resulted in specifications for such girders [18]. The influence of the stiffeners of the flanges has been studied e.g. by Bergman [6], Fuiji [8], Rockey and Skaloud [13,14].

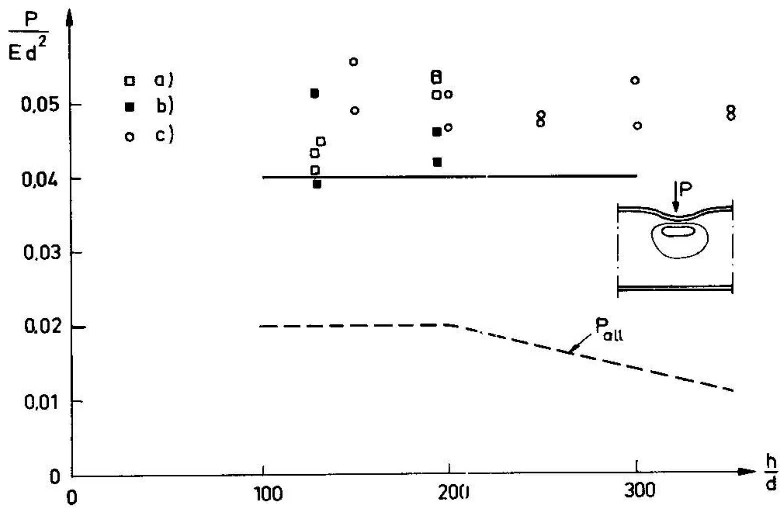


Fig. 1 Test results on web crippling. [5], [9].

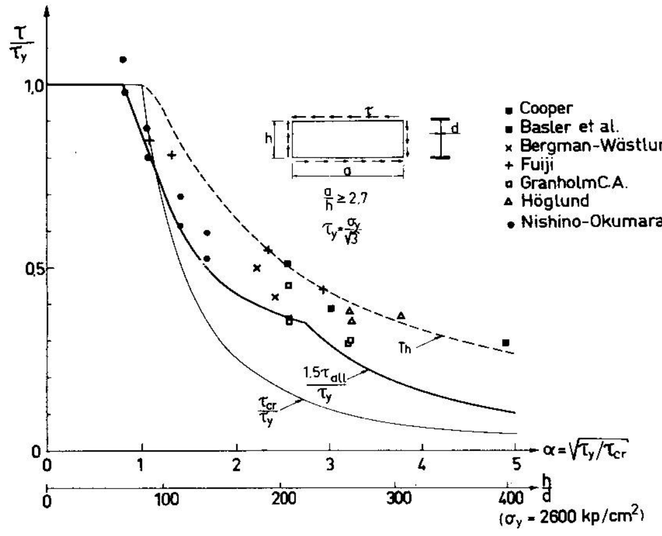


Fig. 2 Test results of welded plate girders subjected to shear. Th = theoretical ultimate load, see fig. 19.

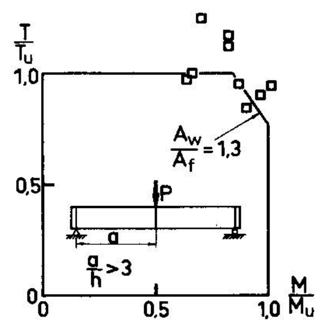


Fig. 3 Test results of welded plate girders subjected to bending and shear. Granholm [9].

$T_u = hd\,\tau_{all}$ where τ_{all} = allowable shear stress according to [19].

$$M_u = \sigma_y 2I/h(1 - 0{,}0005\,\frac{A_w}{A_f}\,(\frac{h}{d} - 5{,}7\sqrt{\frac{E}{\sigma_y}})) \qquad [2]$$

In Sweden special specifications for the design of welded plate girders in roof constructions mainly loaded with dead load [19] have been in use since 1966. The draft of these specifications which contains rules for the complete design of plate girders with thin web was worked out by H. Nylander and the author by order of Gränges Hedlund AB, Stockholm. This paper is a part of the basis for these specifications. In fig. 1 and 2 test results are compared with the allowable load P_{all} regarding to web crippling and the allowable shear stress τ_{all} according to these specifications. In fig. 3 test results for girders loaded in bending and shear are compared with an interaction curve according to Basler [4].

2. TEST PROGRAM AND TEST PROCEDURE

Three girders of structural carbon steel were tested, two with 9 m span and one with 6 m span, see fig. 4.

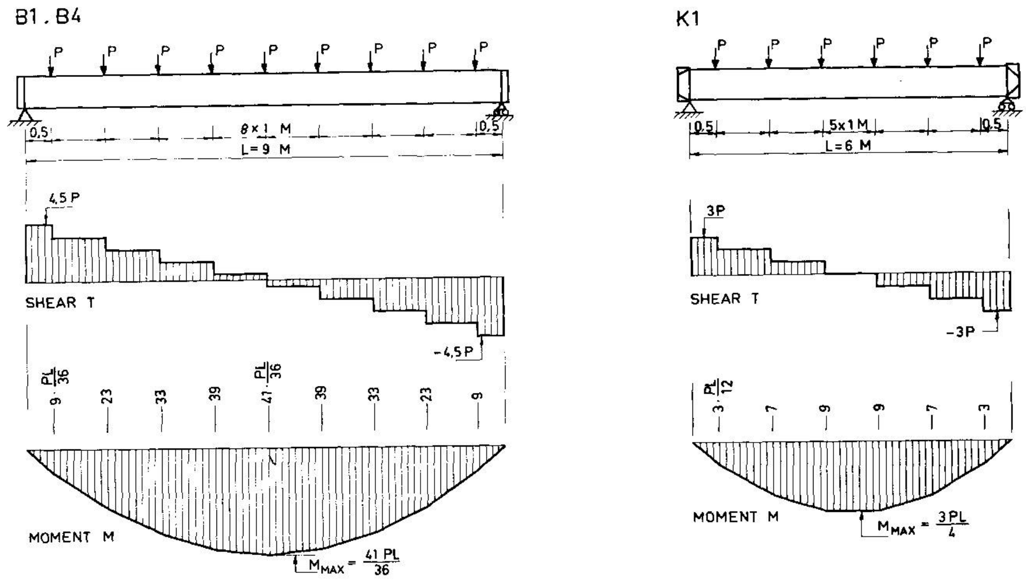


Fig. 4a Distribution of load, shearing forces and bending moment for the test girders.

