Discussion of the report by Professor P. Dubas: "Essais sur le comportement post-critique de poutres en caisson raidies"

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DISCUSSION PRÉPARÉE / VORBEREITETE DISKUSSION / PREPARED DISCUSSION

Discussion of the Report by Professor P. Dubas:
"Essais sur le comportement post-critique de poutres en caisson raidies"
The Conventional Design of Box Girders is unsafe and must be the — at least partial — Cause of the Recent Collapse of Three Large Box Girders Bridges

Discussion du rapport du Prof. P. Dubas:
"Essais sur le comportement post-critique de poutres en caisson raidies"

Diskussion über den Bericht von Prof. P. Dubas:
"Essais sur le comportement post-critique de poutres en caisson raidies"

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1. INTRODUCTION.

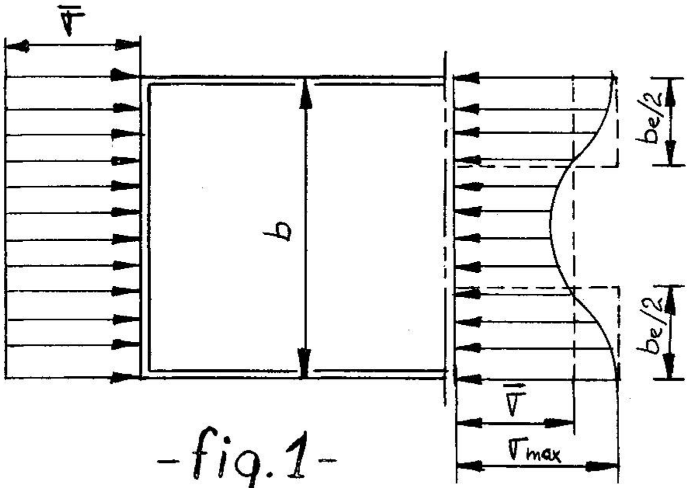
The striking result obtained by professor DUBAS in the test of a box girder described in his report, namely a mean collapse stress less than the critical stress derived from linear buckling theory (see sec.8), has crystallized some grave doubts we had since several years about the safety of the conventional design of box girders. We hold the opinion that, if the safety factor s of about 1.35 against the critical stress of linear buckling theory, adopted in several countries, was justified in the case of the web of a plate girder, because of the stabilizing effect of a postcritical diagonal tension field, the use of the same coefficient for designing the compressed flange of a box girder was totally unjustified, because the stabilizing effect of membrane stresses is, in this case, much less than in the first one.

It may be appropriate to recall here that the senior author has repeatedly insisted [1, 2] on the fact that theoretically strictly rigid stiffeners, (that means stiffeners of relative rigidity γ^* given by linear buckling theory)were never rigid in practice and gave girders with a low safety, barely able to reach 0.95 to 1 times the yield point in the flanges at collapse. To ensure stiffeners remaining effectively straight up to collapse, it was necessary to adopt values $\gamma = m\gamma^*$ with m varying between 3 and 8. The effect of this increase in the rigidity of the stiffeners was to increase the strength by about 25 per cent. This experimental fact has been confirmed recently by OWEN, ROCKEY and SKALOUD [3].
The senior author was therefore convinced that box girders with flanges stiffened by γ^* stiffeners had a particularly low effective safety against collapse.

Now, the spectacular accidents which have struck, during last year, three large steel box girder bridges, namely the bridge over the Danube in Vienna on 6 November 1969, the bridge of Milford Haven in Great Britain on 2 June 1970 and the bridge over the lower Yarra in Melbourne (Australy) on 15 October 1970, have reinforced these doubts about the validity of the linear buckling theory.

It seems demonstrated that, in the case of the Danube bridge at least, the collapse occurred for mean compression stresses barely equal to the critical stress of the linear theory. This would mean that, in this case, at least, no reserve of postcritical strength existed.
One of the purposes of present report is to show theoretically that this is actually the case (see section 6).

The compressed flange of a box girder subjected to bending is not, like the web of a plate girder, strengthened by a rigid frame constituted of the flanges of the girder and the adjacent transverse stiffeners. On the contrary, the most plausible assumption regarding the boundary conditions of this flange is simple support with complete freedom of the plate edges to move and deform in the plane of the plate. The instability phenomenon of such plates is nearer to that of the compressed column than to that of a plate girder web subjected to shear.
In particular, unavoidable imperfections such as buckles due to the welding sequence should exert a strong deteriorating influence. (see sec. 6).
In other words, the transverse distribution of compressive stresses is far from remaining uniform up to collapse. Effectively, as is clearly shown in fig. 8 of DUBAS report, the stress diagram shows a central pocket of increasing magnitude. The stresses in the middle are lagging behind the edge stresses (Fig. 1). When these latter reach the yield point, the capacity of the box girder is practically exhausted (see sec. 4) and the box girder collapses.



-fig.1-

Summarizing, the aim of present report is to prove that the reduction of the mean stress due to buckling, enhanced by imperfections, may upset the gain due to non linear membrane stresses, so that finally the mean collapse stress $\bar{\sigma}$ of the imperfect flange may become even less than the critical stress of the perfect flange according to linear buckling theory (see sec. 6). There is therefore an urgent need to push forward the theoretical and experimental investigation and, pending these researches, to increase notably the safety factors of box girders against plate buckling.

In waiting for these investigations, we have tried to draw the best information from the few papers at our disposal (sec. 2 to 4).

2. SOME CONSIDERATIONS ABOUT THE COLLAPSE OF THREE BOX GIRDER BRIDGES.

As told above, we imagine that the fundamental inadequacy of linear buckling theory as applied to the design of large box girders may have played a role in the recent collapse of three large box girder bridges.
As we have been able to collect detailed information only in the case of the bridge at Vienna, we shall restrict ourselves to the study of this case.

The circumstances of the collapse of the Danube bridge near Vienna seem rather clear [4, 5]. According to professor SATTLER's paper [5], the three experts took following position regarding the causes of the accident:

1. The calculation of erection stresses was made for a uniformly distributed

loading. The actual distribution of the dead weight differs from this assumption and gave at one of the damaged places more unfavourable conditions, so that in reality at this place larger stresses have existed and therefore smaller-buckling and collapse safeties.

2. The temperature effect in the steel structure on the day of the bridge closing had a value that was not to be expected from the responsible personnel from the temperature observations of the preceeding days.
This fact diminished the safety factor against collapse.

3. Besides, there existed constructive as well as unavoidable imperfections (which diminished the safety factor). Taking these imperfections into account is not necessary, according to the specifications, but is covered by the required safety factors. In present case, where the safety against collapse was already diminished by circumstances 1 and 2 above, the imperfections have played a non negligible role as partial cause of the accident

4. The collapse of the entire lower flange of the box girder at a place precipitated the collapse of the whole cross section. The redistribution of internal forces which ensued necessarily explain all other damages as consequences of the first one.

