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Choice of Steel Quality of Steel Bridge Girders with Regard to Support Forces during Launching

Choix de l'acier de poutres de ponts métalliques en relation avec les forces d'appui lors du lancement

Wahl der Festigkeitsklasse von Brückenträgern im Zusammenhang mit den Auflagerdrückern beim Einschieben

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Large steel girder bridges in one or several spans are often erected by launching. The loading cases which occur during the launching procedure are very different from those of the final structures.

The support forces at the launching rolls are concentrated loads acting on the bottom flanges of the main girders of the bridge. During launching these loads move along the whole length of the girder, which means that very large concentrated loads are acting on the flange and an at least partly unstiffened web (as there is of course no economical possibility to arrange stiffeners in extremly small distances). It follows that there may be a risk for local deformation of the flange at the launching support combined with web crippling. This conclusion holds both for deep I-girders and box girders.

The design has to check the safety against both (a) local yielding and (b) buckling, as well as against combined influences.

Web crippling is of course included in various standard specifications and several authors have improved the solution, especially the calculations for combined influence, see e.g. [1]. The limit load is usually considered to be the one for which either the yield stress or the idealized plate buckling load is reached. As in modern steel construction more slender webs are used, the post-buckling range of a plate has to be considered. C A Granholm [2] performed full-scale tests in order to find the web crippling load and suggested that this load (with due safety factor) should be used as limit load. Further improvements [3] have led to the observation that even for girders where buckling is governing the influence of the yield point of the steel σ_y is of great importance (and not only the modulus of elasticity E), which means that the crippling load might be increased by the use of high strength steel.

The various kinds of influences - stresses and buckling - that have to be considered are shown in fig. 1 and are listed below :

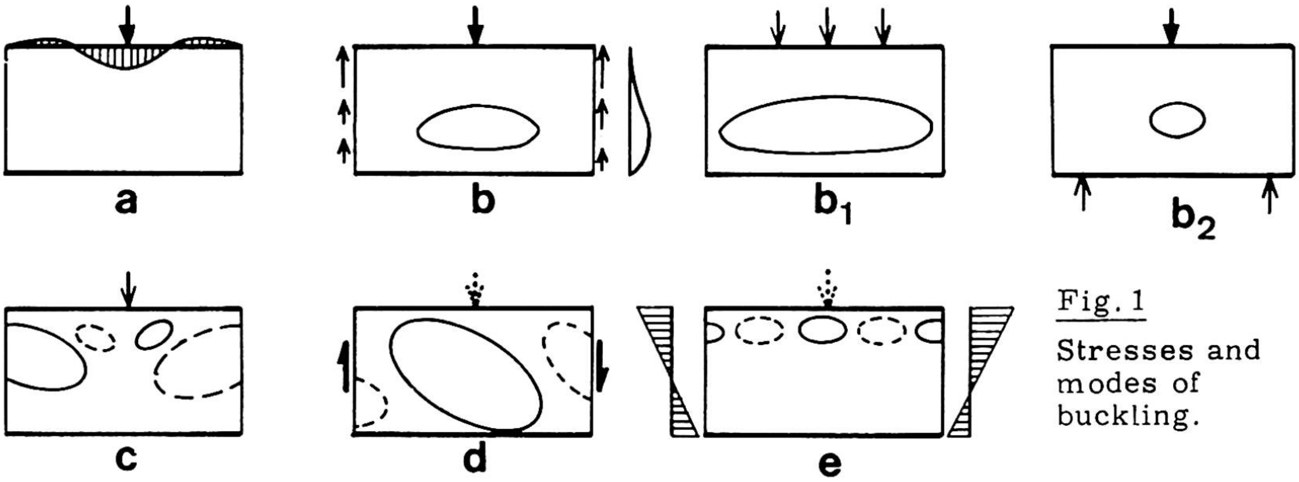


Fig. 1
Stresses and modes of buckling.

a. Local vertical stresses σ_z
b. Plate buckling because of vertical stresses
   b1. Plate buckling of a long part of the web (nearly equal to column buckling) because of distributed load or very stiff flanges
   b2. Plate buckling nearly equal to column buckling because the load is concentrated to a small part of the web due to opposite forces.
c. Shear buckling locally at the load
d. Shear buckling of the girder web
e. Buckling of the girder web due to bending stresses

It is obvious that local effects, such as a) c) and probably e) are interacting mainly with respect to local web crippling. An interaction of the effects b) and d) with respect to buckling occurs in a larger part of the web. The decrease of the bearing capacity due to effect b2) is sometimes greater than the decrease due to the effect e). This explains why it is sometimes found in tests that a large span girder can bear a greater concentrated load than a very short span girder.