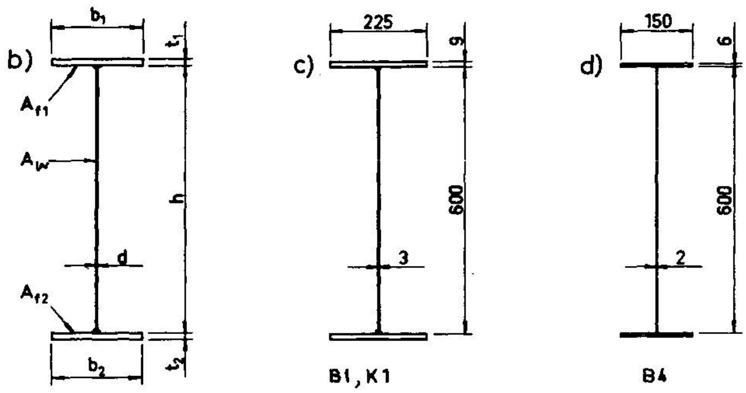
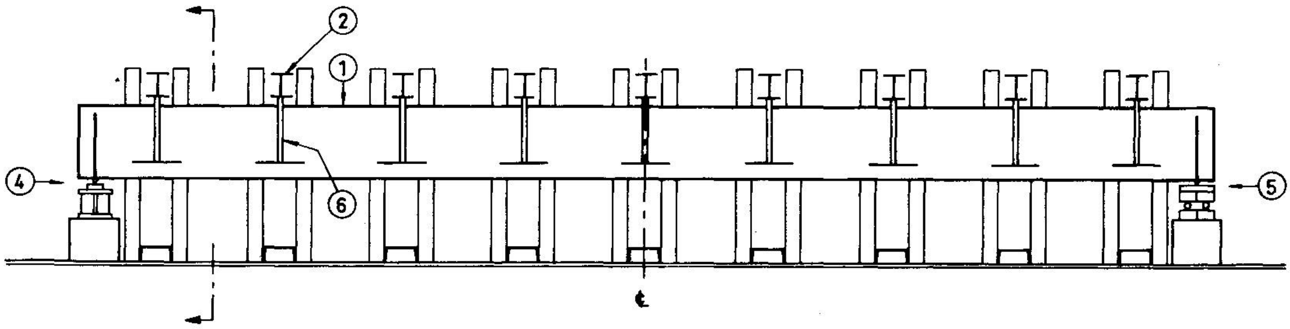


Fig. 4b-c-d. b) Notations for cross section. c) and d) Girder cross sections for the test girders.

The girders were simply supported and loaded with nine or six gravity loads with a spacing of 5/3 of the girder depth. The gravity loads were produced by levers and scales with weights, see fig. 5. The test girders were fabricated of flame cut flange and web plates in an automatic welding machine.



1 Test girder
2 Lever
3 Ball joint
4 Fixed support
5 Movable support
6 Scale
7 Weights

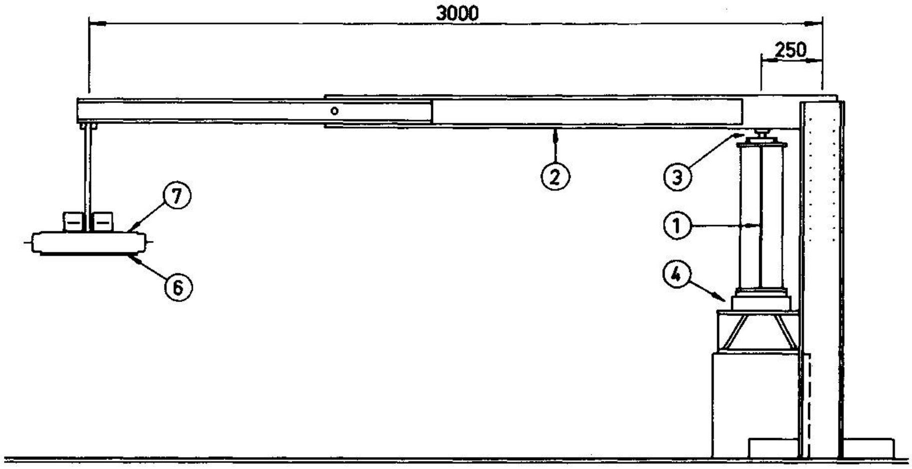


Fig. 5 Test setup.

Table 1. Cross sectional properties and yield point of the flange and the web material.

Test girder	h cm	d cm	b cm	t cm	h/d	I_x cm^4	σ_y kp/cm^2 flange	σ_y kp/cm^2 web
B1	60.0	0.286	22.6	0.99	210	46600	2944	4185
B4	60.0	0.200	15.1	0.61	300	20500	3040	2800
K1	60.0	0.286	22.6	0.99	210	46600	2944	4185

Details of the test girders are given in table 1.

