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Thin-Walled Deep Plate Girders under Static Loads

Poutres à âmes pleines minces et hautes sous charge statique

Hohe Vollwandträger mit dünnen Stegen unter ruhender Last

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Introduction

During the last years there has been an increasing interest in using welded steel plate girders with exceptionally slender webs. Usually such girders have web stiffeners as these increase the bearing capacity considerably.

The cost of the stiffeners are however out of proportion as the manual work disturbs the normal flow of automatic production. They increase the cost of painting and future maintenance too. Also they make it more difficult to use the girder as a standardized structural member as they may restrict the positions of the forces. Thus it has often been discussed to limit the number of web stiffeners or wholly to avoid them.

In USA the 1963 AISC specifications allow omitting of stiffeners up to a web slenderness ratio h/d of 260. In Sweden there are temporary specifications allowing h/d up to 320 with stiffeners at the supports only. According to these Swedish specifications important roof structures have been built with plate girders having clear spans up to 50 m.

Formerly it was a common thought that the upper bound of possible stress in the web was the theoretical buckling stress based on the assumptions of small deflections and no initial deflections. It is now, however, since long well known that this theoretical load by no means corresponds to that shear force, which can be carried by a girder with a slender web. Is is evident that in such a girder redistributions of the inner forces are possible, which lead to much higher failure loads.

If the girder has web stiffeners it is suitably calculated following the theory of tension fields. For a girder without stiffeners it has been pointed out [1] that for a long slender plate, under certain boundary conditions, a shearing force up to 3 times the just mentioned idealized theoretical buckling load ought to be carried.

Extensive tests on plate girders have been reported upon for instance by Massonnet [2] and prior to the working out of the 1963 AISC specifications by Basler and Thürlimann [3]. During recent years Corbit and Marsh, Schilling, Cooper and others have completed our knowledge.

Testing programme

The following tests have been performed in order to get an experimental evaluation of the influence of web stiffeners. The intention was both to limit the number of web stiffeners and to develop new methods of fastening them.

The main view points of the testing programme were concentrated on the following problems.

a. Information on the all-over behaviour of girders with exceptionally slender webs

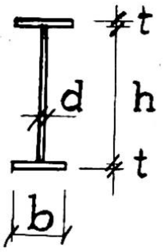
b. General influence of web stiffeners on the bearing capacity of the slender plate girders

c. Web stiffening - with special regards to the method of application, height of stiffeners and other special questions concerning stiffener details.

d. Web crippling under concentrated loads

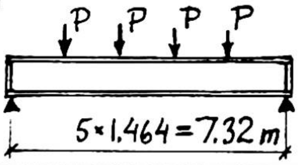
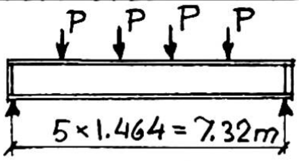
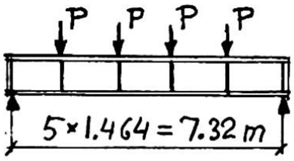
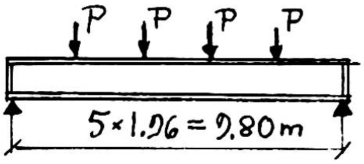
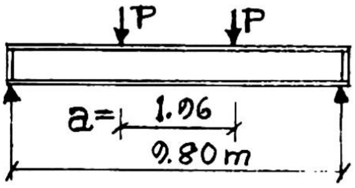
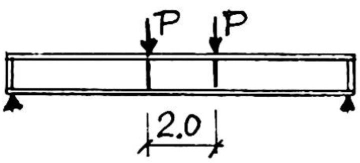
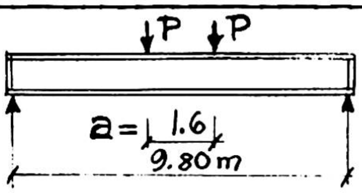
A great many tests have been conducted on test specimens with dimensions as shown in Table 1. The span widths and loading cases are shown in Table 2.