Professor SATTLER has been very kind to send us the detailed report he established as expert for the bridge collapse. According to this report:
The safety factor adopted during erection was :
1.25. against yielding
1.25 against buckling calculated by linear theory.

The steel used was St 44, with a yield point of σ_y = 2900 Kg/cm^2.

At the section where the first buckling damage must have occurred, the lower compressed flange had a breadth of 7600 mm, a thickness of 10 mm (b/t = 760) and was stiffened by 12 flat stiffeners 160 x 12 mm.

According to professor SATTLER's report, the relative rigidity of these stiffeners was chosen strictly to obtain a buckling coefficient k of the whole panel equal to that of a subpanel, namely $k = 4 \times (12 + 1)^2 = 676$.

In other words, the relative rigidity of these stiffeners was equal to γ^*

The ideal buckling stress found in the calculations was σ_{cr}^{id} = 2,235 Kg/cm^2 and the reduced buckling stress taking account of plasticity, was, according to the Austrian Specifications, σ_{cr} = 2,213 Kg/cm^2.

According to professor SATTLER's calculations, due to the circumstances indicated above, the mean stress in the compressed flange must have reached at the time of collapse, the value

$$\bar{\sigma}_{max} = 2{,}224 \text{ Kg/cm}^2.$$

SATTLER considers that the condition $\sigma_{max} = \sigma_{cr}$ is the explanation of the collapse.
We completely agree with this explanation, especially because we shall show in section 6 that collapse can occur even for values of the mean stress $\bar{\sigma}$ less than σ_{cr}.

3. FOUNDATIONS OF THE NON LINEAR THEORY OF BUCKLING OF COMPRESSED PLATES.

The non linear theory of buckling of plates has been developed by von Karman and is represented by following coupled fourth order equations

$$\frac{D}{t}\ \nabla^2\nabla^2 w = \frac{\partial^2\phi}{\partial x^2}\frac{\partial^2 w}{\partial y^2} + \frac{\partial^2\phi}{\partial y^2}\frac{\partial^2 w}{\partial x^2} - 2\,\frac{\partial^2\phi}{\partial x\partial y}\frac{\partial^2 w}{\partial x\partial y} \qquad (1)$$

$$\nabla^2\nabla^2\phi = E\ \left[\left(\frac{\partial^2 w}{\partial x\partial y}\right)^2 - \frac{\partial^2 w}{\partial x^2}\frac{\partial^2 w}{\partial y^2}\right] \qquad (2)$$

where: w is the transverse displacement of the plate,

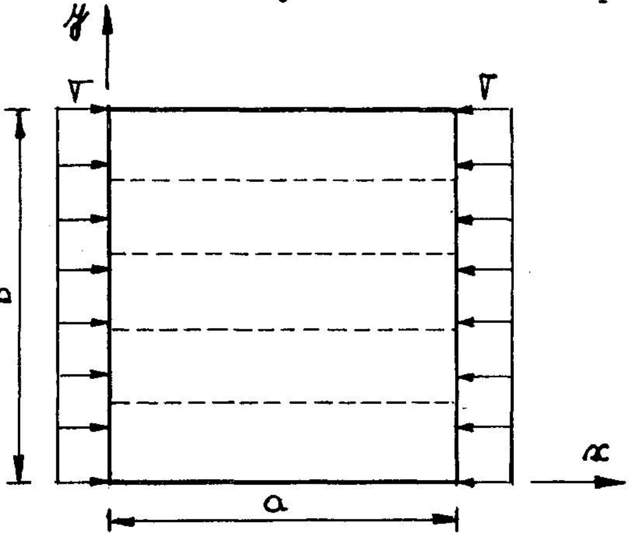
$D = \dfrac{Et^3}{12(1-\nu^2)}$ its flexural rigidity ,

t the thickness

ν Poisson's ratio,

ϕ Airy's stress function.

The stiffened plates constituting the compressed flanges of box girders are supported at their edges in such a way that these edges can move freely in the plate's plane. If the box girder is subjected to pure bending, as in professor DUBAS experiments, there are no shear stresses along the lateral edges before buckling, and it seems reasonable to admit that these shear stresses remain very small even in the postbuckling range. Therefore, the boundary conditions of the stiffened compressed plate (a x b) of fig. 2 may be taken as :



-fig.2-

for $x = 0$ and $x = a$:

$$w = \frac{\partial^2 w}{\partial x^2} + \nu\frac{\partial^2 w}{\partial y^2} = 0$$

$$\frac{\partial^2\phi}{\partial y^2} = \sigma\ , \qquad \frac{\partial^2\phi}{\partial x\partial y} = 0 \qquad (3)$$

for $y = 0$ and $y = b$:

$$w = \frac{\partial^2 w}{\partial y^2} + \nu\frac{\partial^2 w}{\partial x^2} = 0$$

$$\frac{\partial^2\phi}{\partial x^2} = 0\ , \qquad \frac{\partial^2\phi}{\partial x\partial y} = 0 \qquad (4)$$

Starting from equations (1) and (2) with boundary condition (3), (4), SKALOUD and NOVOTNY, in two papers ([7], [8]), have brought a very important contribution to the solution of the problem, for the case of one median longitudinal stiffener or of two stiffeners placed at the thirds of the width. Later on, they have sketched the solution of the same problem when account is taken of the effect of initial deformation or stresses ([9],[10]). The solution is based on Rayleigh-Ritz energy method. As the authors have not taken account of the potential energy corresponding to the compression of the stiffeners, the results obtained do not depend on the relative area of the stiffeners, and are valid only for $\delta = \frac{A}{bt} = 0$.

On the other hand, numerical results are given only for the square plate

($\alpha \equiv a/b = 1$); however, their paper contains the cubic equations which would enable to develop the calculations for other values of α

4. COLLAPSE CRITERION ADOPTED.

SKALOUD and NOVOTNY admit that the strength of the plate is exhausted when the maximum membrane stress σ_{xm} which occurs along the unloaded edges reaches the yield point σ_y of the steel used. In the case where the stiffeners remain rigid up to collapse, the membrane stresses σ^r_{xm} in the plate at the location of the stiffeners reach also σ_y.

The validity of above collapse criterion has been extensively discussed by SKALOUD in other papers (see e.g. [11]). It neglects two circumstances which have opposite effects: the bending stresses in the plate and the plastic redistribution after the yield point has been reached. We admit that these effects cancel each other and therefore the validity of SKALOUD's criterion.

5. CHARTS FOR THE SQUARE PLATE.

From the diagrams obtained by SKALOUD and NOVOTNY for a square plate with one or two equidistant stiffeners, we have constructed the charts of figures 3 and 4. Figures 3a and b apply to the plate with one stiffener, figures 4a and b to the plate with two stiffeners. We have only considered values of the stiffener's relative rigidity $\left[\gamma = \frac{EI}{bD}\right]$ for which $\gamma > \gamma^*$.

First problem.