Numerous tests have confirmed the obvious result that even if the bearing capacity for a web under a concentrated load mainly depends on the dimensions of the web, also the stiffness of the flange has a definite influence. Preliminary results [4] indicated an increase proportional to the variable $(1+0.4\ t/d)$ when $t/d>2$ approximately, where t/d = flange thickness/web thickness. When $t/d < \sim 2$ the tests indicated even a faster increase. Numerous test [5] in mainly the range $1.5 < t/d < 5$ confirmed the preliminary results and the following formulas are proposed

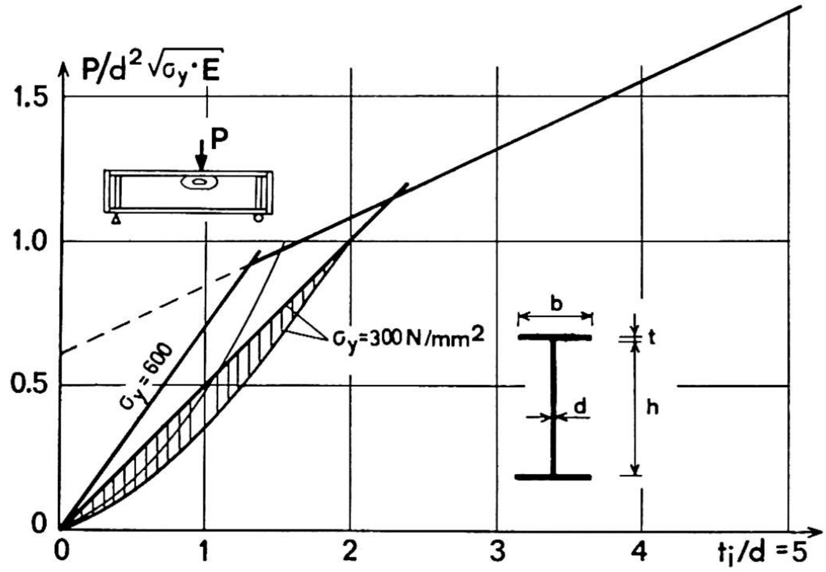


Fig. 2

Yield or crippling load according to Eq. (1).

$$\begin{cases} P^{yield}_{deform} \approx 13 \cdot \eta \cdot t \cdot d \cdot \sigma_y \quad ; \quad t/d <\sim 2 \quad ; & \ldots (1a) \\ P^{crippl.}_{failure} = 0.6 \cdot d^2 \sqrt{\sigma_y \cdot E}\,(1 + 0.4 \cdot t/d) & \ldots (1b) \end{cases}$$

t/d	= 0.5	1.0	1.5	2
η	= 0.55	0.65	0.85	1

Here P^{yield}_{deform} indicate a load above which the deformation begins to increase rather rapidly. If the reduction factor η is omitted (that is changed to 1 or in a certain range a little more) the transformed Eq. (1a) gives a theoretical yield failure load. It is seen that not only the yield load but also the crippling load (combined yielding and buckling) depend on the yield point σ_y, but to a different extent.

A theoretical deduction of Eq. (1a) is given in [4], and the corresponding failure load (inserting $\eta \approx 1$ in Eq. (1a)) is deduced in [5]. Both these deductions are based on an approximate model where the flange is regarded as a beam on an elastic spring bed (the bed being the web). More accurate results may be obtained by existing programmes for computations with the finite element method. A closed formula, however, is preferrable for the designer.