TABLE 1. DIMENSIONS OF TEST GIRDERS

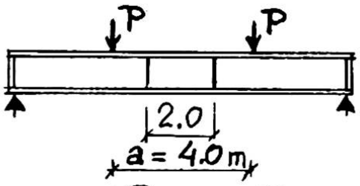
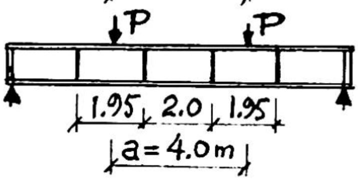
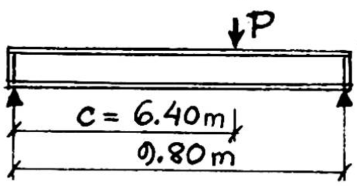
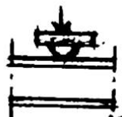
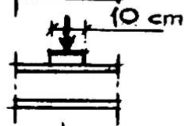
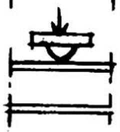
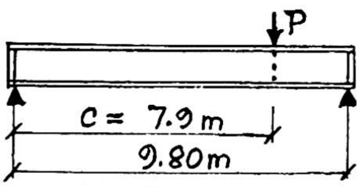
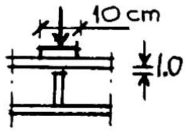
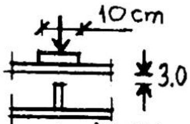
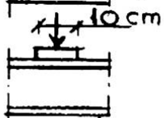
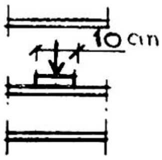


Girder No.	h cm	d cm	b cm	t cm	h/d	$\sigma_{yield, flange}$ $\sigma_{yield, web}$ kp/cm²
1	60	0,2	17,5	0,6	300	2900 3400
2	60	0,2	17,5	0,6	300	St 37
3 - 4	59	0,3	20	0,8	197	St 44
5	59	0,3	20	0,8	197	3100 2800
6	59	0,3	20	0,8	197	2750 3300
7	59	0,3	20	0,8	197	2950 3270
8 - 9	30	0,2	10	0,6	150	St 37
10 - 11	40	0,2	10	0,8	200	St 37
12 - 13	50	0,2	10	1,0	250	St 37
14 - 15	60	0,2	10	1,2	300	St 37
16 - 17	70	0,2	10	1,5	350	St 37

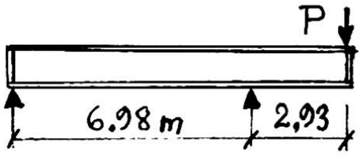
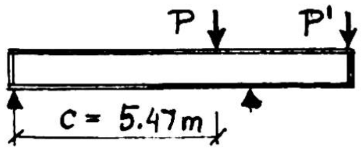
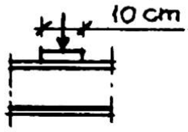
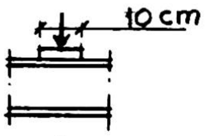
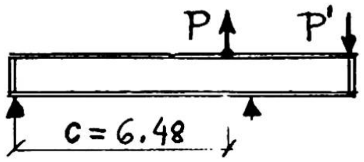
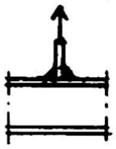
TABLE 2. LOADING CASES, BENDING AND SHEARING STRESSES

Girder No.	Test No.	Loading cases and span widths meters	Maximum observed load, P Collapse loads P_c Megapond (1000 kilopond)	Maximum stress σ_{max}, τ_{max} kp/cm^2	Notations $\frac{\tau_{max}}{\tau_{cr}}$ etc.
1	1		P_c = 2,8 Total compression flange instability	σ = 1640 τ = 467	4,1 See fig. 1
2	1		P = 2,0 Two loading cycles. No rupture		
	2		P_c = 3,8 Local flange buckling under the second point load	σ = 2220 τ = 635	
3	1		P = 5,2 Exc. lat. defl.	σ = 2750 τ = 587	2,25
	2		P = 6,85 Exc. lat. defl.	σ = 2400 τ = 387	1,48
	3	a = 4,0	P_c = 9,2 Web crippling	σ = 2400 τ = 520	1,98
	4	a = 5,0	P_c = 10,2 Web crippling	σ = 2200 τ = 576	2,20
	5	a = 6,0	P_c = 10,95 Web crippling	σ = 1865 τ = 620	2,37
	6	a = 8,0	P_c = 11,7 Web crippling	σ = 940 τ = 650	2,50
	7		P_c = 10,7 Local flange buckling under one P load	$\sigma > \sigma_{yield}$ τ = 605	
4	1		P_c = 3,1 Excessive flange rotation under one point load.	σ = 1140	The jack was hinged to the flange

TAB.2.cont.