Given a square steel plate whose dimensions a = b, t, are known, it is asked to determine the rigidity γ_p^* required from the stiffener(s), in order that this (these) remain rigid up to collapse, as well as the value of the collapse mean stress $\bar{\sigma}$.

The solution is immediate by figures 3a and 4a. $\bar{\sigma}$ and γ_p^* depend only on the thinness b/t of the plate and their values are obtained at the intersection of the corresponding curves with the horizontal of the ordinate b/t.

A vertical drawn from the γ_p^* value to the curve of factors $m = \frac{\gamma_p^*}{\gamma^*}$, gives the multiplier of value of γ^*, which itself is known in function of α and of the relative area δ (see note at the bottom of next page)

By the relation

$$m\,\gamma^* = 10{,}92\,\frac{I_r}{bt^3},$$

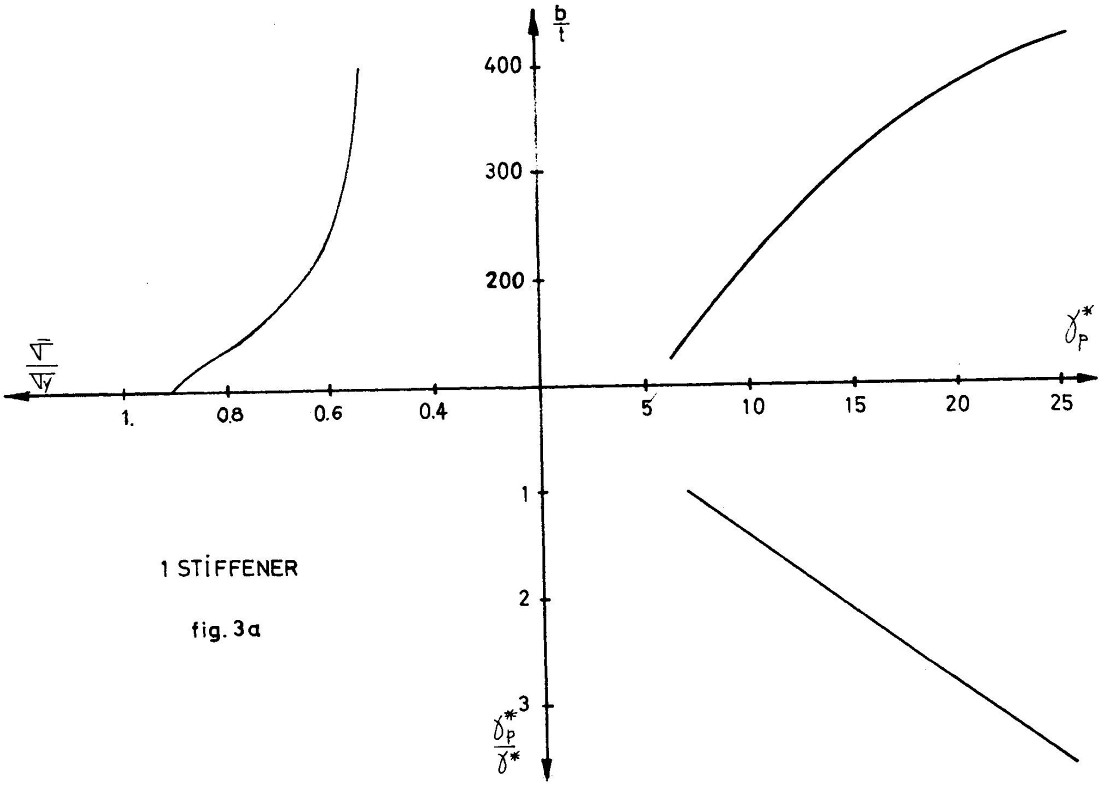
the moment of inertia of the stiffener(s) required becomes

$$I_r = \frac{m\gamma^* bt^3}{10{,}92}$$

Example:

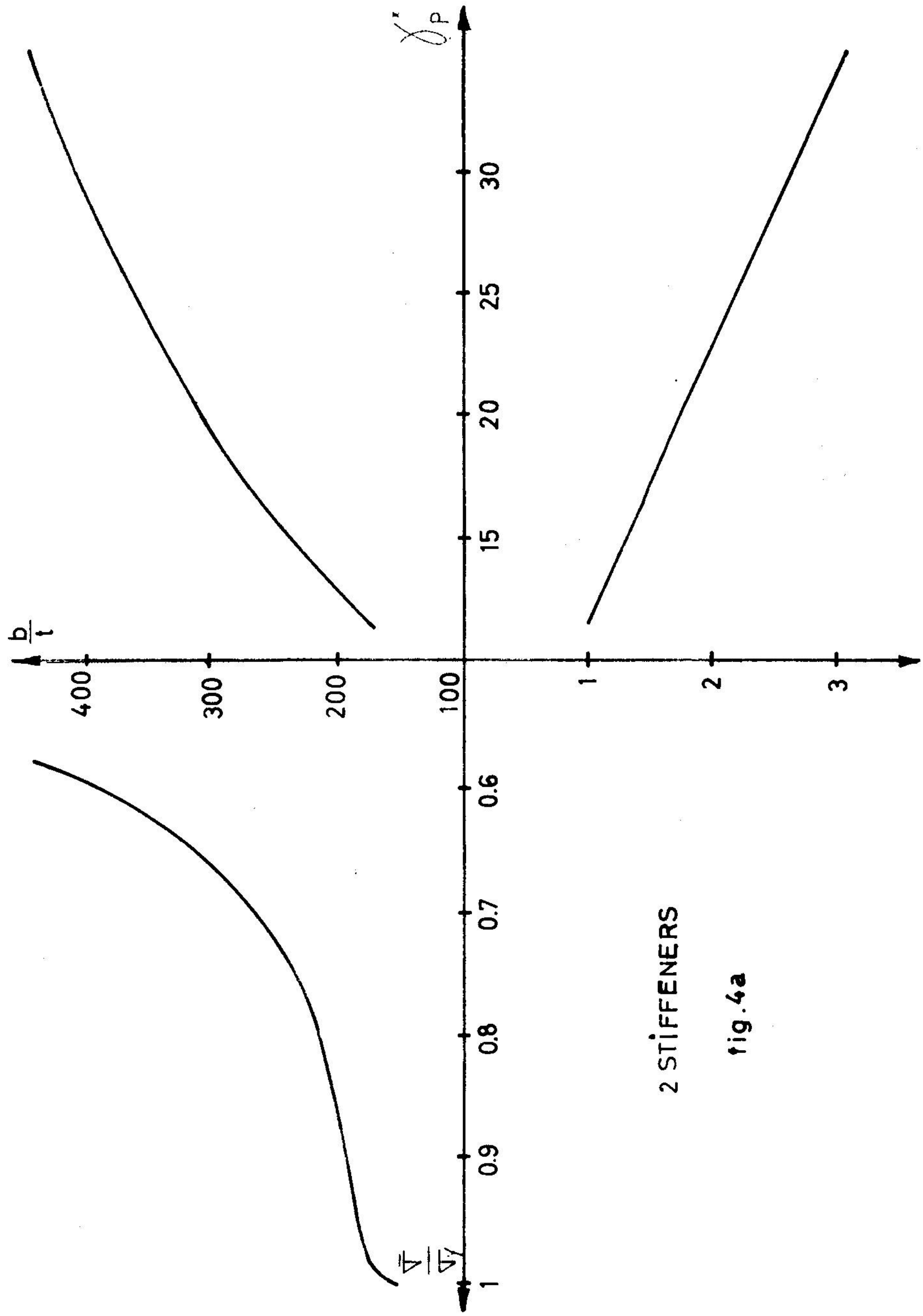
a = b = 200 cm , t = 0,8 cm. One stiffener