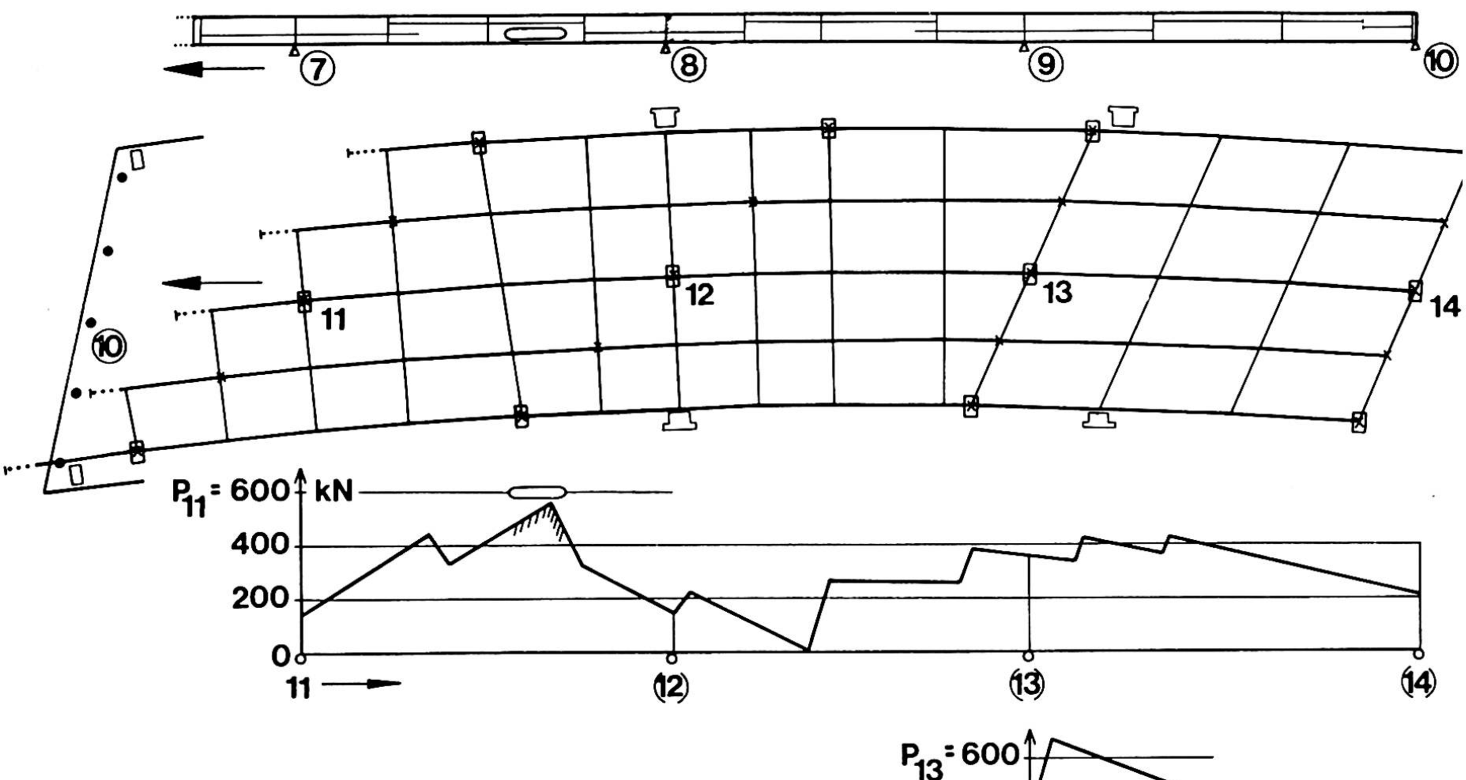
The simple formulae (1a) and (1b) hold for a flange width-to-thickness ratio $b/t \approx 25$. If the deviation from the value is great or if the flange is not rectangular, t in Eq. (1a), (1b) should be replaced by t_i from Eq. (2). The web "slenderness ratio" h/d is assumed to be of the order 200 but even great deviations from this value influence the coefficient only very little. If the load is not absolutely concentrated but distributed over the length c the bearing capacity is increased by multiplying with a coefficient f (c) the value of which is in principle given by Eq. (3), up to a maximum of $f(c) \approx 1.3$.

$$t_i = \sqrt[4]{\frac{12}{25}\cdot I_{fl}} \approx t\sqrt[4]{b/25\cdot t} \quad \ldots (2); \qquad f(c) = \frac{\gamma}{1-e^{-\gamma}\cdot\cos\gamma}, \text{ where } \gamma = c/2L \text{ and } L \approx 6.7\cdot t \quad \ldots(3)$$

Eq.(1a) corresponds mainly to influence of the kind shown in fig. 1a, while Eq.(1b) includes also influences as shown in fig. 1c and 1b. To this ought to be added influences as shown by fig. 1e, but the tests indicate that this does not give any reduction for bending moments M smaller than 0.6 M_f.

The failure moment M_f is to be calculated with regard to the reduction caused by web buckling. This reduction may be performed e.g. following a simplified method given in [6]. In this method the compressed part of the web is taken into account only by the portion $s = 1.95\cdot d\cdot\sqrt{E/\sigma}$ and from this portion 2/3 are placed near the neutral axis and 1/3 is placed near the compressed flange.