Girder	Test	Loading cases	Maximum load	Max. stress	Notations
4	2	a = 2,0	P_c = 5,5 Lateral deflection and local yielding	σ = 1930	
	3	a = 3,0	P_c = 5,62 Excessive lateral deflection	σ = 1740	
	4	a = 4,0	P_c = 5,9 Excessive lateral deflection	σ = 1550	
	5		P_c = 5,48 Excessive lateral deflection	σ = 1430	
	6		P_c = 6,0	σ = 1560	For a second loading cycle P_c = 5,5
	7		P_c = 8,45 Web crippling		
	8	c = 2,38	P_c = 9,48 Web crippling	σ = 1530 τ = 405	
	9	c = 7,4	P_c = 9,8 Web crippling	σ = 1600 τ = 418	
	10	c = 2,4	P_c = 8,8 Web crippling		
	11		P_c = 13,7 Web crippling	σ = 1870 τ = 625	
	12	c = 6,9	P_c = 12,4 Web crippling	σ = 2270 τ = 490	
	13	c = 7,4	P_c = 9,9 Web crippling	σ = 1610 τ = 670	
	14	c = 6,4	P_c = 8,7 Web crippling	σ = 1730 τ = 320	

TAB.2.cont.

Girder	Test	Loading cases	Maximum load	Max. stress	Notations
5	1		P = 10,53		Three loadi cycles
5	2		P' = 8,0 P_c = 10,63 Web crippling		P' remaine constand du ring loading
	3	c = 6,48	P' = 9,0 P_c = 9,9 Web crippling		
	4		P' = 9 P_c = 13,7 Yielding in tension flange		P' remaine constant
6	1 - 46				See fig. 4
7	1 - 27				See fig. 4
8 - 17					See fig. 3
8S - 17S				Stub tests on girders 8 - 1	

All test girders were made of steel qualities with the designations SIS 1311 or SIS 1411 corresponding to the Swedish Steel Specifications. This means among others that the yield and failure stresses ought to be at least 2200 and 3700 respectively 2600 and 4400 kp/cm^2 (St 37 resp. St 44). Generally the real yield stresses were higher.

Beside of the usual measurements by mechanical dial gauges for vertical and lateral deflections the strain distributions over the beam cross sections have been measured by electrical strain gauges. The compression flange was braced against both lateral deflection and rotation, except for test 1:1 and 8 S - 17 S.

In some cases special measurements have been carried out. Some of the beams rested at one end on a two-point support in order to measure the overturning moment acting on the beam. The two parts of the support consisted of a loading cell each. Knowing the distance between the cells it was possible to get an estimate of the overturning moment and the degree of instability.

<u>All-over behaviour of girder and influence of web stiffeners</u>

Here it is chosen to describe test 1:1 from Table 1 and 2 in order to illustrate the all-over behaviour. A comparision with test 2:2, which is the same girder but now with stiffeners on it, gives the influence of web stiffeners.

