Zeitschrift: Ingénieurs et architectes suisses
Band: 111 (1985)
Heft: 1-2

Artikel: Trapezoidally corrugated girder webs: shear buckling, patch loading
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DOI: https://doi.org/10.5169/seals-75605

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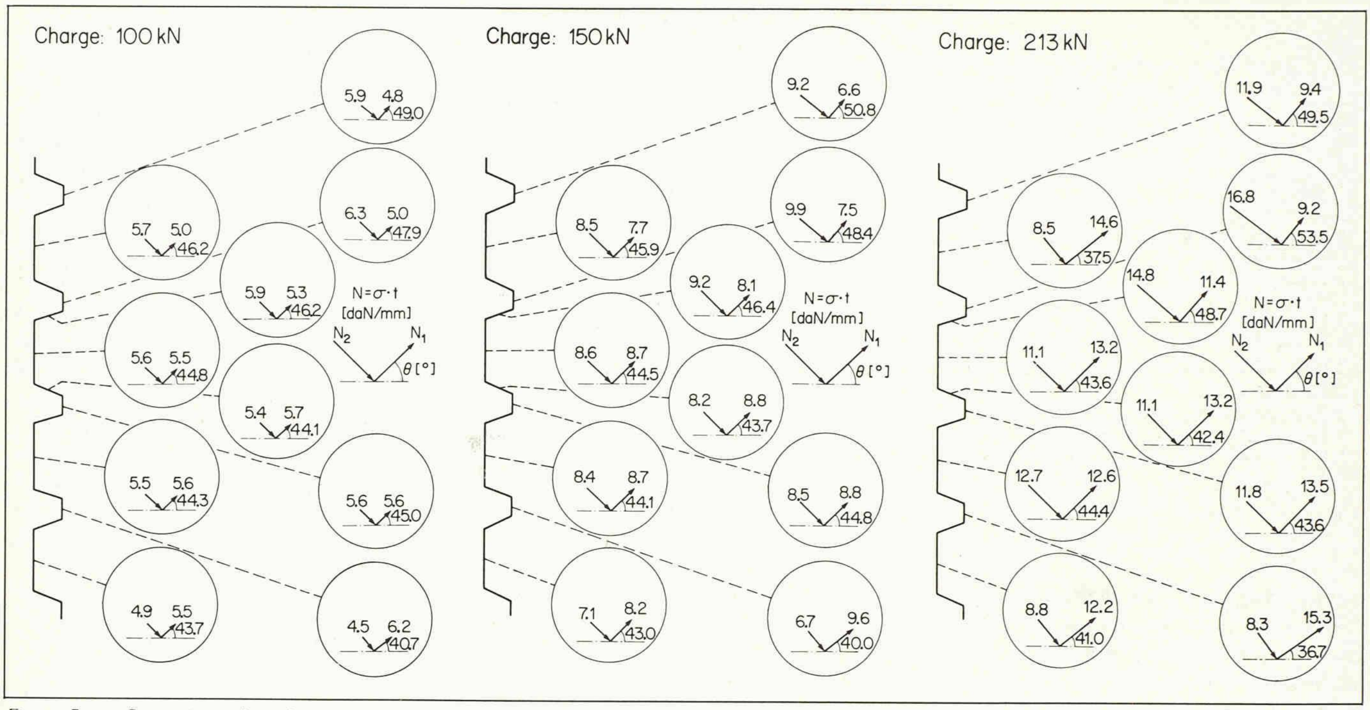

Figure 7. — Contraintes dans les axes principaux sous différentes charges (modèle n° 1).

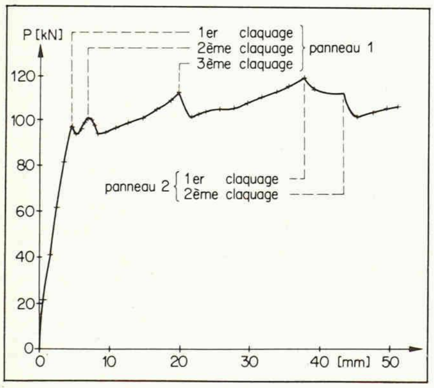

Figure 8. — Evolution de la flèche centrale au niveau de l'âme du modèle n° 3.

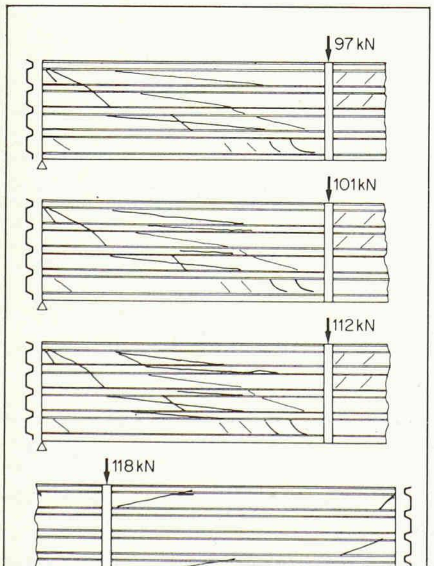

Figure 9. — Mécanisme de ruine du modèle n° 3.

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Trapezoidally corrugated girder webs

Shear buckling. Patch loading

by Allan Bergfelt, Bo Edlund and Luis Leiva, Göteborg

Introduction

Since 1961 welded steel I-girders with extremely thin, flat webs ($h/t \simeq 220$) without any intermediate stiffener, neither vertical nor horizontal, have been manufactured as a standard product in Sweden. The use of these girders increased quickly during the 1960's and continued during the first years of the 1970's, so that they dominated over rolled and ordinary welded girders for roof structures with light loads.