The surface strains were measured with electrical strain gauges at points near the supports on the web, on the stiffeners and on the flanges. The out of plane deflections of the web were determined with a photogrammetic method.

Fig. 6, 7 and 8 show load versus midspan deflection curves for the test girders. The following reference loads are given in the figures:

$P_{\tau cr}$ = shear buckling load for the web calculated under the assumption that the web is simply supported, very long and subjected to pure shear

$P_{\sigma cr}$ = the buckling load in bending for the web at midspan section of the girders

$P_{\sigma s}$ = the bending moment $\sigma_y I/(h/2)$ at midspan.

The ultimate loads are denoted P_{br}^{Bl} and P_{br}^{Kl}. The web deflection configuration at ultimate load is indicated in the figures.

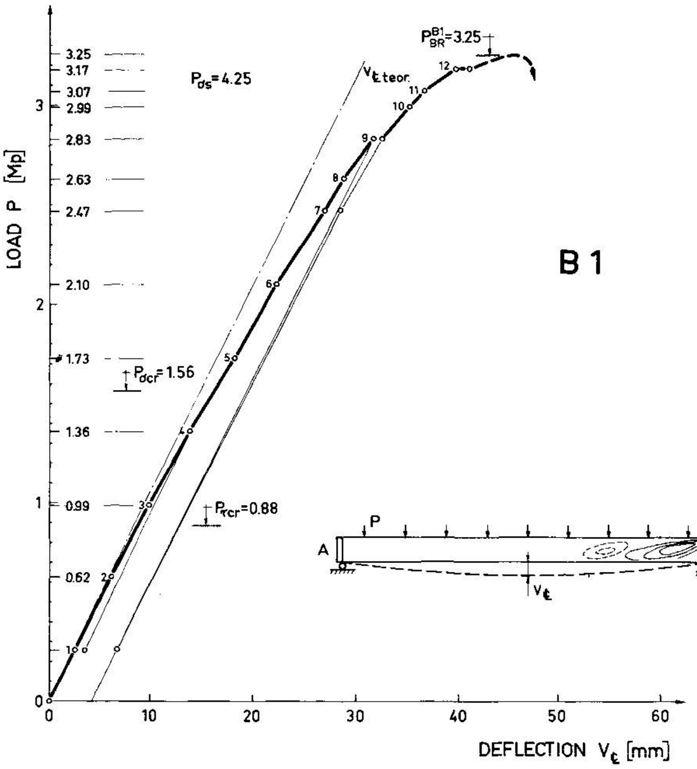


Fig. 6 Load-deflection curve of test girder B1.

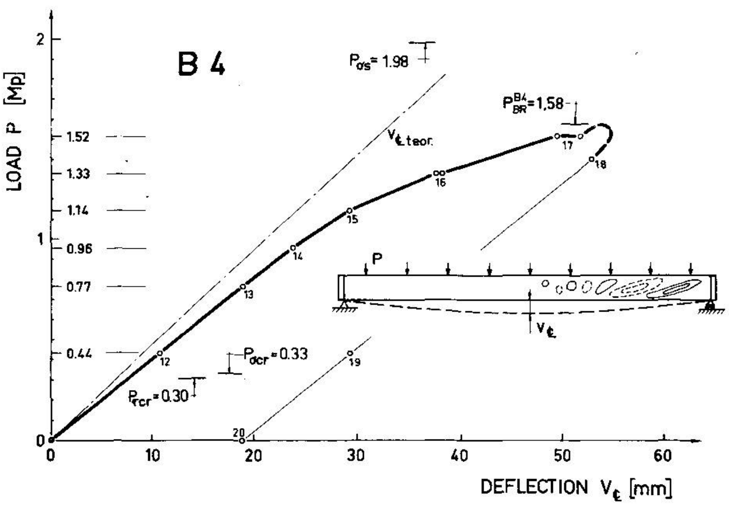


Fig. 7 Load-deflection curve of test girder B4.

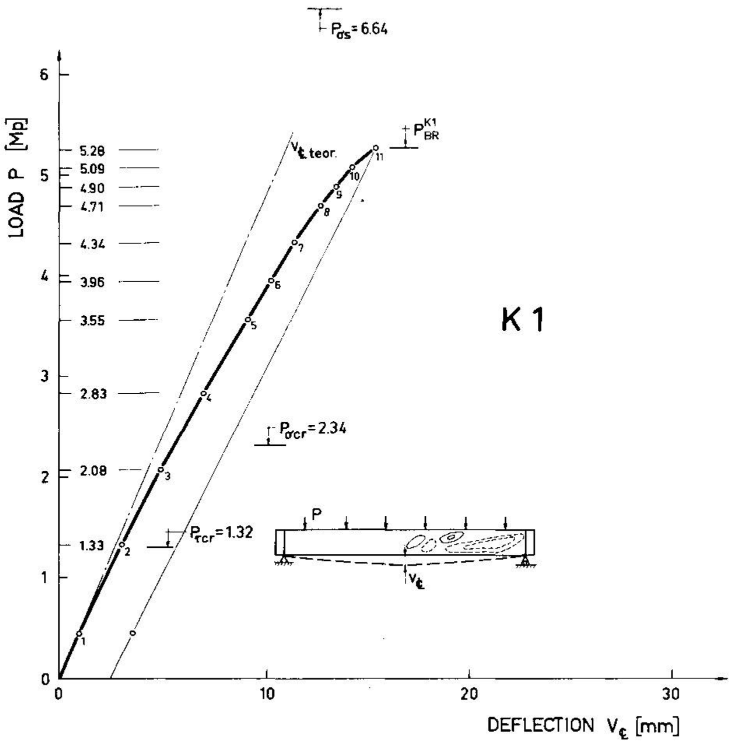


Fig. 8 Load-deflection curve of test girder K1.