Test 1:1 is concentrated in fig. 1, where the mid-span deflection is shown

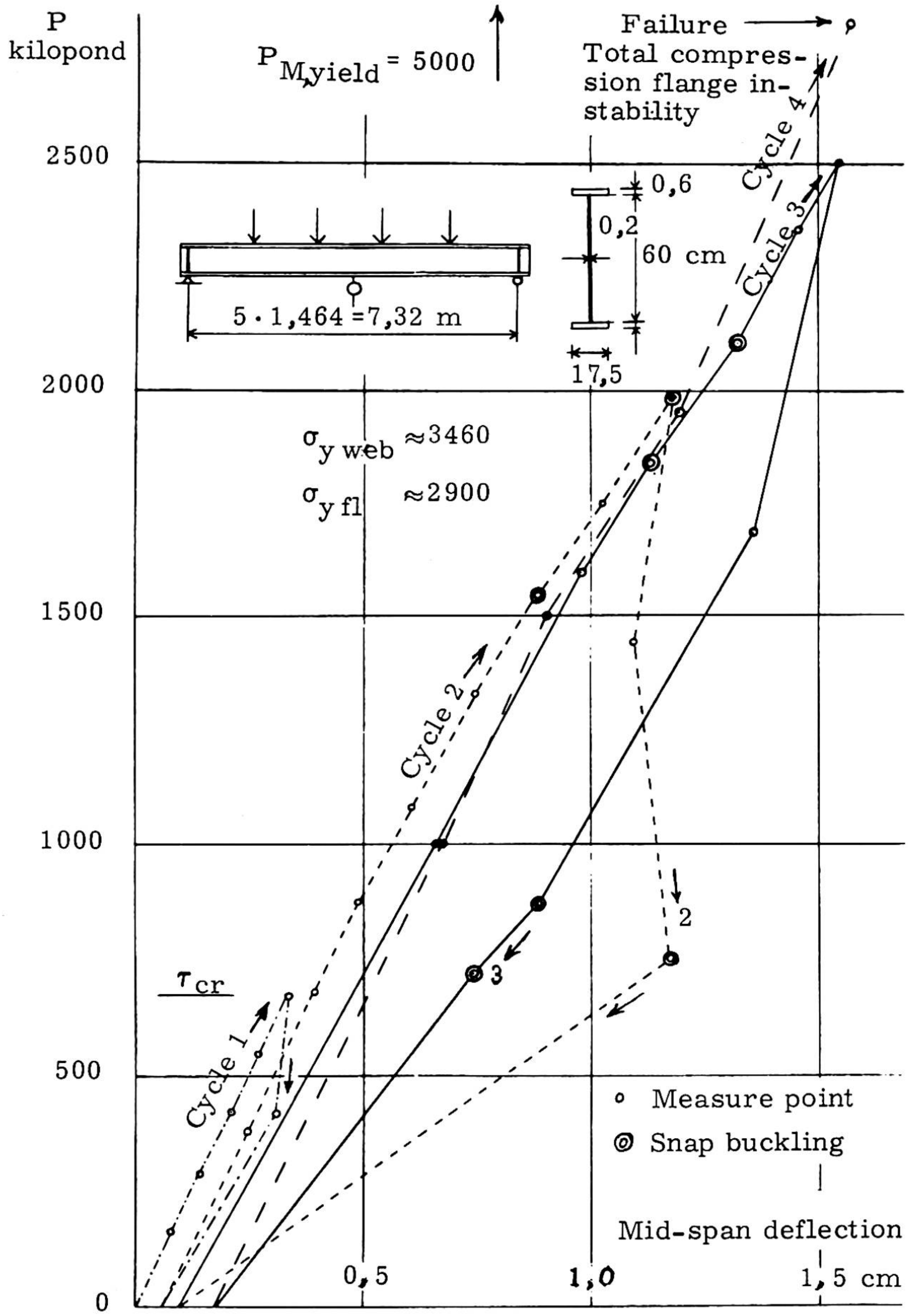


Fig. 1 Ratio between jack loads and mid-span deflection. Test girder 1.

versus the load of each loading jack. The theoretical span of the girder is 7,32 m, the depth 60 cm and the web thickness 0,2 cm. The slenderness ratio is thus h/d = 300. The loading were four point loads and the compressed flange may rotate, but not deflect laterally (cp. p. 5).

In the diagram the jack force giving τ_{cr} for the traditionally calculated buckling load is marked and also the force giving yield stresses σ_y following Navier's theory of bending. The girder was first loaded to τ_{cr}. Then unloaded and reloaded to 3 τ_{cr}. After another loading cycle to near under a supposed failure load it was finally loaded to collapse.

When the load passes τ_{cr}, nothing in particular happened and even passing 3 τ_{cr} had no influence even on this girder with stiffeners only at the supports at a distance of 7,32 m apart. Soon there-after it collapsed, however. This happened before reaching the compression yield stress and was a stability failure. Suddenly the steered compression flange rotated in its whole length and the transverse bending stiffness of the web was so little that a plastic zone developed in the web reaching from support to support.

A quite different behaviour was observed in the case of tests 3:1, 3:2, 4:3 and 4:4, where even in the case of a braced compression flange the web deflected laterally and caused the tension flange to warp and tilt.

Notwithstanding there was no indication of the importance of τ_{cr} during the test, it possibly had some influence when passing it at unloading. As marked in the diagram of fig. 1 one observed web snap buckling in three cases just at the level of τ_{cr}. Comparing this with fig. 2 which is taken from p. 657 of ref. [3] it may perhaps be said that it illustrates the branching conditions near the formal buckling load. Basler and Thürlimann state that if the plate at this stress level could be brought to a position on the dashed branches in quadrant 3, which are unstable equilibrium positions, it could then snap to either side of the reference plane and stabilize there. As indicated in fig. 1 snap buckling was observed also in several cases at over double the just mentioned stress level (but in other girder tests also at several other levels).

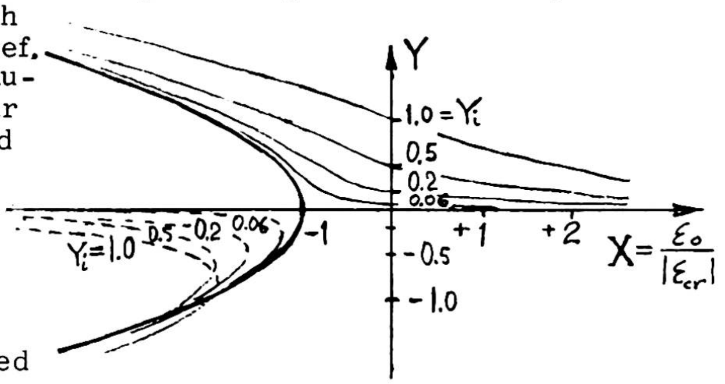


Fig. 2 Plate deflection Y versus applied edge strain X, ref [3].