For the girders with slender, flat webs the design criterion was not based on the critical shear stress τ_{cr} but on the experimental maximum stress τ_F. This failure stress may be fairly well given by an extension and generalization of von Kármán's approximation to $\tau_F = \sqrt{\tau_y \cdot \tau_{cr}}$. For these slender girders the stress τ_F is clearly smaller than the yield stress τ_y.

In the mid 60's some production of girders with trapezoidally corrugated webs started. If the flat parts of the web were designed so wide that for that part in shear $\tau_{cr} \simeq \tau_y$, it was possible to calculate the web with adequate safety only from

Notations

- a distance between hinges of the failure pattern under patch loading
- f flange width
- c 'non-effective' width, see Fig. 10
- b, d, q, s, t web dimensions, see fig. at Eq. (6)
- h girder depth
- k buckling coefficient; k_u global buckling coefficient
- p_g "global buckling product" (coefficient)
- p proportion of stress combined with patch loading stress, see Eq. (10)
- D_x, D_y plate stiffness in bending, see Eq. (6); I_y moment of inertia
- N_{cr} critical shear force for global buckling; E modulus of elasticity
- P patch load; V shear load; M bending moment
- σ_y yield stress (σ_y^w for the web material; σ_y^{fl} for the flange material)
- τ shear stress; τ_d design shear stress
- τ_{cr} critical (bifurcation) shear stress
- τ_g global buckling stress
- τ_F experimental failure stress, cf. Eq. (1)
- $\hat{\tau}$ maximum shear stress defined by the interaction formula Eq. (11)

τ_y. These girders also had the advantage to be stiffer during handling than girders with flat webs. Since the authorities demanded that girders with flat webs must have double-sided fillet welds, while one-sided welding was sufficient for the trapezoidal web, manufacturing costs were about the same for both types of girders.
The girders with trapezoidally corrugated webs thus increased their part of the market and since the mid 70's they dominate the Swedish market for small and medium span steel roofs, at least in the regions of Sweden which are easily reached with transport from the girder factories.

Test Series

As there was an interest to increase the span of the girders and still use very thin-walled web plates, the Division of Steel and Timber Structures at Chalmers University of Technology (CTH) in 1981 started research into the behaviour of girders with trapezoidally corrugated webs as a part of its general research program within the field of thin-walled structures.
A pilot test series (Series L) was performed. It consisted of three girders each of which had a 2 mm web on half the girder length and a 2.5 mm web plate on the other half. The dimensions of the girders are shown in Fig. 1 and it is seen that 6 almost square girder parts could be tested: 3 with 2.0 mm web and 3 with 2.5 mm web, each kind with a girder depth of 1.0, 1.5 or 2.0 m. Two equal test loads were applied over the stiffeners in the central part of a test girder and so each test panel had an almost constant shear force.
Of course, the panel with 2.0 mm web plate buckled first and after unloading it was strengthened with a diagonal bar; then the test could proceed by focusing the interest on the panel with 2.5 mm web plate. The tests and their results are described in detail in Ref. [1][1]. The results indicate that there might be some influence of global buckling at least for the deepest girders. Therefore it was decided to test both small depth girders and very deep girders.
(Here *local* buckling means that a buckle occurs in one of the flat subpanels—with b = 140 mm in the test girders of series L and B—and *global* buckling means that a larger portion of the web—at least four flat subpanels and the intermediate inclined parts—are involved.)
The low girder test series (Series B) consisted of four 600 mm deep girders from the normal production of a manufacturer (Ranaverken AB). As there could certainly be no influence of global buckling for these girders it was decided to use the opportunity to study *both* the influence of load introduction through one or two stiffeners *and* the influence of initial imperfections. Two of the girders, B1 and B4, were thus allowed to have initial shape imperfections of the web profile geometry (especially near the butt welded joint). These imperfections were so large that these two girders should have been rejected in a normal delivery. (Only girder B4, however, had a much lower failure load than the others and in this case the panel which failed was strengthened with a diagonal bar and the girder was tested again, now called girder B4b, and then the buckling occurred in the other panel, which had only a small imperfection.)
The measured dimensions of the test girders are given in Fig. 2. A report of the tests is to be published, Ref. [2]. The girders of test series L and B were fabricated according to the standard production with webs of 2 or 2.5 mm plate and trapezoidal geometry having 50 mm deep folds and 140 mm wide flat parts. Very deep girders—of more than 2 m depth—are inconvenient to test and therefore model girders were produced for the next series.

[1] See references at the end.

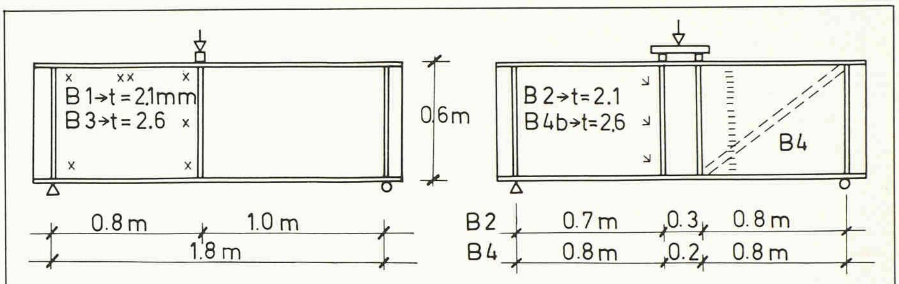


Fig. 2. — Main dimensions of test girders, series B (with strain gauge locations marked).