Figure 4a gives for b/t = 200/0,8 = 250



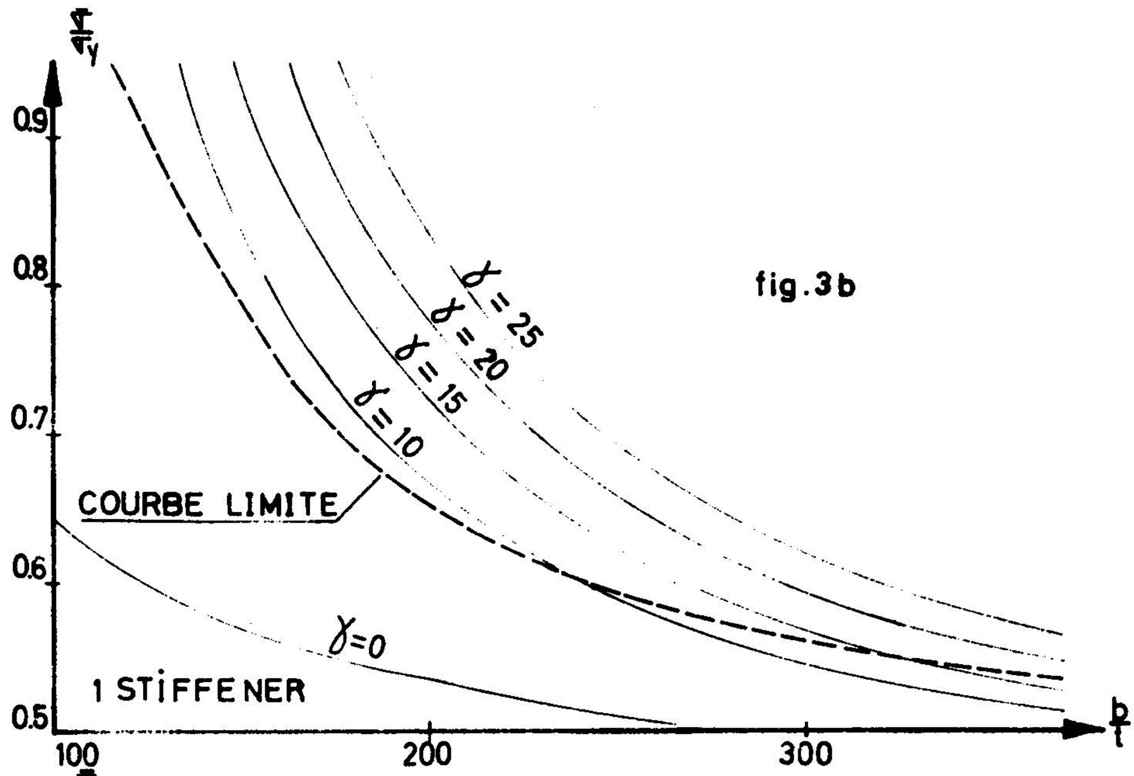
1 STİFFENER

fig. 3a



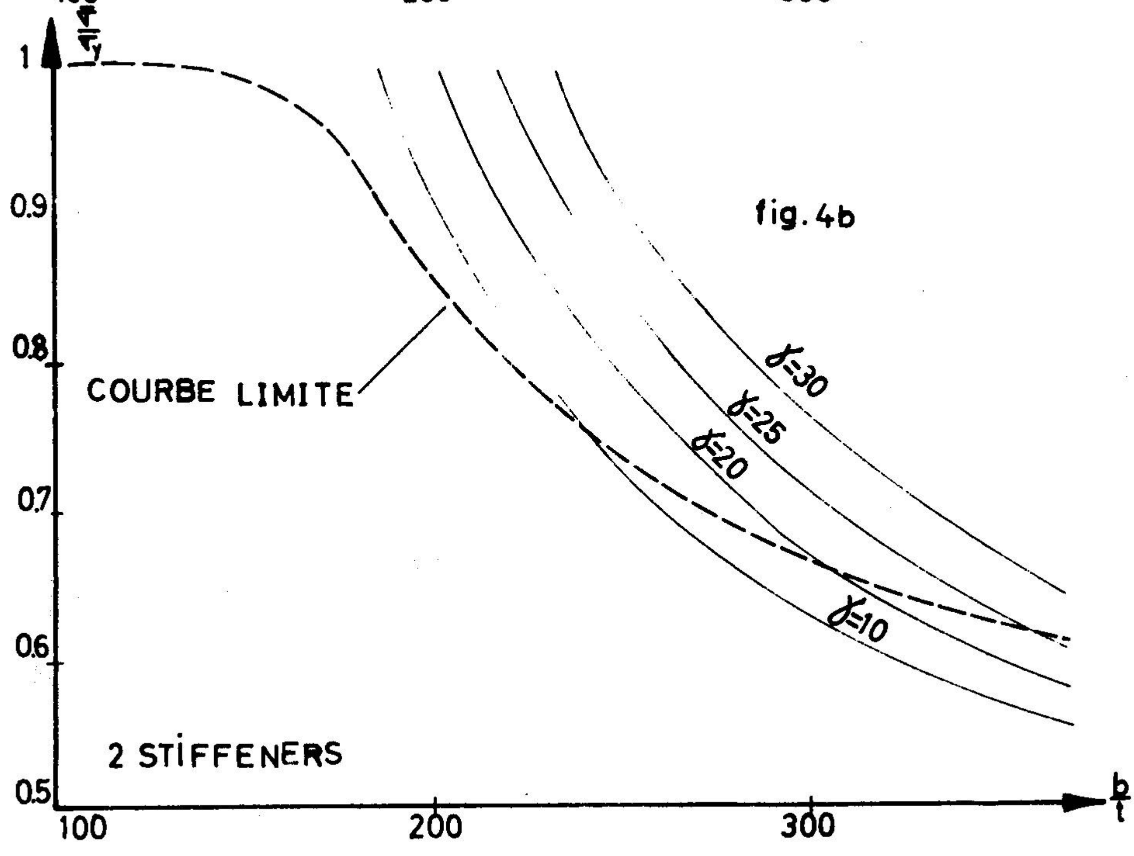
2 STIFFENERS

fig.4a



σ/σy
0.9
0.8
0.7
0.6
0.5
γ = 25
γ = 20
γ = 15
γ = 10
COURBE LIMITE
γ=0
1 STIFFENER
100
200
300
b/t

fig.3b



σ/σy
1
0.9
0.8
0.7
0.6
0.5
COURBE LIMITE
γ=30
γ=25
γ=20
γ=10
2 STIFFENERS
100
200
300
b/t

fig.4b

$\frac{\bar{\sigma}}{\sigma_y}$ = 0.59, whence $\bar{\sigma}$ = 1416 Kg/cm^2.