The snap buckling at the unloading part of load cycle 2 has strongly influenced the deflection of the girder. This is often the case in snap buckling. The snaps are strongly dependant on the rate of loading and the energy releases increase with increased rates of loading. It was impossible to state if there was a simple and direct relation between the intensity of load and the snap.

As to calculation of the load causing flange yielding in bending the following comments may be of interest. The technical beam theory is based upon the assumptions of Hooke and Bernoulli. But Bernoulli s assumption of plane sections remaining plane after deformation is applicable only to a certain degree and the usual assumption that the cross section remains rigid during deformation does not hold. The consistency of these assumptions is a function of the slenderness and of the degree of stiffening applied to the web (web stiffeners) and to the flanges (bracing).

The different measurements of strain distributions over the cross sections showed that great transversal bending stresses existed in the web and the middle part of it was ineffective in taking up in-plane bending. Fig. 3 illustrates typical

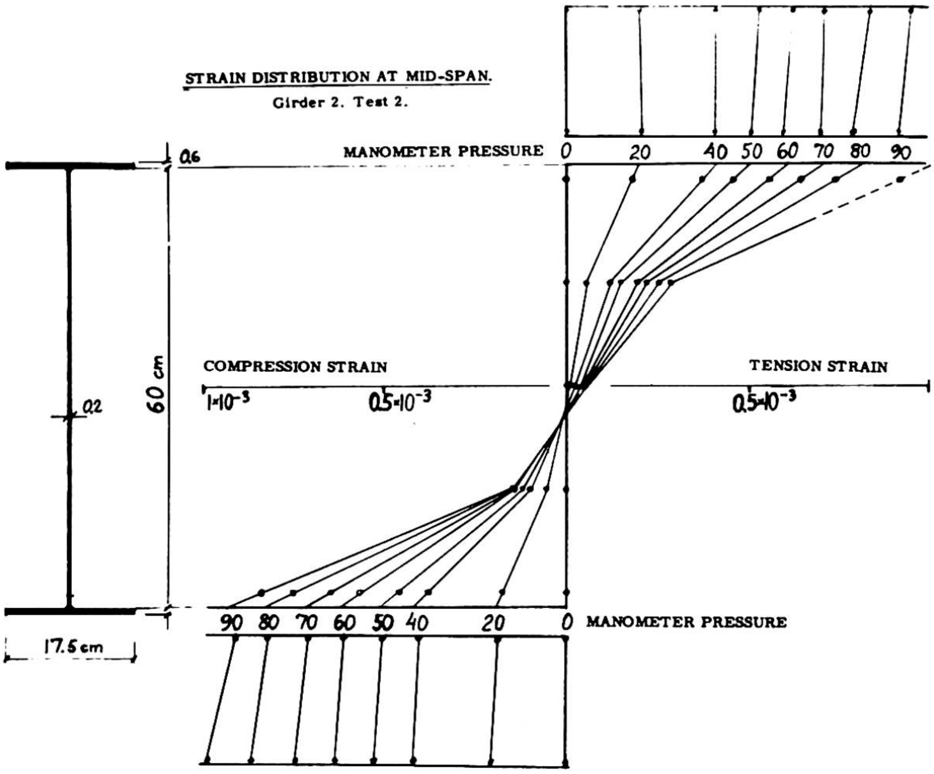


Fig. 3 Strain distribution at mid-span.

test-results. The test points marked on the figure show that the average strain in the web is much less than compared to a linear stress distribution. This is to be expected because of the initial and additional lateral deflections of the web. The relative weakness is especially typical for the compression side, but it has to be observed that a deviation from the linear theory also occurs at the tension side. This do not confirm the results of Basler and Thürlimann [3], who stated that only the compressed part of the web was not taking up bending stresses in full.

Test 2:2 compared to test 1:1 shows the influence of web stiffeners. The failure occured here not until the load was about 35 % higher and nearer to that giving yield stresses in the flanges. The failure reason was local buckling of the compression flange. The web stiffeners hold the tension flange so that there was no folding of the web (cp. test 3:1, 3:2, 4:3, 4:4 and even 4:5).

Web crippling

As shown the web stiffeners are useful to uphold the invariability of the cross section of the girder. They increase the bearing capacity by making tension fields possible and prevent excessive web buckling. Another reason, which often makes them necessary, is to prevent local web crippling.

At the supports they are practically inevitable. Between supports intermittent stiffeners could often be avoided, however.