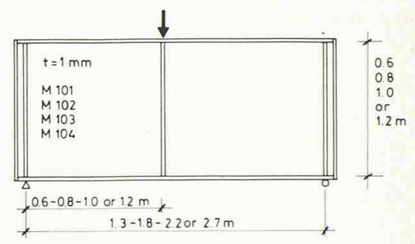


Fig. 3. — Main dimensions of test girders, series M.

The third series, the model girder tests (Series M) had webs of 1.0 mm plate and a longitudinal section characterized by 15 mm deep folds with flat parts only 70 mm wide. The girder depths were 600, 800, 1000 and 1200 mm, see Fig. 3.
The detailed report of test series M is under preparation, but some preliminary results will be given here in the section "Test results".

Calculations

There are various limiting stresses, and the different influences may interact so that failure may be premature. The tests are planned and performed with the intention that shear stress, and the associated buckling, is intended to be the reason for failure. Near the points of loading it is almost inevitable, however, that normal stresses have an influence, too.
It is presumed that the shear stress is evenly distributed over the total depth of the web.

$$\tau = \frac{V}{th} \tag{1}$$

a) The stress must theoretically be less than the yield limit τ_y.

$$\tau \leq \tau_y = \frac{\sigma_y}{\sqrt{3}} = 0.577\ \sigma_y \tag{2}$$

b) The ideal (local) buckling of a plate of the web corresponds to

$$\tau_{cr} = \frac{k\pi^2 \cdot E}{12\ (1-\upsilon^2)\ (b/t)^2}, \tag{3}$$

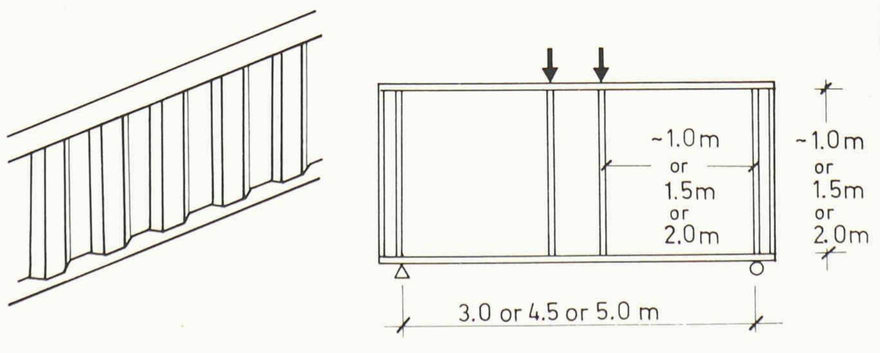


Fig. 1. — Main dimensions of test girders, series L.

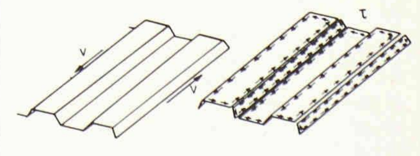


Fig. 4. — Part of trapezoidal panel under shear.

where k = 5.34 for a long plate with hinged edges and 8.98 for fixed edges.

Assuming ideal buckling and hinged edges (which gives a minimum value) you will have

$$\tau_{cr}/\sigma_y = \frac{4.83}{\lambda_w^2},$$

$$\text{where } \lambda_w^2 = (b/t)^2 \cdot \sigma_y/E \qquad (4)$$

c) Assuming that the stresses in a plate might be "anchored" at the edges you may calculate the average failure stress according to an extension and generalization of von Kármán's formula. The edges *here* are the fold lines of the trapezoidal shape and are hardly stiff enough to correspond fully to that assumption, but *if* we still keep the assumption the result is the following

$$\tau_F = \sqrt{\tau_{cr} \cdot \tau_y} = \sqrt{0.577 \cdot \tau_{cr} \cdot \sigma_y}$$
$$(\text{valid if } \tau_{cr} < \tau_y) \qquad (5)$$

$$\tau_F/\sigma_y = \sqrt{0.577 \cdot \tau_{cr}/\sigma_y} = 1.66/\lambda_w$$

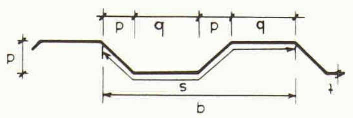
d) When the depth and length of the web are very large compared to the widths of single plates in the trapezoidal folded plate structure it might be possible that there appears a long-wave buckle over a larger portion of the web. This is called "Global buckling".

Theoretical calculations, Ref. [3], [4], result in a formula for the critical shear force N_{cr} per unit length that may be written:

$$N_{cr} = k_u \cdot (D_x \cdot D_y^3)^{1/4}/h^2 \qquad (6a)$$

where

$$D_x = \frac{q}{s} \cdot \frac{Et^3}{12} \text{ and } D_y = \frac{EI_y}{q} \qquad (6b)$$



with $I_y = 2bt(d/2)^2 + 2\sqrt{2} \cdot d^3 t/12$ for a trapezoidal shape with 45° folds.