As an example the bridge in fig. 3 will be discussed, which was to some extent damaged during launching. The bridge has a length of about 200 m in 6 spans. A part of the bridge of somewhat more than 3 spans (ca 100 m) was erected by launching in order not to disturb the traffic on a number of under-



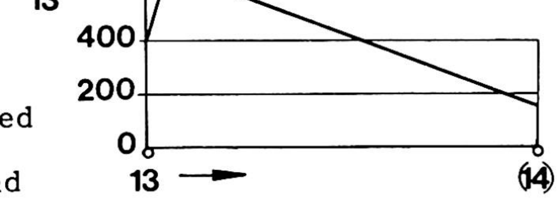


Fig. 3

Steel girder bridge erected by launching.
On top : The central main girder of the launched part (supports 7 to 10) and plan.
Diagrams : Variation of support forces P_{11} and P_{13} during launching.

lying railways. The supports had to be placed rather irregularly because of the railway tracks and as the bridge was slightly curved horizontally, it was very laborious to perform a complete analysis of all support forces during the various launching stages. The bridge has 5 main girders and in the final position of the permanent structure all girders are supported at every support line (the launching rolls were 2-roll bogies). In the provisional bridge bed from where the launching started, there were only 3 launching rolls in every support line (using single rolls). The reason for the different roll arrangements was of course the great risks if something unforeseen happened when launching above the railway lines.

Before the launching the supporting forces were only roughly checked mainly using the method of [1]. During launching a buckle was observed in the web in a region of the central main girder web, which is marked with an oval in the girder in fig. 3. Because of the buckle a computation of the supporting force was made afterwards for every roll (or bogie) for every stage of the launching. Here only two diagrams are shown of which one gives a reason for the buckle. (The other one gives a greater max. load but there is no risk for the web because of the thick flange).

Fig. 3 shows the girder of the bridge laying on the starting bed before the launching. The provisional supporting rolls are marked with squares and the positions of the rolls over the permanent supports are marked with filled circles (only these of the first permanent support line are seen quite to the left). The diagrams of the supporting loads give the load at the start in the utmost left and then the variation as the launching goes on.

The dangerous point, where also the buckle was afterwards observed, is marked with an oval in the force diagram for P_{11} in Fig. 3. In this section t/d was 16 mm/12 mm and Eq. (1a) is governing. The steel quality of the bridge girder had σ_y = 260 N/mm^2. This gives P^{yield}_{deform} = $13 \cdot 0.8 \cdot 16 \cdot 12 \cdot 260$ N = 520kN = = 52 Mp.

The supporting force was 56 Mp in this point as seen in the diagram in fig. 3 and thus larger than the load giving large deformations. The force was, however, less than the failure load (where $\eta \approx 1$) which is $P_{failure} \approx$ 65 Mp. At such high loads, the deformations ought to be very large, and the relatively stiff transverse girders would distribute the supporting force to the neighbouring girders. Even with such a load distribution the margin to the failure load was small.

After the launching a part of the flange and the web at the deformed region was cut out and replaced. Inspection showed that the web had small deformations indicating incipient web crippling at some other locations too, and there some small adjustments only were undertaken before the bridge was taken into traffic.

— When the concentrated load is too great there are several possibilities. As already mentioned one can use launching rolls designed with bogies, one can distribute the load by arranging a crane rail below the flange, or one can use a thicker web or use a higher quality for the steel of the web.

For t/d < 2, where Eq. (1a) is valid, the advantages of the two latter solutions are immediate apparent. For t/d > 2; and Eq. (1b), the best illustration is given by an example. For the bridge just described there were parts having t/d = 35/12 and in order to compare the web thickness neeeded for two steels with σ_y = 260 and 400 N/mm^2 respectively, you use Eq. (1b) :

$$0.6\cdot 12^2\sqrt{400\ E}\,(1+0.4\cdot \frac{35}{12}) = 0.6\cdot d_1^2\sqrt{260\ E}\,(1+0.4\cdot \frac{35}{d_1})$$

From this we find d_1 = 13.9 mm, that is 16 % thicker than the 12 mm web. If the difference in steel prize is less than 16 % it is thus cheaper to use the steel of higher quality.

*

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SUMMARY

A formula is given for the support forces due to launching that a steel girder can resist. The formula is applied to a bridge with damaged (crippled) web. The use of high strength steel is discussed.

RESUME

Une formule est donnée montrant la force d'appui qu'une poutre peut supporter pendant une construction par lancement. La formule est appliquée à un pont où l'âme est voilée. L'influence de différentes qualités d'acier est discutée.

ZUSAMMENFASSUNG

Es wird eine Formel angegeben für den Auflagerdruck, den ein Stahlträger während des Einschiebens aufzunehmen vermag. Diese Formel wird für eine bestimmte Brücke angewendet, deren Steg beschädigt wurde (Beulen). Die Verwendung von hochwertigem Stahl wird diskutiert.