γ^* = 11.6. whence m = 1.65.

The moment of inertia of the stiffener must at least be I_r = 109 cm^4.

Second problem.

Given a square plate whose dimensions a = b, t are known, stiffened by one (two) stiffener(s) of given relative rigidity, it is asked to determine his ultimate strength.

The solution is immediate by figures 3b and 4b, established for a yield point σ_y = 2400 Kg/cm^2. These figures give the value of $\bar{\sigma}/\sigma_y$ as a function of the thinness b/t of the plate.

The various curves correspond to definite values of γ.

Example: a = b = 400 cm ; t = 1 cm ; I_r = 549.5 cm^4.

First, the relative rigidity

$$\gamma = \frac{10.92 \times 549.5}{400 \times 1} = 15$$

is calculated. Then, from figures 3b or 4 b, one reads :

a) for one stiffener : $\frac{\bar{\sigma}}{\sigma_y} = 0.515$

b) for two stiffeners : $\frac{\bar{\sigma}}{\sigma_y} = 0.530$

(*)

δ being the ratio $\frac{\text{area of the stiffener's cross section}}{\text{area of the cross section of the plate}} = \frac{A_r}{bD}$,

one has:

one stiffener: $\alpha \leq \sqrt{8(1+2\delta)-1}$: $\gamma^* = \frac{\alpha^2}{2}\left[16(1+2\delta)-2\right] - \frac{\alpha^4}{2} + \frac{1+2\delta}{2}$

$\alpha \geq \sqrt{8(1+2\delta)-1}$: $\gamma^* = \frac{1}{2}\left[8(1+2\delta)-1\right]^2 + \frac{1+2\delta}{2}$

two stiffeners :

$\alpha \leq \sqrt{18(1+3\delta)-1}$: $\gamma^* = \frac{\alpha^2}{3}\left[36(1+3\delta)-2\right] - \frac{\alpha^4}{3} + \frac{1+3\delta}{3}$

$\alpha \geq \sqrt{18(1+3\delta)-1}$: $\gamma^* = \frac{1}{3}\left[18(1+3\delta)-1\right]^2 + \frac{1+3\delta}{3}$.

6. EFFECT OF AN INITIAL DEFORMATION.

SKALOUD and NOVOTNY have studied in [9] the effect of an initial deformation of a stiffened plate on its ultimate strength for the case of one median stiffener. They have established for this case and for various values of the relative rigidity γ of the stiffener charts giving $\bar{\sigma}/\sigma_y$ (for σ_y = 2400 Kg/cm^2) as function of the relative thinness b/t of the plate. The curves are labelled in terms of f_o/t, where f_o is the initial deflection in the middle of the panel and t the plate thickness. The shape of the initial deformation is

$$w_o = f_o \sin \frac{\pi x}{a} \sin \frac{\pi y}{b} .$$

and the boundary conditions are the same as previously.

The collapse criterion is the same as that discussed in section 4.

The loss in ultimate strength is especially marked for small values of the thinness b/t and becomes negligible for a certain value of b/t which is the smallest for stiffeners with the largest values of the relative rigidity.

Figures 5 and 6 reproduce the charts of SKALOUD and NOVOTNY for the values γ = 10 and 20 of the relative rigidity, and for f_o/t = 0,1,2. The curve corresponding to f_o/t = 2 has been obtained from the curves f_o/t=0 and f_o/t = 1 by assuming the $\bar{\sigma}/\sigma_y$ varies linearly with f_o/t for b/t fixed, which is approximately correct.

In test N° 1 of professor DUBAS, (see his fig. 11, p. 14), one has an initial deflection f_o = 4.8 mm for a thickness t = 3.3 mm. The corresponding ratio f_o/t = 1.46. As it concerns a laboratory test where the specimens are fabricated with especial care, it is logical to admit that, in an actual bridge, the ratio f_o/t may reach 2.

Above theoretical results are limited to the case of a single stiffener. Pending new researches, we have supposed that they can be extended to a plate with two longitudinal stiffeners under following assumptions:

a) The two plates have same critical buckling stress according to the linear theory ;

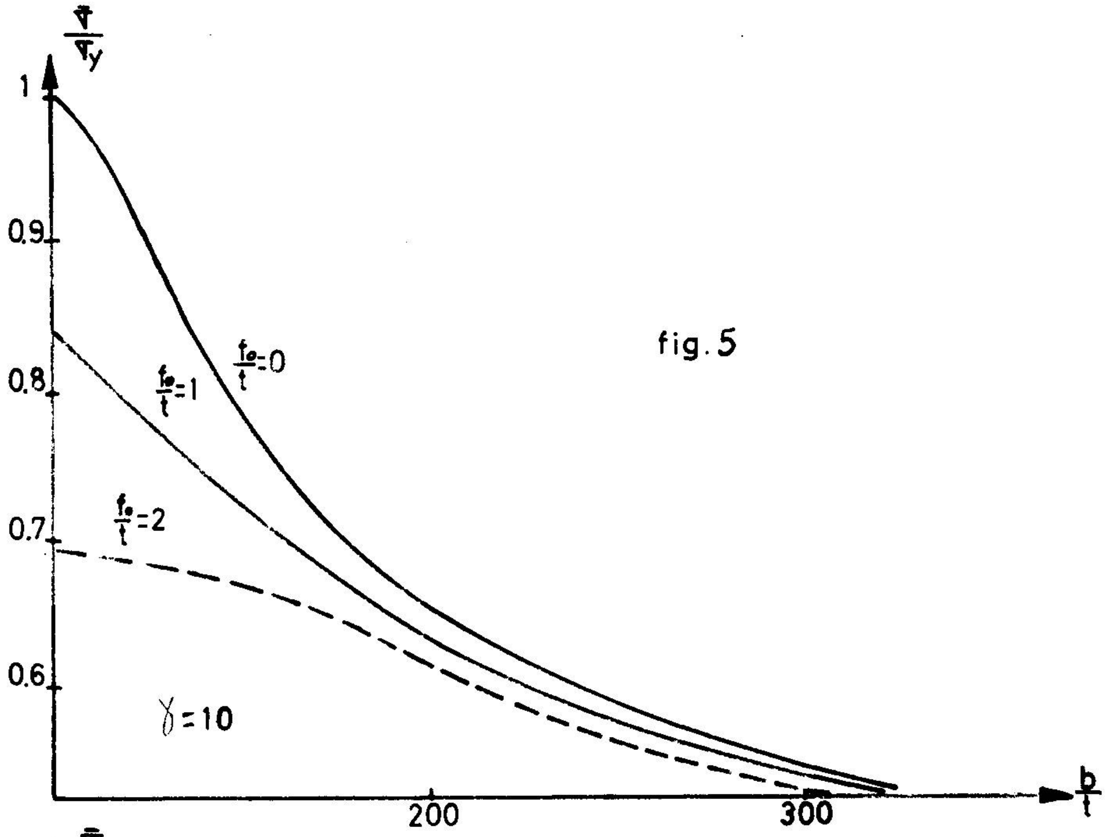
b) The ratios of the actual relative rigidity of the stiffeners to the relative optimum rigidity γ^* of the linear theory are identical.