The result of a calculation of k_u, where the wave-length of the buckle is taken into account as well as the degree of edge restraint of the folded plate, is given in Ref. [4]. The result is k_u = 32.4 for hinged edges and 60.4 for fixed edges. (In most of the formulas given E is used instead of $E/(1-\upsilon^2)$ even for the plate stiffness D_x.)

The critical shear stress for global buckling thus is

$$\tau_g = N_{cr}/t = 60.4 \cdot (D_x \cdot D_y^3)^{1/4}/h^2 t \qquad (7)$$

or with D_x and D_y inserted:

$$\tau_g = N_{cr}/t = \frac{60.4}{\sqrt[4]{96}} \cdot \underbrace{\left[\frac{q}{s} \cdot \left(\frac{\sqrt{2}}{3} + \frac{b}{d}\right)^3 \cdot \left(\frac{d}{q}\right)^3\right]^{1/4}}_{p_g} \cdot \frac{d\sqrt{td}}{h^2} \cdot E \qquad (7b)$$

The Swedish Code for Light-Gauge Metal Structures [7] prescribes values for τ_d, which means the design value of τ within the method of partial safety factors. From this value allowable values may be determined with regard to load factors and safety factors.

Comparing the previous theoretical values and the design values it is seen that $\tau_d \simeq 1.15 \cdot \tau_y$ and also $\tau_d \simeq 1.15 \cdot \tau_{cr}$, where these influences are governing. On the other hand $\tau_d \simeq 0.84\, \tau_F$, probably in order to avoid too large deformations. Specifications with respect to τ_g are given only for the case of an orthotropic plate with hinged boundaries.

Test Loading

The test loads were applied by displacement-controlled hydraulic jacks and the shear force was determined by measuring the support reaction using force transducers. The applied load dropped almost immediately when a buckle appeared in the web. With continued deformation the load increased again. The deflection was measured at mid-span. The loading rate was 1 mm deflection increase in about 6 minutes, which corresponds to a support reaction increase ΔR = 8 to 10 kN/min for the various girders during loading up to the first buckle. When the load had dropped after each buckle the loading speed was changed during the new load increase.

Strain-gauges were used to determine *both* the shear stress distribution along the depth of the girder *and* the stresses at both the flat and the inclined parts of the web. The aim was to see if there was any difference between the stresses in those two parts, especially near the flanges. The stress distributions are shown in the full reports, but not in this short version.

The Results

The test results are collected in Table 1. An example of a load-deflection diagram is shown in Fig. 5, Fig. 6 and Fig. 7a, b and c illustrate the buckling pattern. It was very evident that for the *low girders* with panel length larger than the girder depth the first buckle appeared close to the loaded flange and the stiffener at the loading point. At this intersection between flange and stiffener there are stress concentrations, welding stresses and normal stresses due to girder bending. (Girders B1, 4b, 3 and 2.)

For the *medium depth* girders with square panels, where the bending stresses were small, the first buckle appeared either close to the load or near the support. (Girders L1A, 1B, 2A and 2B.)

For the *deep girders* already the first buckle forms with a lound bang and a large buckle appears diagonally across a trapezoidal wave, thus joining two initially plane parts of the web, and flattening the inclined part between them. In

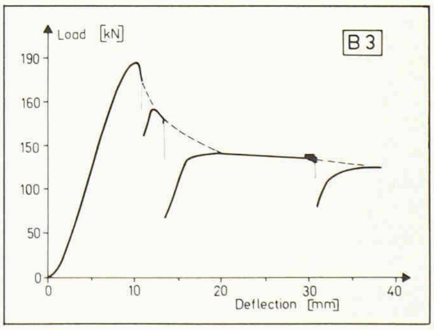


Fig. 5. — Load versus deformation diagram of girder B3.

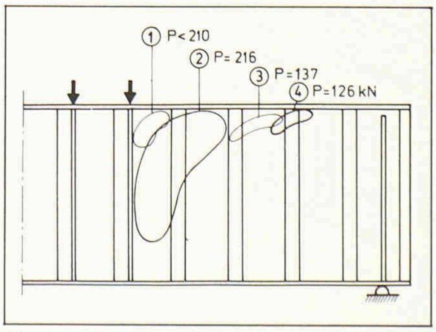


Fig. 6. — Buckling propagation of girder B4b.

TABLE 1: *Results from the three shear test series L, B, and M.*

Girder	t mm	h mm	σ_y N/mm²	λ_w	τ_d^{loc} N/mm²	R_{max} mm	τ_F N/mm²
L1A	1.94	1000	292	2.69	152	280	140
L1B	2.59	1000	335	2.16	217	502	201
L2A	1.94	1500	282	2.64	150	337	112
L2B	2.54	1500	317	2.14	207	564	150
L3A	2.01	2000	280	2.54	154	450	113
L3B	2.53	2000	300	2.09	201	775	155
B1	2.10	600	341	2.65	181	208	165
B4	2.11	600	363	2.72	187	183	145
B4b	2.11	600	363	2.72	187	217	171
B3	2.62	600	317	2.04	212	246	156
B2	2.62	600	315	2.04	211	273	174
M101	0.99	600	189	2.10	127	97	89
M102	0.99	800	190	2.11	126	144	99
M103	0.95	1000	213	2.23	134	154	85
M104	0.99	1200	189	2.10	127	192	88