3. TEST RESULT

Prior to testing, the initial deformations of the web plate were determined by photogrammetric method. The testing procedure consisted of taking new photos, dial gauges and strain readings at each load level.

The web of test girder B1 was at support A stiffened with two web stiffeners and with a single stiffener at support B.

Fig 9 and 11 show the deformation of the web of girder B1 at different loads. The principal stresses in the middle surface calculated from strain measurements at six points of the web are shown with stress vectors. Typical curves for the relation between the principal stresses and the load for two points in the web, one at each support, are shown in fig 10 and 12. As in other tests the compressive principal stress σ_2 reached a certain value near the shear buckling stress and remained approximately at this level up to the maximum load.

The web of test girder B4 and K1 was stiffened with two stiffeners at both supports. Fig. 13 shows the web deflection and fig. 14 the flange strains at support of girder B4.

The ultimate load for the three test girders varied from 3.7 to 4.7 times the shear buckling load, se table 2.

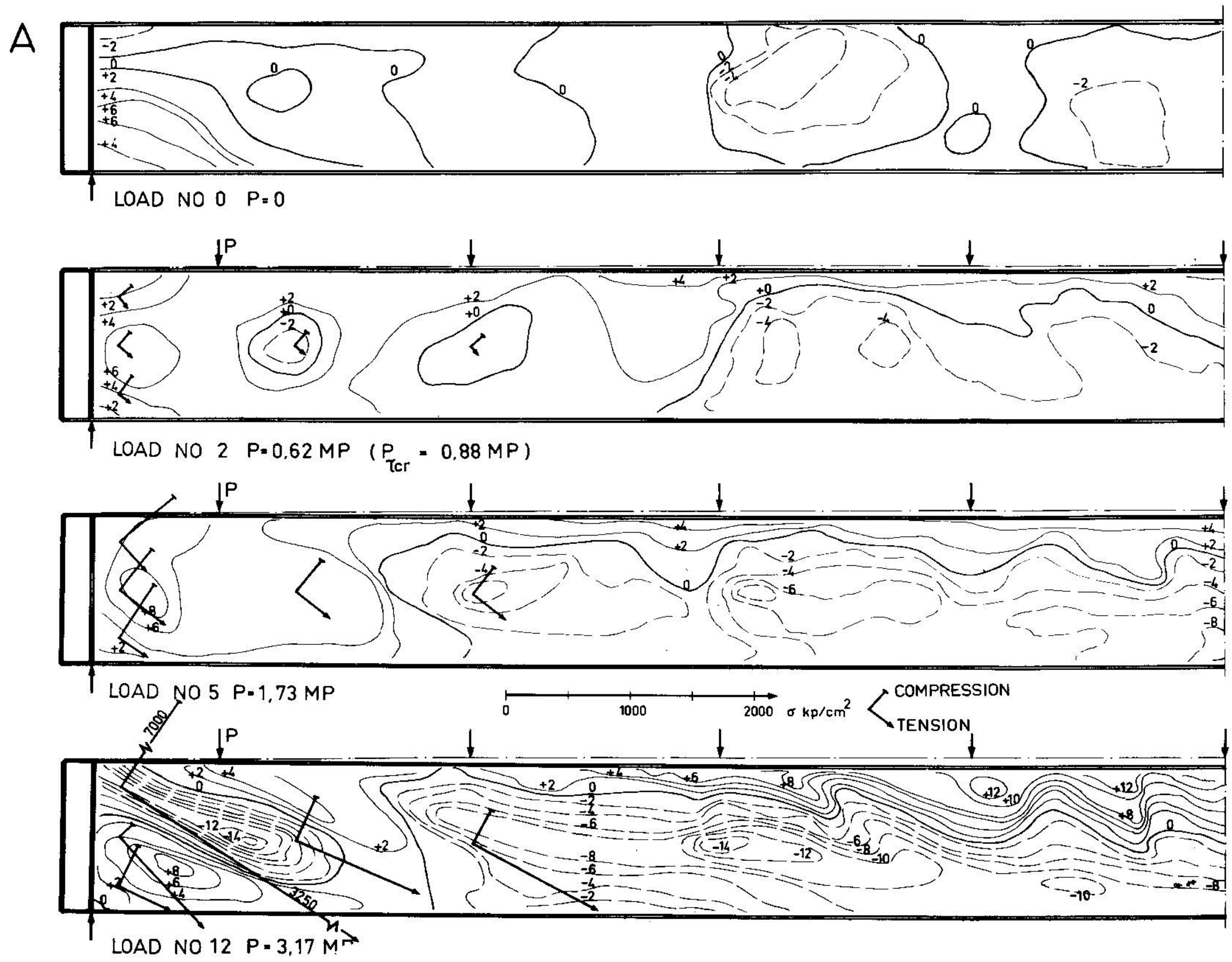


Fig. 9 Web deflection (in mm) and principal stresses in the web of test girder B1 at support A.