Under these conditions, we assume that the relative reductions of collapse strength are identical for the two plates :

$$\left[\frac{(\bar{\sigma})^{2\ \text{stiff.}}_{f_o/t=n}}{(\bar{\sigma})^{2\ \text{stiff.}}_{f_o/t=0}}\right] \left(\frac{\gamma_{2\ \text{stiff.}}}{\gamma^*_{2\ \text{stiff.}}}\right) = m \qquad = \qquad \left[\frac{(\bar{\sigma})^{1\ \text{stiff.}}_{f_o/t=n}}{(\bar{\sigma})^{1\ \text{stiff.}}_{f_o/t=0}}\right] \left(\frac{\gamma_{1\ \text{stiff.}}}{\gamma^*_{1\ \text{stiff.}}}\right) = m \qquad (5)$$

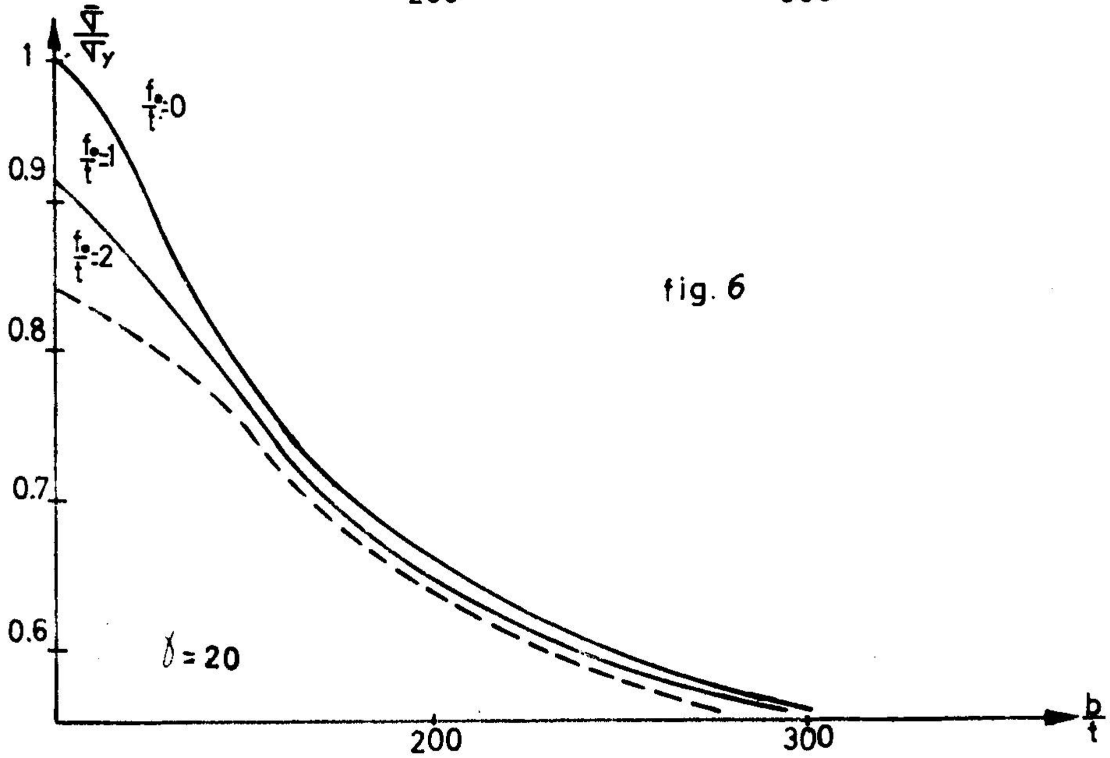
$$\sigma_{cr}^{2\ \text{stiff}} = K \qquad\qquad \sigma_{cr}^{1\ \text{stiff.}} = K$$

This generalization is illustrated by example 2 hereafter.



$\frac{\bar{\sigma}}{\sigma_y}$
1
0.9
0.8
0.7
0.6
$\frac{f_0}{t}=0$
$\frac{f_0}{t}=1$
$\frac{f_0}{t}=2$
$\gamma = 10$
200
300
$\frac{b}{t}$

fig. 5



$\frac{\bar{\sigma}}{\sigma_y}$
1
0.9
0.8
0.7
0.6
$\frac{f_0}{t}=0$
$\frac{f_0}{t}=1$
$\frac{f_0}{t}=2$
$\gamma = 20$
200
300
$\frac{b}{t}$

fig. 6

We shall now try to simulate by numerical examples the conditions existing at collapse in the case of the bridge over the Danube (cf. Section 2) namely an actual stress amounting to the critical stress given by linear buckling theory, because of an error on the actual dead weight distribution and of some additional temperature stresses.
Due to lack of theoretical data, we are obliged however, to consider a square panel in mild steel (σ_y = 2400 Kg/cm^2) stiffened by one or two stiffeners only.

Example N° 1 : box girder bridge with the compressed flange defined by following data :

$$\alpha = \frac{a}{b} = 1 \ , \ \gamma = 10 \ ; \ \frac{b}{t} = 126 \quad ; \quad \text{Steel AE24 } (\sigma_y = 2400 \text{ Kg/cm}^2)$$

one median longitudinal stiffener.

As the adopted γ is larger than $\gamma^* = 7$, the linear buckling theory gives for the buckling coefficient $k = 4 \times 2^2 = 16$ and the critical stress is

$$\sigma_{cr} = 1920 \text{ Kg/cm}^2.$$

If we follow the Austrian Specifications for erection conditions, we should adopt for design stress the lowest of the two ratios (cf. sec. 2) :

$$\frac{\sigma_{cr}}{1.25} = 1536 \text{ Kg/cm}^2$$

$$\frac{\sigma_y}{1.25} = \frac{2400}{1.25} = 1920 \text{ Kg/cm}^2,$$

that means 1536 Kg/cm^2.