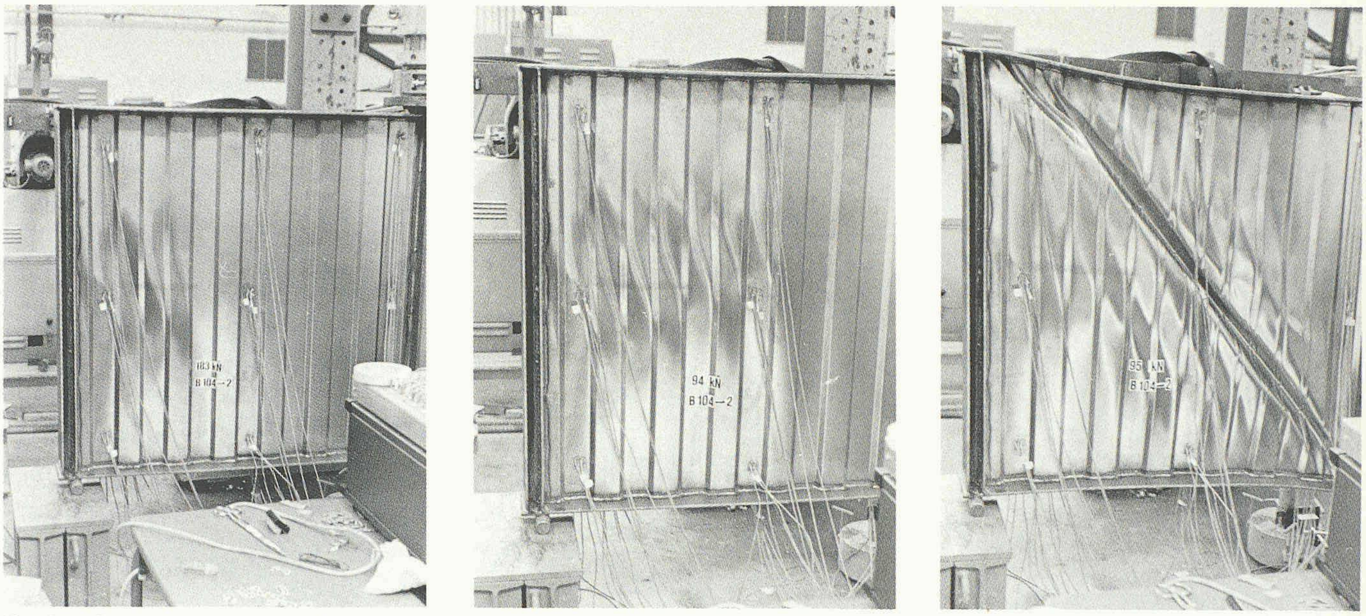


Fig. 7. — Buckling propagation of girder M104 : a) *Local-zonal buckle ;* b) *Global buckling ;* c) *Tension field.*

order to distinguish this buckling from both *local* and *full global* buckling it may be called "*zonal* buckling".

The zonal buckling starts somewhere in the middle of the panel. (Girders L3A, 3B and M103, M104.) With increased deformation during reloading buckling occurs in several web waves. These waves may form sequentially and finally end up as a type of global plastic buckle (c.f. girder M104, Fig. 7). At the end of the tests there was— most pronounced in series M—a deep fold diagonally across the whole panel, indicating a tension field action (c.f. Fig. 7c).

An exception from the buckling pattern for low girders was, of course, girder B4 which as said had large initial deformations. Here the buckle stretched over the inclined part with the weld joint and over the two adjoining flat parts. In order to preserve a picture of the original shape of the webs—which was of importance especially for the series B where girders B1 and B4 had large initial deformations —a 40 mm deep strip of concrete was moulded on the lower flange directly to the web. (The strip was, of course, removed before the tests.) With this mould as a guide the web shape was then drawn on a paper with a 2 mm thick pencil ; that line thus almost coincided with the actual longitudinal section of the web.

The ideal shape of the webs of series L and B and the most deformed part of girder B4, as well as the web shape of series M are drawn in Fig. 8.

The main reason for the investigation was to check if there might be any influence of global buckling on the failure. Table 1 shows that this influence does not seem to be important in these tests. Using the highest value of k_u given above, i.e. 60.4, in the formula (7b) this becomes

$$\tau_g = 19.3 \cdot p_g \cdot E \cdot \frac{d\sqrt{td}}{h^2} \qquad (7c)$$

Here "the global buckling product" p_g, defined in formula (7b), will be close to 0.5 for the web shape in question. The ideal web shape with 50 mm deep waves will give $p_g = 0.52$, the various imperfect waves of girders B1 and B4 will result in $p_g = 0.48 - 0.53$, and the model webs have $p_g = 0.54$.

In Fig. 9 the values τ_F/τ_d are plotted for all the tests, and the curves for τ_g correspond to τ_d = 152 and 185 N/mm² for the 2 mm web, to τ_d = 210 N/mm² for the 2.5 mm web, and to 130 N/mm² for the 1 mm web. (In Fig. 9 the horizontal scale for the M series tests corresponds to an equivalent girder depth of a girder having the profile geometry of series L and B and with web thickness 2.5 mm. The equivalent girder has the same relative global buckling stress τ_g/τ_d, with τ_g from Eq. (7c), as the actual model girder ; this means that the curve for global buckling coincides with that of the 2.5 mm webs. To achieve this it is also necessary to take the differences in yield stress into account, for example by making the intersections with $\tau/\tau_d = 1$ equal.) It is seen that almost all values τ_F/τ_d range from 0.7 to 0.93. This is what ought to be expected as in buckling situations τ_F in practice is about 0.7 to 0.8 τ_d in the region where $\tau_{cr} \simeq \tau_y$. The global buckling thus seems to have only a very small reducing influence on the buckling load even for these very thin-walled webs in rather deep girders. (It may be observed that the "zonal" or global buckling of deep girders will not reach all over the depth. One may thus suspect that the dependence of h^2 in Eq. (7c) is not valid for very deep girders, but only up to some limit depth.)