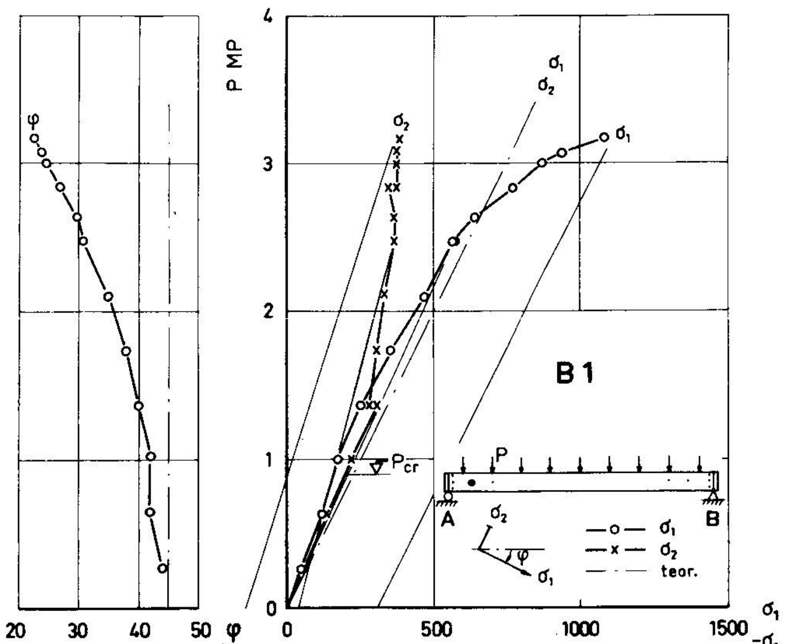


Fig. 10 Principal stresses in the web of test girder B1 at support A as a function of the load P.

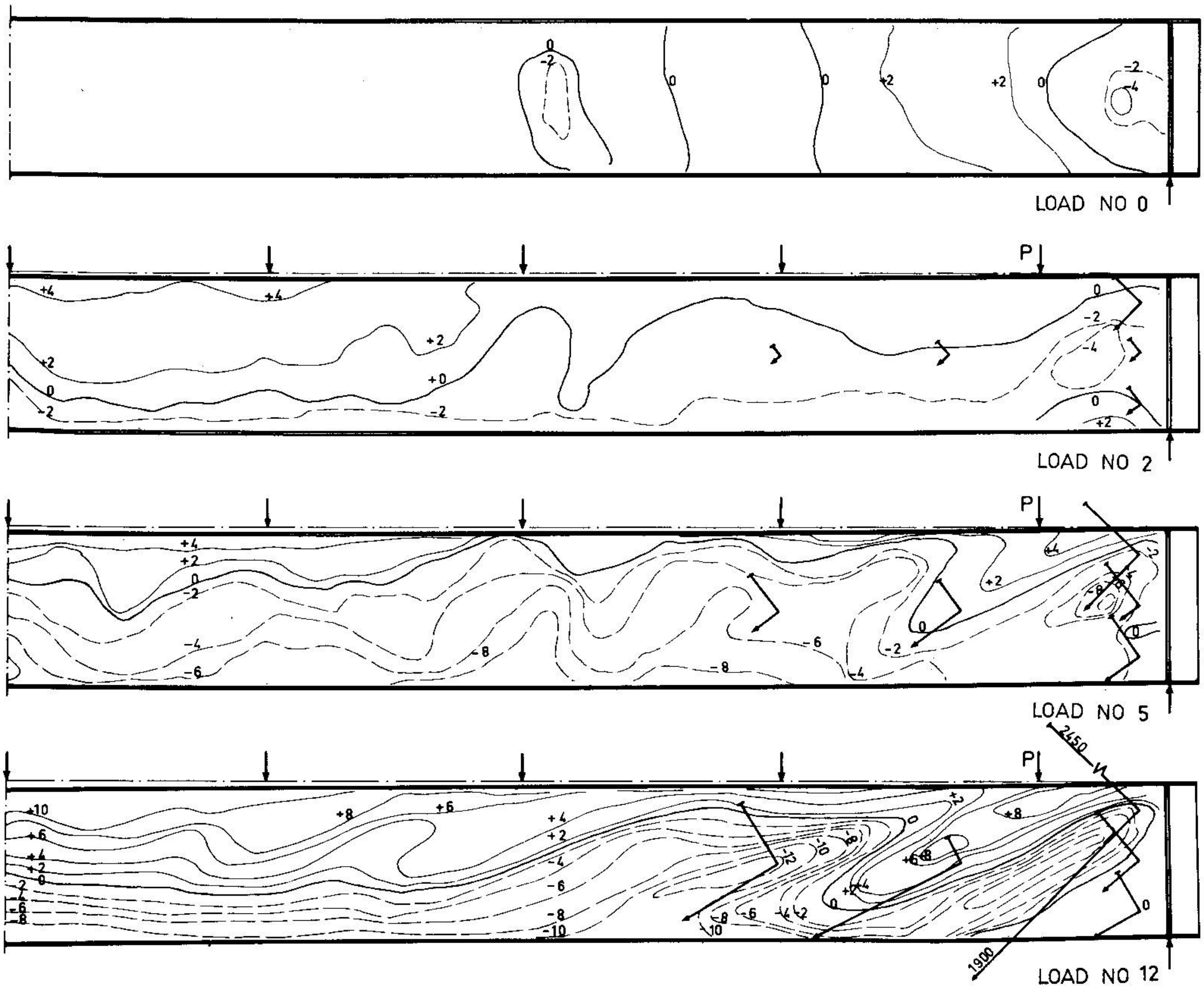


Fig. 11 Web deflection (in mm) and principal stresses in the web of test girder B1 at support B.