However, due to above effects, the actual stress has amounted effectively to

$$\sigma_e = \sigma_{cr} = 1920 \text{ Kg/cm}^2.$$

According to the non linear theory of SKALOUD and NOVOTNY, we find, according to figures 5 and 6 :

For $\gamma = 10$ and $\frac{f_o}{t} = 0$: $\frac{\bar{\sigma}}{\sigma_y} = 0.895$, whence $\bar{\sigma} = 2148$ Kg/cm^2

$\frac{f_o}{t} = 1$: $\frac{\bar{\sigma}}{\sigma_y} = 0.785$, whence $\bar{\sigma} = 1884$ Kg/cm^2

$\frac{f_o}{t} = 2$: $\frac{\bar{\sigma}}{\sigma_y} = 0.682$, whence $\bar{\sigma} = 1637$ Kg/cm^2

For $\gamma = 20$ and $\frac{f_o}{t} = 0$: $\frac{\bar{\sigma}}{\sigma_y} = 0.895$ whence $\bar{\sigma} = 2150$ Kg/cm^2

$\frac{f_o}{t} = 1$: $\frac{\bar{\sigma}}{\sigma_y} = 0.846$ whence $\bar{\sigma} = 2030$ Kg/cm^2

$\frac{f_o}{t} = 2$: $\frac{\bar{\sigma}}{\sigma_y} = 0.805$ whence $\bar{\sigma} = 1932$ Kg/cm^2

We may now calculate the safety given by the effective stress σ_e = 1920 Kg/cm^2. The results are given in following table

Values of the effective safety factor for erection conditions $s = \frac{\bar{\sigma}}{\sigma_e}$

f_o/t \ γ/γ^*	1,43	2,86
0	1.118	1.118
1	0.981	1.057
2	0.853	1.006

We see that the effective stress may exceed the mean collapse stress according to SKALOUD - NOVOTNY as soon as the initial imperfection of the plate is of the order of the thickness. If it is recalled in addition that the stiffeners of the Danube bridge gave a ratio γ/γ^* approximately equal to one only (cf. sec.2), above table demonstrates that the considered box girder bridge will collapse during erection for values of the relative imperfection $f_o/t < 1$.

Example N° 2 : We apply now the generalization to a plate with two stiffeners proposed in this section and represented by formula (5).
We assume once more that the effective stress is equal to the critical stress, e.g. 1920 Kg/cm^2.

The plate homologous to that of example 1 has the following characteristics :

$$\gamma_{2\ stiff} = \gamma^*_{2\ stiff} \times \frac{\gamma_{1\ stiff}}{\gamma^*_{1\ stiff}} = 11.33 \times \frac{10}{7} = 16.2$$

$\gamma_{2\ stiff}$ being larger than $\gamma^*_{2\ stiff}$, the two stiffeners remain straight according to linear theory and $k = 4 \times 3^2 = 36$.

$$\frac{b}{t} = \sqrt{\frac{36 \times 1,900,000}{1920}} = 189$$

From figure 4b, one has

$$\left[\left(\frac{\bar{\sigma}}{\sigma_y}\right)^{1\ stiff}_{f_o/t=0}\right]_{\substack{\gamma=16,2 \\ \frac{b}{t}=189}} = 0.9 \text{ (limit curve)}$$

whence

$$(\bar{\sigma})^{2\ stiff}_{f_o/t} = 21.60 \text{ Kg/cm}^2$$

and therefore, according to formula (5)

for $\frac{f_o}{t} = 1$: $(\bar{\sigma})^{2\ stiff}_{f_o/t=1} = 21.60 \times \frac{1884}{2148} = 1893 \text{ Kg/cm}^2$

$$\frac{f_o}{t} = 2 \;:\quad (\bar{\sigma})^2_{f_o/t=2} \text{ stiff} = 2160 \times \frac{1637}{2148} = 1643 \text{ Kg/mm}^2.$$

The safeties during erection are therefore respectively :

f_o/t \ γ/γ^*	1,43
0	1.125
1	0.986
2	0.855

and the same conclusions as for example 1 ensue.

7. EFFECTIVE WIDTH FORMULAE.

It is well known that the irregular stress distribution across the width b of the plate (fig. 1) with mean value $\bar{\sigma}$ and maximum value σ_{max} may be replaced by a uniform distribution of the maximum stress σ_{max} on a fictitious width called effective width. For equal resultants, we need

$$\sigma_{max}\, b_e = \bar{\sigma} b$$

whence

$$\frac{b_e}{b} = \frac{\bar{\sigma}}{\sigma_{max}} = \phi \qquad (6)$$

If the stiffeners remain rigid up to collapse, we admit that the maximum stress σ_{max} is attained also in the plate at each junction with the stiffeners, so that the effective width formula may be applied to the subpanels.

Let us call :

σ_{cr} : the critical stress of buckling of a sub-panel ;

σ_{max} : the maximum membrane stress at the edges of a subpanel, taken equal to σ_y at collapse according to SKALOUD's criterion adopted in section 4;

β : the ratio of the width b of a subpanel by the half wave length of longitudinal buckling.

Among the formulae proposed for the effective width for the considered case (simply supported edges and free relative movement of these edges) we shall retain the following

$$\phi = 0.44 + 0.56 \frac{\sigma_{cr}}{\sigma_{max}} \qquad \text{(PAPCOVITCH)} \qquad (7)$$

$$\phi = \sqrt[3]{\frac{\sigma_{cr}}{\sigma_{max}}} \qquad \text{(MARGUERRE)} \qquad (8)$$

$$\phi = \sqrt{\frac{\sigma_{cr}}{\sigma_{max}}} \qquad \text{(von KARMAN)} \qquad (9)$$

$$\phi = \frac{1+\beta^4}{3+\beta^4} + \frac{2}{3+\beta^4}\left(\frac{\sigma_{cr}}{\sigma_{max}}\right) \qquad \text{(SECHLER)} \qquad (10)$$

$$\phi = \sqrt{\frac{\sigma_{cr}}{\sigma_{max}}}\left(1 - 0.22\sqrt{\frac{\sigma_{cr}}{\sigma_{max}}}\right) \qquad \text{(WINTER)} \qquad (11)$$

WOLMIR [12] indicates however that formula (8) applies only for $\frac{\sigma_{cr}}{\sigma_{max}} > 0.2$. He mentions also that formula (9) of von KARMAN is especially applicable in the case of stiffeners whose relative rigidity is much less than γ_p^*. The same is true for formula (11) of WINTER, which derives from von KARMAN's formula.

The table which follows gives the values of $\phi = \frac{b_e}{b}$ for various plates, calculated with formulae (7) to (11), and compares them with the values obtained by the theory of SKALOUD-NOVOTNY.

The comparison of the results shows that the results of PAPCOVICH, SECHLER and SKALOUD agree generally sufficiently well.
The MARGUERRE values also are satisfactory in the domain $\frac{\sigma_{cr}}{\sigma_y} > 0.2$, whereas the values derived from the KARMAN and WINTER formulae are substantially lower.
Using formulae (7) and (10) which are the most satisfactory, we have tested the result obtained experimentally by DUBAS with his test specimen N° 1.