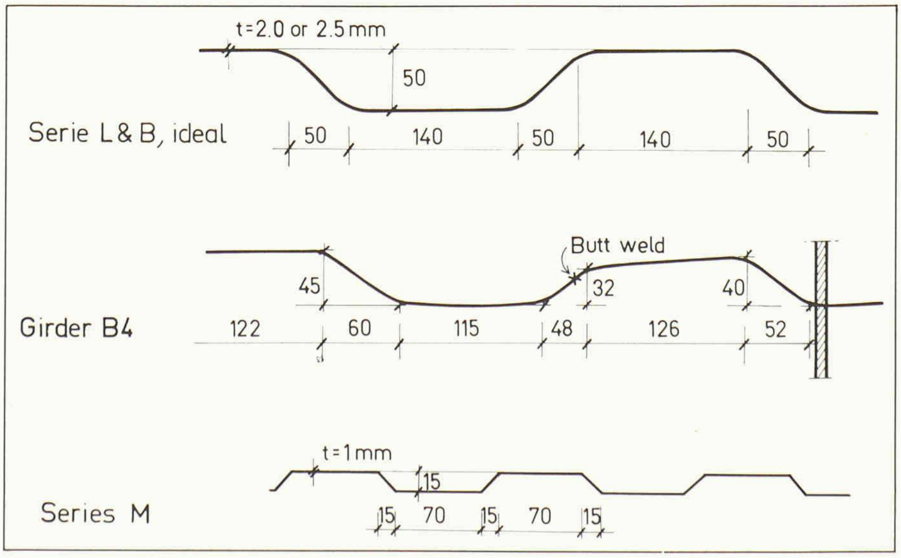


Fig. 8. — Longitudinal sections of the webs. Ideal shape of girders in series L, B, and M, and the shape of the most imperfect part of the web of girder B4.

Patch Loading

In a supplementary series some tests with patch loading were also performed. The load was applied on the top flange, located either over the central portion of a flat part of the web or over an 'inclined' part.

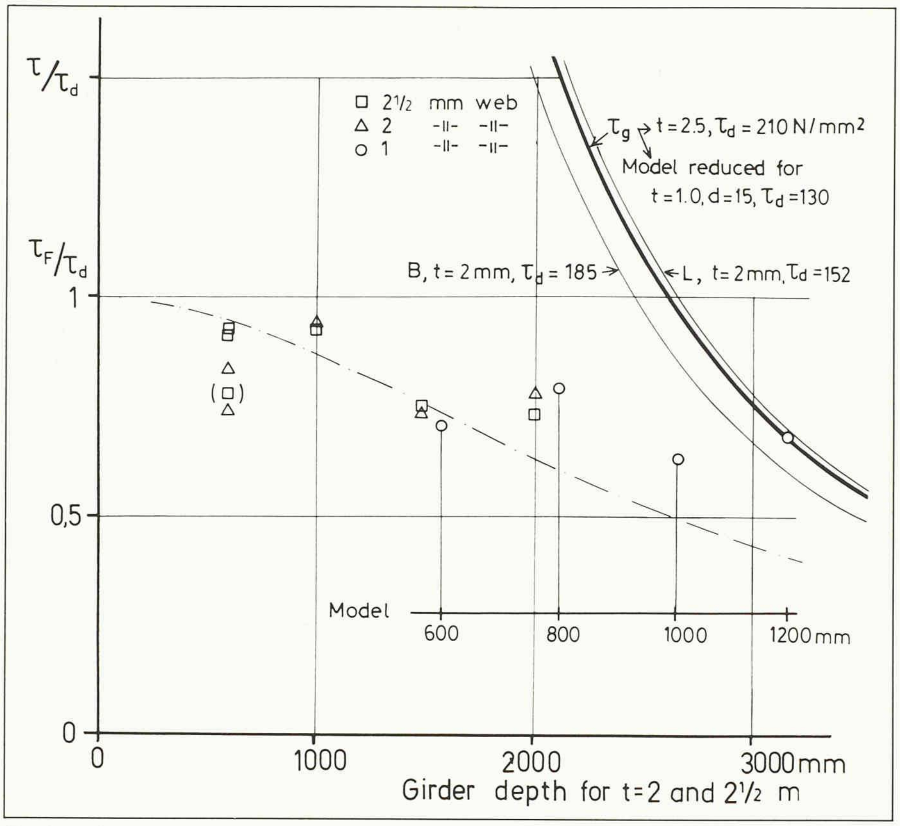


Fig. 9. — Shear stress at failure compared to critical stress τ_g for global buckling.