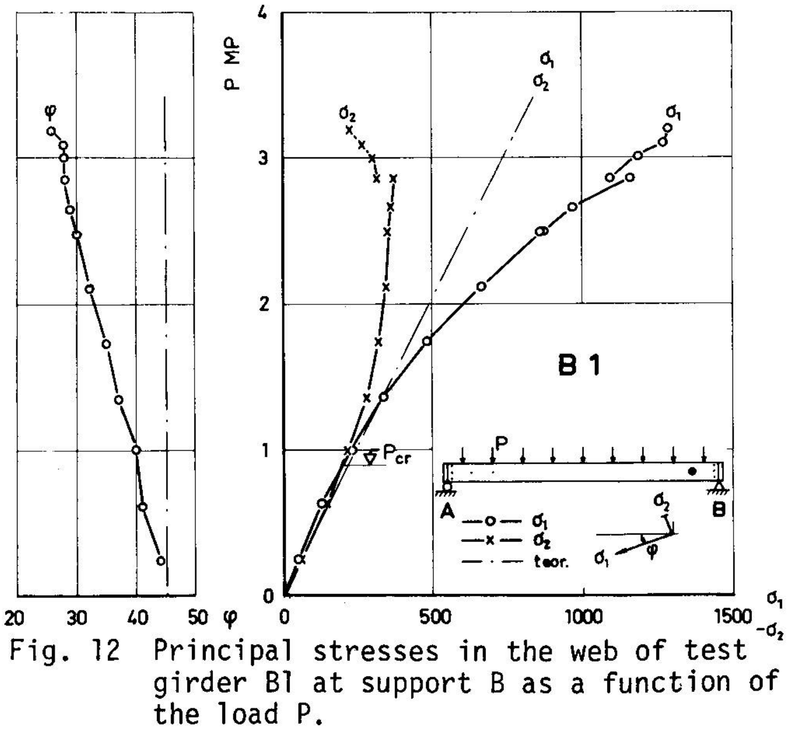


Fig. 12 Principal stresses in the web of test girder B1 at support B as a function of the load P.

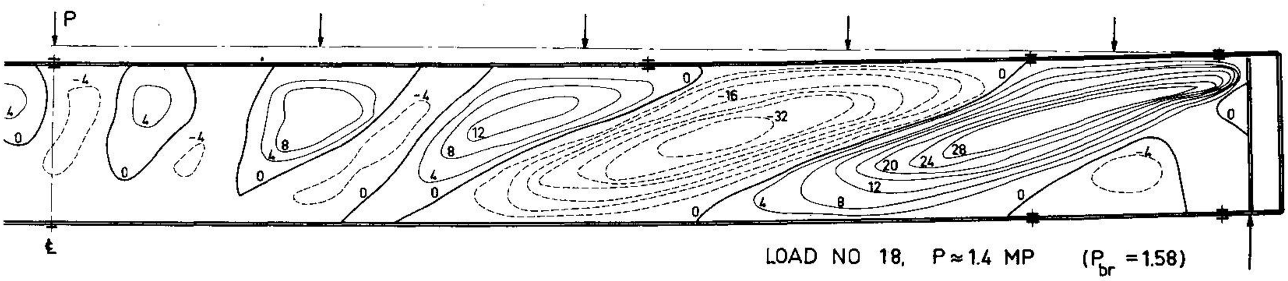


Fig. 13 Web deflection (in mm) of the test girder B4.

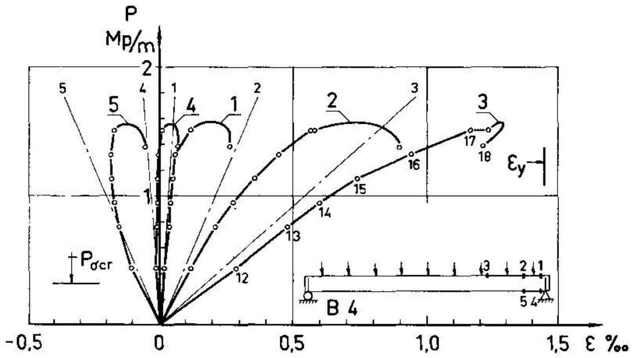


Fig 14 Flange strains in test girder B4.

Table 2. Summary of test results

Test girder	$\frac{h}{d}$	τ_y kp/cm²	τ_{cr} kp/cm²	α	P_{br} Mp	τ_u	$\frac{\tau_u}{\tau_y}$	$\frac{\tau_u}{\tau_{cr}}$
B1	210	2420	231	3.24	3.25	853	0.35	3.69
B4	300	1620	113	3.78	1.58	529	0.37	4.68
K1	210	2420	231	3.24	5.28	924	0.38	4.00

$\tau_y = \sigma_y/\sqrt{3}$; $\alpha = \sqrt{\tau_{cr}/\tau_y}$; P_{br} = ultimate load (1Mp = 2205 ℓb)

τ_u = maximum shear force at ultimate load.(1 kp/cm²=14,2 ℓb/sq in)

4. THEORY

In order to obtain an intelligible model of the shear loaded girder the web is replaced with a system of bars shown in fig. 15. The angle between the tension bars and the flanges is denoted β and the compression bars are perpendicular to the tension bars.

When the angle β is decreased the buckling load T_{cr} for the bar system is increased. The stress σ_c in the compression bars for the buckling load is almost independent of β and approximately the same as the shear buckling stress τ_{cr} for the infinitly long web plate. This is still valid then the deflections become large. For a given bar system the load increases only a little when the deflections become large.