Données						PAPCO-VITCH	(*) MARGUERRE	(*) von KARMAN	SECHLER	(*) WINTER	SKA-LOUD
nombre de raidis-seurs	$\frac{b}{t}$	$\frac{b'}{t}$	α'	σ_{cr} Kg/cm²	$\frac{\sigma_{cr}}{\sigma_y}$						
1	200	100	2	759	0.316	0.617	0.680	(0.563)	β=1 0.658	(0.493)	0.65
1	270	135	2	417	0.174	0.537	(0.558)	(0.417)	β=1 0.587	(0.378)	0.58
1	360	180	2	240	0.100	0.496	(0.464)	(0.316)	β=1 0.550	(0.294)	0.54
1	450	225	2	150	0.062	0.475	(0.396)	(0.249)	β=1 0.531	(0.235)	0.53
2	210	70	3	1550	0.646	0.802	0.865	0.804	β=1 0.823	(0.662)	0.82
2	300	100	3	759	0.316	0.617	0.680	(0.563)	β=1 0.658	(0.493)	0.66
2	375	125	3	486	0.202	0.553	0.598	(0.450)	β=1 0.601	(0.409)	0.60
2	450	150	3	337	0.140	0.518	(0.520)	(0.374)	β=1 0.570	(0.344)	0.57

Values of $\phi = \frac{b_e}{b}$ or $\frac{b_e'}{b'}$ for a stiffened plate ($\alpha = 1$) whose stiffeners remain rigid up to collapse, for $\sigma_{max} = \sigma_y = 2400$ Kg/cm².
((*) see the remarks made after formula (11))

8. APPLICATION TO DUBAS TEST N° 1

For the actual dimensions of the model DUBAS, the parameters have following values :

thinness of the plate : $\frac{b}{t} = 242$;

side ratio of the panel : $\alpha = 1.125$

relative rigidity : $\gamma = 17.7$

relative area : $\delta = 0.041$

From these characteristics, DUBAS draws from his buckling chart (Fig.4) pertaining to a uniformly compressed plate with three longitudinal stiffeners the value of the buckling coefficient

$$k = 50.7.$$

As the "optimum" relative rigidity γ^*, in the sense of the linear theory, is about 20, the stiffeners will be deformed by the plate and this fact has, indeed, been observed experimentally by DUBAS.

The critical stress calculated by linear buckling theory is

$$\sigma_{cr}^{lin} = 50.7 \cdot \frac{1,900,000}{(\frac{b}{t})^2} = 1640 \text{ Kg/cm}^2.$$

From the measurements made for P = 1.75 t, a load for which the whole cross section is effective, we calculate the value of coefficient

$$K = \frac{\sigma}{P} = \frac{335}{1750} = 192.10^{-3} \text{ cm}^{-2}.$$

As the experimental collapse observed by DUBAS does not coïncide with SKALOUD's collapse criterion adopted in present paper (sec. 4), we must assume that collapse occurs when the yield point $\sigma_y = 3000$ Kg/cm^2 is reached at the edges of the stiffened plate, or, equivalently, when ε reaches $\frac{3000}{2,100,000} = 1.43$ ‰ . From the (P,ε) diagram given by DUBAS (his fig. 8), we see that corresponding load is P = 7.4 t and constitutes <u>our</u> collapse load. The corresponding mean compressive stress calculated by NAVIER formula

$$\bar{\sigma}_{exp}^{(1)} = 192.10^{-3} \times 7400 = 1420 \text{ Kg/cm}^2$$

is the experimental collapse stress.

This stress is less than the critical buckling stress of the linear theory in the ratio

$$\frac{\bar{\sigma}_{exp}^{(1)}}{\sigma_{cr}^{lin}} = \frac{1420}{1640} = \frac{1}{1.15}.$$

If now be abandon SKALOUD's collapse criterion and revert to the experimental collapse load P = 7.95 t. observed by DUBAS, we have to adopt as collapse stress $\bar{\sigma}_{exp}^{(2)} = 1525$ Kg/cm^2.

It is seen that this collapse stress of the whole structure is lower than the critical stress of the stiffened plate given by linear theory, which justifies the statement made at the beginning of the introduction of this paper.

In his report, professor DUBAS has applied the effective width formula to estimate the magnification factor $m = \gamma^*_p/\gamma^*$ of the stiffeners. By applying effective width formulae (7) and (10) to the full plate of DUBAS considered as unstiffened, we obtain successively :

$$\beta = \alpha = 1.125 \; ; \; \sigma_{cr} = \frac{7{,}593{,}400}{(242)^2} = 130 \text{ Kg/cm}^2 \; ; \; \sigma_{max} = \sigma_y = 3000 \text{ Kg/cm}^2$$

$$\frac{\sigma_{cr}}{\sigma_y} = 0.0434.$$

a) PAPCOVITCH formula (7) gives $\frac{b_e}{b} = 0.465$ whence $\bar{\sigma} = 1395$ Kg/cm^2.

b) SECHLER formula (10) gives $\frac{b_e}{b} = 0.475$ whence $\bar{\sigma} = 1425$ Kg/cm^2.

We see that a rather good agreement exists between the experimental value $\bar{\sigma}^{(2)}_{exp} = 1525$ Kg/cm^2 and the two theoretical estimates.

However, in principle, the use of an effective width formula for the entire plate is not permissible when this plate is stiffened, because the stiffeners increase the stability of the plate, even if they do not remain straight. One of the main aims of future research is precisely to establish the effect of this kind of stiffeners on the mean collapse stress.

The rather good agreement obtained hereabove between DUBAS test and effective width formulae (7) and (10) only means that the stiffeners of DUBAS first test have a very low efficiency in the collapse stage. The unstiffened plate has a very low buckling stress (130 Kg/cm^2), but above calculations show that it has a very large postbuckling strength, which is nearly the same as that of the stiffened plate.

9. RECOMMENDATIONS FOR FUTURE RESEARCH.

Research on stiffened box girders has to be pushed forward theoretically as well as experimentally.

In the field of theoretical research, we need first calculations of the same type as those developed by SKALOUD and NOVOTNY, but for a larger number of stiffeners.

A second phase would consist to investigate the effect of new parameters, such as geometrical imperfections, welding residual stresses, dissymetry of the stiffeners with respect to the mean plane of the stiffened plate, effect of the relative area δ of the stiffeners and of their eventual torsional rigidity (in the case of closed section stiffeners).

From all these non linear calculations, practical design charts should be built, which would take account realistically of the conflicting effects of imperfections and postcritical resistance.

In parallel with above theoretical studies, an important series of tests should be undertaken to control the theoretical results.
In particular, we need some fatigue tests to investigate the effect of the repeated "breathing" of the compressed flange due to its imperfections.
In waiting for the conclusions of such researches, the safety factors against buckling should be immediately increased for box girders, so as to avoid new accidents.

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