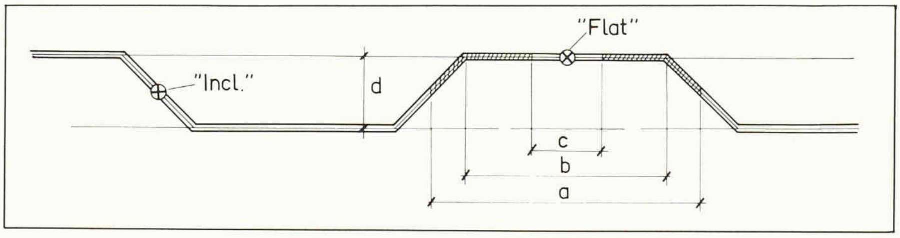


Fig. 10. — Location of patch loads. Web section. Two loading cases: Either above the flat part of the web or above the inclined part.

The calculation may be performed according to the three-hinge-flange theory which was given for a special case in Ref. [5, p. 76] or from an extension of the more general formulas given in Ref. [6, p. 6].

$$P = \sigma_y^w \cdot t_w \cdot a \; ; \quad M = \frac{P}{2} \cdot \frac{a}{4} - M \; ;$$

$$M = \sigma_y^{fl} \cdot Z \qquad (8)$$

(with $Z = \dfrac{f \cdot t_{fl}^2}{4}$ where f is flange width)

The effective width of the flat part b at buckling will be $\sim 50\,t$ and thus $c = b - 50\,t$. If the total length "a" of the compressed part of the web ends in the inclined parts of the web, the equation corresponding to the two first equations of the Eqs. (8) will be

$$P = \sigma_y^w \cdot t_w \left[\sqrt{2}(a-b) + b - c\right] \qquad (8a)$$

$$M = \sigma_y^w \cdot t_w \left[(a-b)\frac{\sqrt{2}}{2}\left(\frac{d}{2} + \frac{a-b}{2 \cdot 2}\right) + (b-c)\,\frac{1}{2}\left(\frac{b}{2} - \frac{b-c}{2 \cdot 2}\right)\right] - M \qquad (8b)$$

Introducing M in Eq. (8b) will result in

$$a^2 = 2\sqrt{2} \cdot \frac{\sigma_y^{fl}}{\sigma_y^w} \cdot \frac{f \cdot t_{fl}^2}{t_w} + \left(1 - \frac{\sqrt{2}}{2}\right) b^2 + \frac{2}{\sqrt{2}} \cdot c^2 \qquad (9)$$

It ought to be observed that if the flange is subject to an even stress $p \cdot \sigma_y$ due to bending of the girder, the stress σ_y^{fl} in the preceding formulas (8, 9) has to be substituted by

$$\sigma_{yp}^{fl} = (1 - p^2) \cdot \sigma_y^{fl} \qquad (10)$$

The formulas corresponding to Eq. (8) will be a little more complicated when the load is located at the inclined part of the web. The numerical calculations even for this case however, have been performed (by the third author) and for the tested cases the results will be

Girder	Flat Incl.	Calc kN	Test kN
P1, P3, P4	F	167	149, 152, 168
P2	I	179	170
P6	I	153	124

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Conclusions

Even for the extremely thin trapezoidal webs in the deepest tested girders the global web buckling does not seem to affect the buckling behaviour to any greater extent. It seems necessary, however, to make some more tests on very deep girders before we have covered a range of web slenderness ratios large enough for a more definite statement.

Even if the interaction is not great, some influence might be seen. An interaction formula

$$\frac{1}{\hat{\tau}} = \frac{1}{\tau_d} + \frac{1}{\tau_g} \qquad (11)$$

has been tested and the corresponding curve is drawn in Fig. 9. It is seen that Eq. (11) gives a fair prediction of failure stresses τ_F for girders with normal fabrication tolerances and with depths up to just below 2 m, but that it is too conservative for deep girders. For such girders, interaction effects thus seem to be much less than predicted by Eq. (11). The coefficient k_u in the global buckling formula was previously, [3], [4], given as values varying from about 30 to 60.4. In our evaluation the largest value 60.4 was used. The increase of the ultimate (or maximum) load compared with that of a flat web in shear is considerable. An accepted formula for a tension field of a girder with an extremely slender web is

$$T_{max} = \frac{1}{2} \cdot \sigma_D \cdot t \cdot h \cdot tg\phi \; ; \; \sigma_D \simeq \sigma_y \; ;$$

$$\phi = \frac{1}{2}\, arctg\, \frac{h}{l} \qquad (12)$$

For the girders tested, Eq. (12) predicts a load carrying capacity that is only about 20-50% of the failure loads of Table 1. A calculation of failure patch loads is possible following the three—hingeflange theory [5], [6].

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Vie de la SIA

150 Jahre SIA Bern

Giornate Tage Bern
Journées 7/8.6.1985

Après les fêtes et le répit de fin d'année bien mérité, le comité d'organisation des journées SIA des 7 et 8 juin 1985 à Berne s'est remis à l'ouvrage avec entrain. Les préparatifs sont déjà bien avancés et presque tout a pris forme.

Si vous désirez passer deux plaisantes journées à Berne, veuillez prêter attention à l'invitation qui figurera, avec le programme détaillé et la formule d'inscription, dans *Ingénieurs et architectes suisses* n° 5 du 28 février prochain.

Toutes les manifestations sont organisées sous forme de rencontres informelles, ce qui présente l'appréciable avantage de pouvoir choisir, dans un programme très riche, celles qui semblent avoir le plus d'attrait. Les journées SIA de Berne vous offrent la possibilité de choix sur mesure selon votre humeur et vos goûts, sans rien débourser. Toutes les manifestations du vendredi et les excursions du samedi sont comprises dans la carte de fête.