Tests 8-17 are put together in fig. 4. They constitute a test series intended to show the influence of a point load on the web behaviour. The web thickness

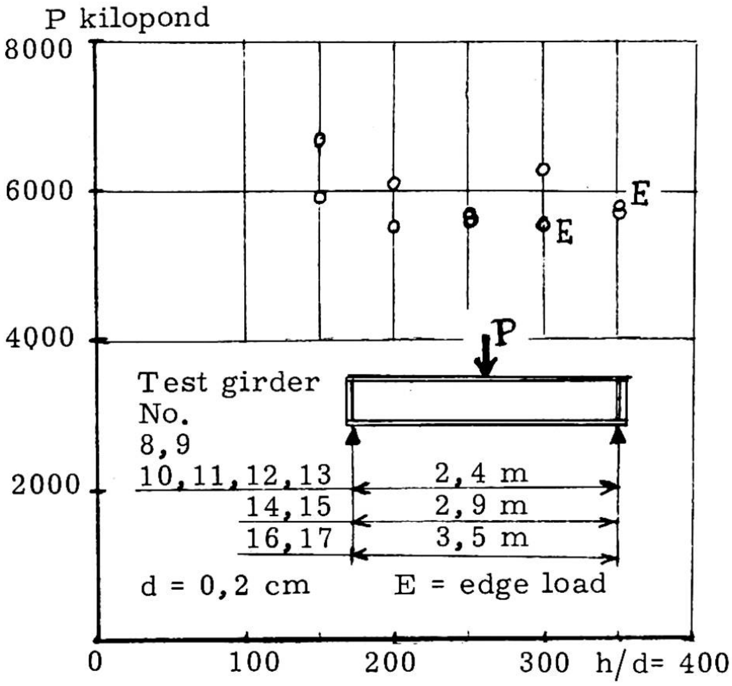


Fig. 4 Crippling loads for different web ratios.

was 0,2 cm. The depth varied from 30 to 70 cm and the flange areas from 0,6 x 10 to 1,5 x 10 cm^2. It may be seen in fig. 4 that the cripple load only varied between approximately 5500 and 6500 kp. The failure loads seem thus almost independent both of web depth and flange dimensions. Other tests show that they depend almost only on the web thickness. Even the span of the girder, and with it the bending stresses, seem to have a very small influence. Not until the bending compression stress in the web approaches the yield stress the influence seems to be more pronounced. From some other tests, no 3:3, 3-6, 4:7-9 and 5:2-3 it is seen that in these cases, however, the cripple load diminishes with increasing bending stress. Even the influence of the type of load transfer seems very small - either it was characterized by transmitting the point load through a half round or a short rectangular bar or through a 18 cm long plate.

Of course the results were quite different for tests on stub girders without any web stiffeners. If they were pressed directly over the supports web buckling took place according to the corresponding case of simple column buckling.

Previous tests have been made [4] and the cripple load was then given as $P_c \approx 0{,}9 \cdot 10^5 \cdot d^2$ with d in cm and P_c in kp, or $P_c \approx \sigma_s \cdot d^2/0{,}03$. The current tests also point on the fact that P_c is mainly depending on d, but perhaps the exponent should be smaller than the one used in this formulea.

When working out a theoretical estimate one usually starts from the theory of beams on elastic foundations. The flange is considered as the beam and the web as the elastic foundation. Considering the flange as a beam one reaches the formulae of Klöppel and Lie [5]. The beam thus reaches a certain "elastic length". In the elastic range the flange ought to function as a T-beam together with the nearest part of the web. Already with only a small part of the web co-operating, the web will have a dominating influence on the inertia of the "beam" and thus d is an important factor. This is the reason perhaps, why the load P_c greatly depends on d and not so much on the flange dimensions.

For the slender girders tested some uncertainities were unevitable. Owing to fabrication methods the flange was not centric over the web and the flanges were not plane. Sometimes the flange was so distorted that a proper contact between flange and jack was difficult to establish without filling in plates. Such excentricities of course initiated early buckling and crippling.

Web stiffeners and their heights.

Often the loads taken by the web with stiffeners at the supports only, are fairly large. Sometimes the point loads on the span are still larger or the total load is so large that using stiffeners is the correct solution.

Test 6 and 7, the results of which are concentrated in fig. 5, is a series treating on one hand the interaction of combined bending moment and shear forces as well as cripple loads, and on the other hand the length of any vertical web stiffeners.

It is seen that a lower bound for the measured points is near under the horizontal line $P_c/P_o = 1$, where $P_o \approx 9000$ kp, which ought to be cripple load for the unstiffened web (length of stiffeners = 0). The fact that this line is practically horizontal, confirms what just has been said about the cripple load being almost independent of the bending stresses. A bound to the right in fig. 5 is a vertical line corresponding to the yield moment.