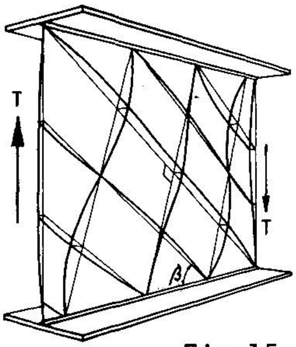


Fig 15

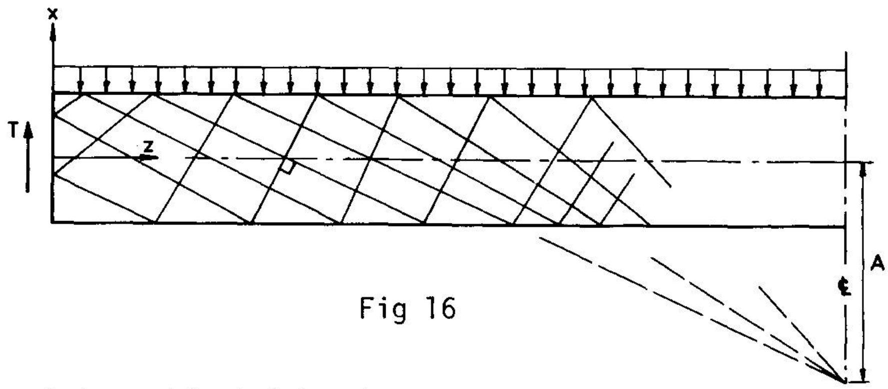
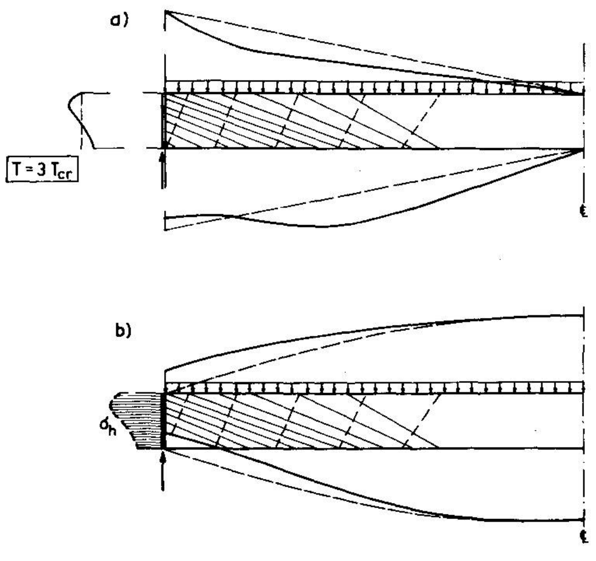


Fig 16

Fig. 15 Bar system for a girder subjected to shear.

Fig. 16 Bar system for a simply supported girder with distributed transverse load.

For the simply supported girder with distributed load the inclination of the bars is varied along the girder length because the shear force varies. The stresses in the compression bars are assumed to be less than or equal to τ_{cr} except at the upper corners at the supports where the bars are shorter and can resist stresses larger than τ_{cr}.



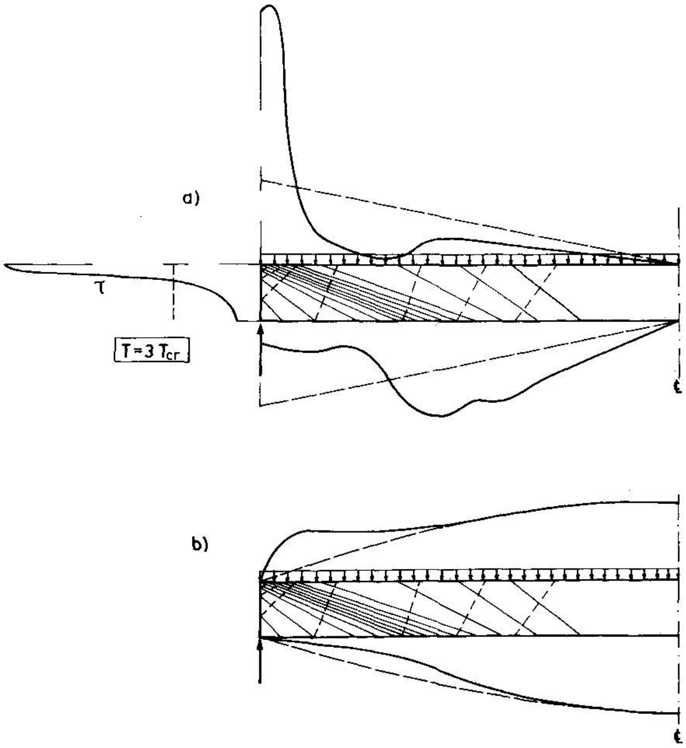


Fig. 17 Strong web stiffener Fig. 18 Weak web stiffener
a) Calculated distribution of shear stresses between the flanges and the web. b) Distribution of flange stresses.