Celle-ci peut être commandée par le bulletin d'inscription qui sera encarté au n° 5 d'*Ingénieurs et architectes suisses, le 28 février prochain.*

Le comité d'organisation

Après les scrutins sur l'énergie du 23 septembre 1984

Quelques propos tardifs

La bataille est gagnée, les fumées dissipées. Partisans et adversaires des initiatives ont dressé leur bilan interne et public.

Satisfait du résultat de ces votations, le comité central de la SIA se rend compte que les problèmes n'en ont pas pour autant trouvé leur solution. Le côté libéral qui l'emporte doit être conscient de la responsabilité accrue qui lui incombe de ce fait. Les architectes et les ingénieurs se sentent tout particulièrement mis en cause.

L'élucidation de cette problématique dépasserait le cadre de ces modestes propos. Il s'agit maintenant en premier lieu de régler quelques points restés en suspens.

Diverses correspondances, approbatrices ou critiques, ont en effet apporté un important complément de réflexion aux «Principes concernant une politique énergétique suisse: le point de vue de l'ingénieur» publiés par le comité central[1]. Elles ont été remises à la Commission SIA pour les problèmes d'énergie qui, après réétude des thèses présentées, contactera les correspondants.

Selon une lettre émanant des milieux économiques distributeurs de gaz naturel, l'affirmation que «cette énergie ne nous arrive qu'en quantités limitées» pourrait donner lieu à des interprétations erronées. Le correspondant nous prie de préciser que notre système d'approvisionnement serait en mesure, sans autres investissements, de fournir le double des quantités livrées en 1983. En outre, des mesures appropriées en permettraient un accroissement supplémentaire sensible. Le réseau de distribution de gaz naturel alimente aujourd'hui des régions étendues du pays. Dont acte.

Deux autres lettres et des discussions en table ronde portent sur une question de procédure: la SIA doit-elle prendre parti dans des questions politiques d'actualité, et dans l'affirmative comment doit-elle se faire une opinion? Cette problématique est reprise en détail dans la lettre reproduite ici.

Le comité central se félicite de cette amorce de discussion touchant à une question d'importance, qui sera d'ailleurs prochainement abordée par le comité central et par la conférence des présidents avant d'être reprise par l'assemblée des délégués.

Sans préjuger du résultat de ce débat, voici quelques considérations émanant du comité central. Celui-ci reste persuadé — et le récent sondage y relatif le confirme dans cette idée — que la grande majorité des membres SIA accueillerait favorablement l'idée de voir la SIA se prononcer davantage sur des questions d'actualité même lorsqu'elles comportent des aspects politiques. Cela exigerait certes que l'on pût agir assez rapidement, en excluant pour des raisons de délais la consultation générale des membres. Il est d'ailleurs parfaitement normal que les organes directeurs de l'association disposent d'une certaine marge de manœuvre, en particulier lorsque les statuts le prévoient. De l'avis du comité central, il va de soi que de telles appréciations devraient être autant que possible étayées par l'opinion des membres quand les délais le permettent.

[1] Ingénieurs et architectes suisses n° 14 du 5 juillet 1984

Le problème perd une bonne part de son acuité lorsque de tels points de vue impliquent sans ambiguïté qu'ils émanent d'une majorité de délégués ou de membres. Le comité central a respecté ce principe en se prononçant sur la question de l'énergie en précisant qu'il s'agissait de son propre point de vue. En précisant dans le titre qu'il s'agissait du point de vue de l'ingénieur, on a peut-être bien involontairement pu prêter le flanc à la critique. Le lecteur distrait pouvait en effet en tirer des conclusions hâtives. Il serait de mise à l'avenir de nuancer davantage les questions délicates. Les réactions mettent toutefois en évidence que cette appréciation correspondait en fait fort probablement à l'avis de la grande majorité des membres SIA.

En ce qui concerne la lettre reproduite ici, on relèvera encore que c'est avec l'accord exprès des auteurs que sa publication n'intervient qu'aujourd'hui. Il s'agissait en effet de reporter le débat de fond sur les questions de procédure, pour éviter de le voir s'engager avant le scrutin sur une voie peut-être indésirable.

Nous remercions les auteurs de leur compréhension. La discussion est ouverte.

A. Jacob, Dr sc.techn.
Président de la SIA

Le comité central se plaît à prendre des risques!

Observations sur la publication, par le comité central de la SIA, des «Principes d'une politique énergétique suisse: le point de vue de l'ingénieur» (IAS n° 14, du 5 juillet 1984).

La Commission SIA pour les problèmes d'énergie a élaboré un «papier» qui recueille l'approbation complète du comité central. Les organes dirigeants de l'Association décident aussitôt de publier ces thèses et de les présenter à l'opinion publique comme celles «de l'ingénieur».

En guise de conclusion — et c'était apparemment le but visé — on rejette les deux initiatives devant être soumises au souverain en automne et portant sur l'approvisionnement en énergie et l'avenir sans centrales atomiques.

Les centrales nucléaires, les solutions de rechange, l'environnement: ce n'est pas ici le lieu d'en débattre. En revanche, on peut observer ce qui suit quant à la *procédure* appliquée:

1. Par le net rejet des deux initiatives, le comité central donne une *consigne de vote*. Or, rien de tel n'est prévu par les statuts, ni sous la rubrique des buts sociaux (art. 1-2), ni sous celle des tâches du comité central (art. 34 ss). Tout au contraire, dès qu'il s'agit de