It may also be seen that for stiffeners of the height h/3 to h/2 the cripple load will be higher, but in the case of large bending stresses only a bit higher than for no stiffeners. With stiffeners over the total web height the level of failure load is further increased. Usually the bound does here not any longer depend on the cripple load but on the shear force. The bearing capacity of the girder

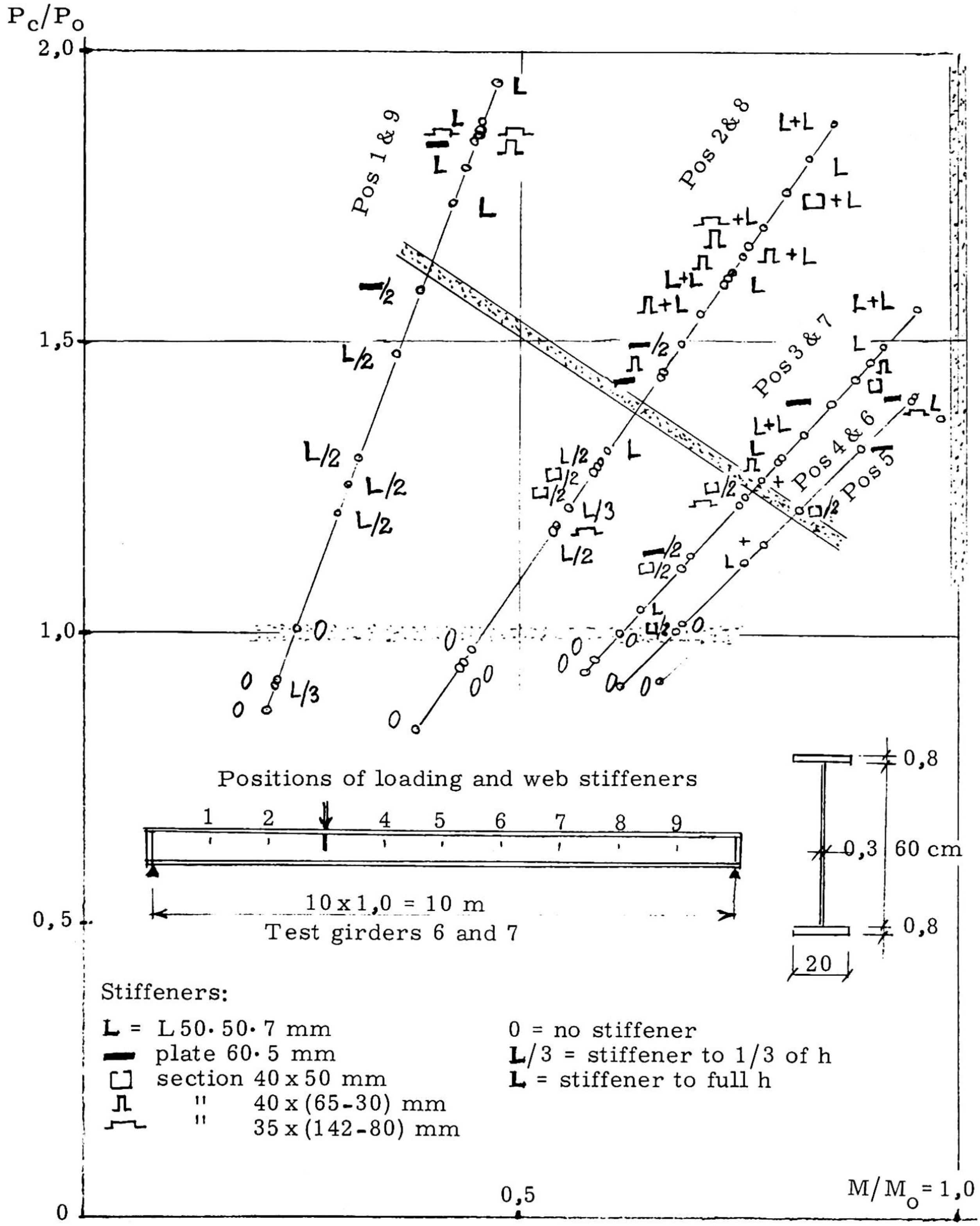


Fig. 5 Failure Loads P_c/P_o as function of M/M_o for different type of stiffeners.

is the sum of the shear force in the web and the load in the tension field between the stiffeners. There is an agreement with the calculations according to ref. [3].

The compression flange

The post-critical bearing capacity of the web is rather large. This holds true, however, only if the girder as a whole is able to support the load. Yielding of the flanges is of course a limit. Another is often the lateral or local buckling of the compression flange. Several tests on these phenomena have been made and reported. For the actual girders with web slenderness h/d = 150 to 400, tests were made to verify how to brace the compression flange and how to find the necessary bracing forces.

In connection with these problems it was necessary to penetrate which calculating model is the correct one for the stability behaviour of the compression flange. The question has been raised if the compression flange forces shall be considered as direction-invariant or if they point in the tangent direction to the center line of the flange.