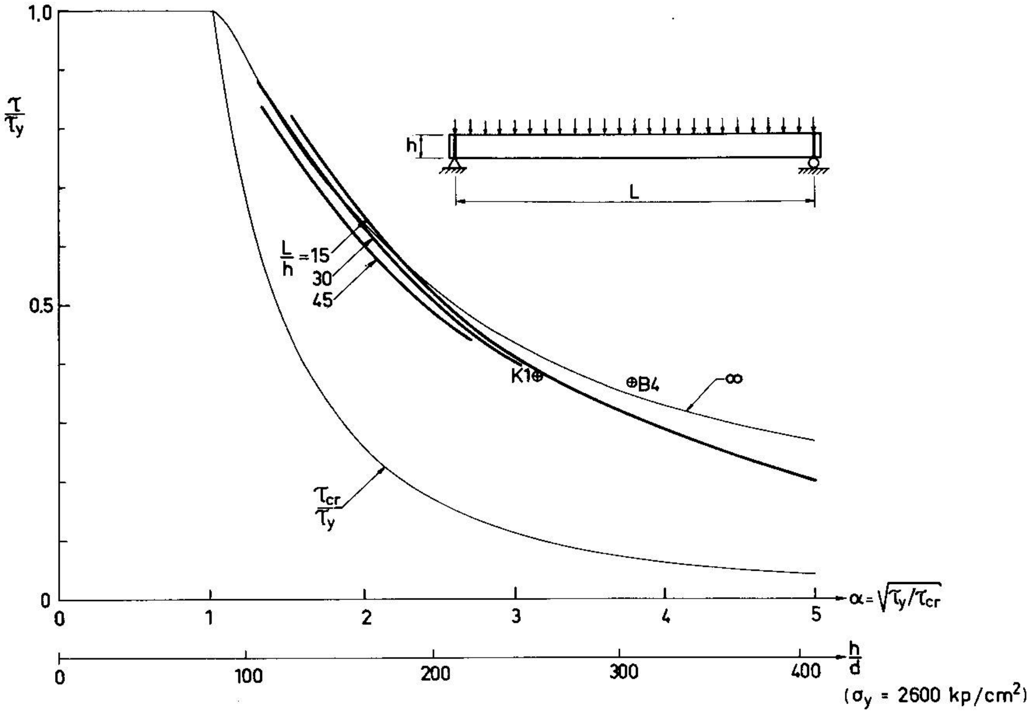


Fig. 19 Calculated ultimate load for girders with strong web stiffeners at the supports.

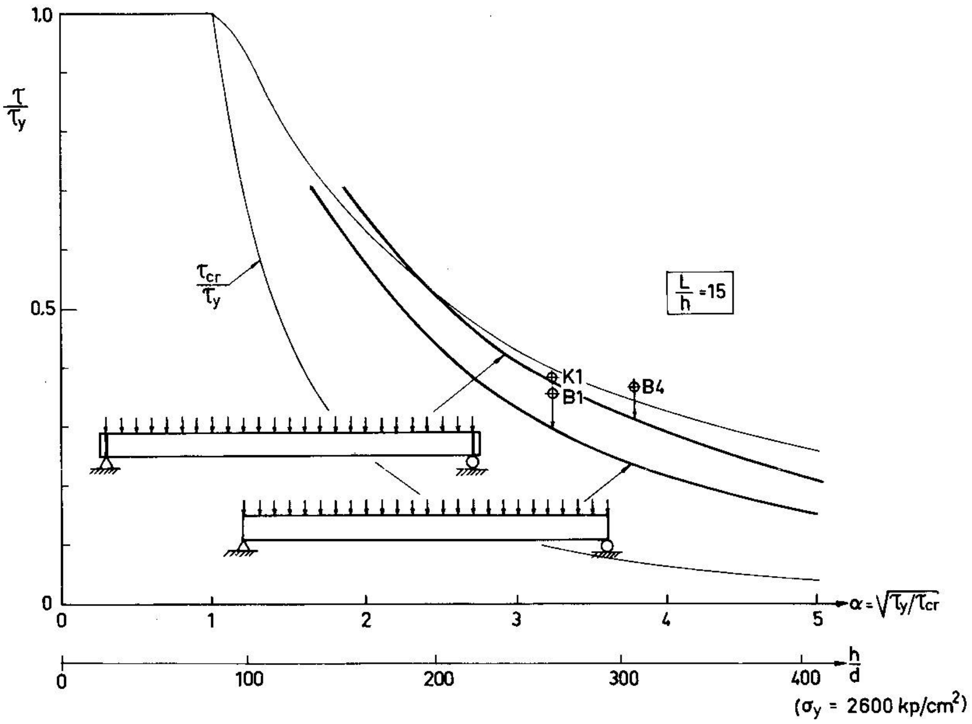


Fig. 20 Calculated ultimate load for girders with weak web stiffeners and strong web stiffeners at the supports.

Some results of the calculations for two cases are shown in fig. 17 and 18. The distribution of the shear forces between the web and the flanges, the web and the stiffeners (fig. 17a and 18a) is different for the two cases depending on the different bending stiffness of the web stiffeners at the supports. The strong web stiffeners (fig. 17) can resist the horisontal stress components of the tension and compression bars and the tension bars can be distributed over the girder depth. When the stiffeners are weak (fig. 18) the tension bars must be concentrated to the upper corners at the supports. Fig. 17b and 18b show the corresponding flange forces.

The tests confirm the theory with regard to stresses in the web, the flanges and the web stiffeners except in one respect:
The measured principal compression stresses are greater than the shear buckling stress for a long, simply supported panel with constant shearing force, which may be part of the explanation of the fact that the theory underestimates the ultimate load, see fig. 19 and 20.

5. ACKNOWLEDGEMENT

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SUMMARY

Tests were performed on long simply supported thin plate I-girders with web stiffeners only at the supports. The girders were loaded with nine or six gravity loads. The depth to thickness ratio of the web ranged from 200 to 300. A theory is briefly presented where the web is assumed to be composed of a system of compression and tension bars.

RESUME

Des essais ont été effectués sur des poutres à âme mince simplement appuyées, avec raidisseurs aux appuis seulement. On a chargé les poutres par neuf ou six charges concentrées. Le rapport entre la hauteur et l'épaisseur de l'âme variait entre 200 et 300. On en présente une théorie en supposant que l'âme soit composée d'un système de barres de tension et de compression.

ZUSAMMENFASSUNG

Es wurden Versuche an langen, dünnen, einfach gelagerten I-Vollwandträgern durchgeführt, welche lediglich an den Enden Stegaussteifungen besassen. Man belastete die Träger mit sechs oder neun konzentrierten Kräften. Das Verhältnis der Höhe zur Dicke variierte zwischen 200 und 300. Es wird kurz eine Theorie vorgelegt, unter der Annahme, der Steg bestehe aus einem System von Zug- und Druckstäben.