Comment on the girder tests

Owing to the extremely thin dimensions and special methods of fabrication new factors stress to be of importance compared to those met with in conventional construction with rolled beams and heavier sections.

The following factors tend to be of interest and may have great influence on girder behaviour:

The inhomogenity of the material
The diviations of the real from the nominell dimensions
The presence of internal stresses due to fabrication methods
Owing to the fabrication methods initial deflections and lack of straightness are induced in the finished girder. The lack of straightness may be considerable.

The factors influence the test results and cause a wide divergence. In this way even girders of the same dimensions may show a difference in behaviour during the loading tests. Anyhow the increased instability risks call for additional bracing compared to girders with thicker dimensions.

Comparison between specifications

It is known that for slender webs the post-critical reserve in bearing capacity as characterized by the critical stresses according to the linear small deflection theory is greater than for thicker ones. This fact has resulted in a certain audacity among specification writes. As an example the beforementioned. temporary Swedish specifications allow stresses that for a slenderness ratio larger than 81 are 1,05 to 2,92 times those of the 1963 AISC specifications.

h/d	$\tau_{\text{temp Sw.}} / \tau_{\text{AISC}}$
81 to 221	$0,756 + 0,442 \cdot (h/d)^2 \cdot 10^{-4} =$ $= 1,05$ to $2,92$
>221	2,92

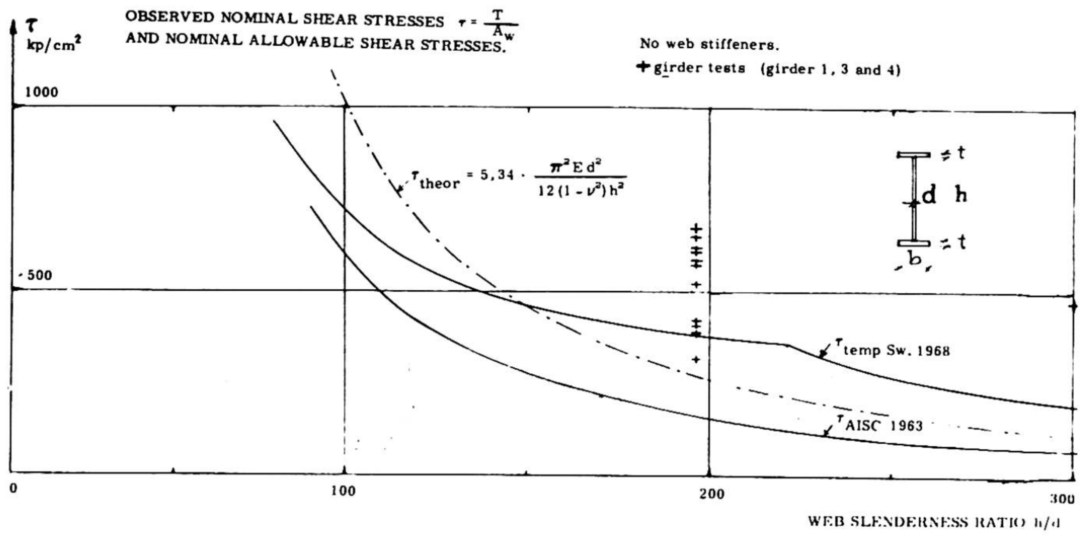


Fig. 6 Comparasion between theoretical, allowed and measured shear stresses.

Fig. 6 illustrates the comparasion. It has to be observed that the curves in the diagram apply to girders with unstiffened webs, e. g. $a/h = \infty$. Just at the web slenderness ratio h/d = 221 there is a minor anomaly in the Swedish temporary specifications, which depends on approximations in the simplifying curves used. On the diagram the nominal shear stress at failure for the beams 1, 3 and 4 without web stiffeners are marked. Some of the points lie rather low, but it has to be observed that in no case there occurred a shear failure. The maximum loads were limited be excessive lateral deflections, flange rotations or local web crippling under concentrated loads.

Despite this audacity as regards the allowable stresses the specifications have been and are still used for many important structures with hitherto good results as mentioned in the introduction.

*

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SUMMARY

The paper summarizes results from several tests on slender, welded steel plate girders with special considerations to web behaviour and web stiffening.

RÉSUMÉ

L'article traite brèvement les résultats d'essais sur poutres soudées avec l'âme mince et en considération spéciale du comportement des âmes et leur raidissage.

ZUSAMMENFASSUNG

Dieser Aufsatz beschreibt knapp die Ergebnisse der Versuche mit geschweissten, hohen Blechträgern mit besonderer Berücksichtigung des Stehblechverhaltens und dessen Aussteifungen.