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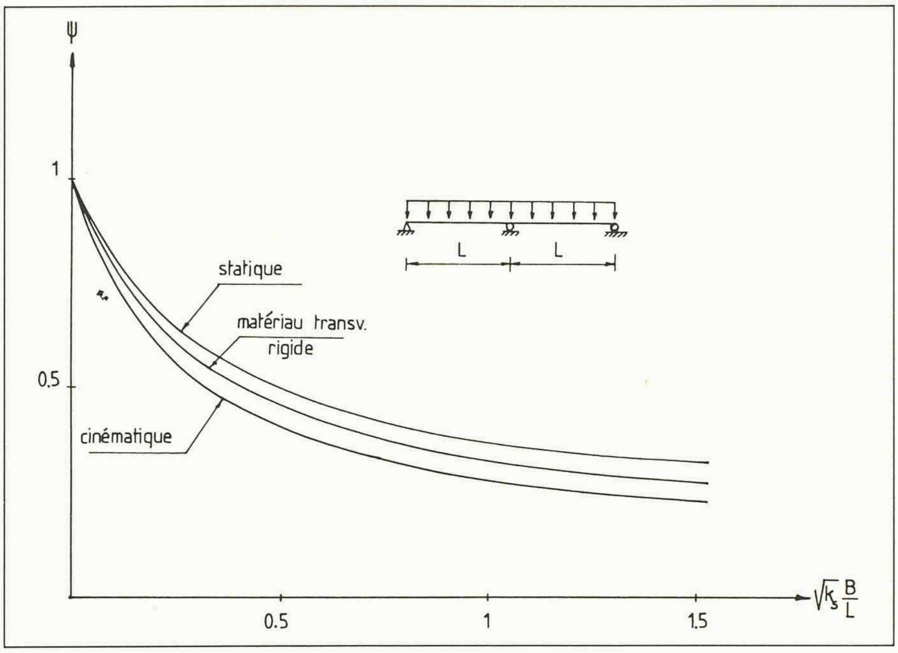


*Fig. 4. — Poutre continue à deux travées égales chargées uniformément.*

Dans le cas d'une semelle isotrope, on a: $E_x = E_y$ et en admettant $E/G = 2{,}6$ pour les matériaux métalliques, $\omega^2$ vaut 2,191. La largeur effective vaut alors:

$$\psi = \frac{1}{1 + 0{,}757\,\frac{Pb}{M}} \qquad (35)$$

On peut généraliser au cas d'une charge supplémentaire uniformément répartie, on trouve aisément:

$$\psi = \frac{1}{1 + \frac{1}{3}\frac{E}{G}\frac{pb^2}{M} + \frac{P\omega b}{4\,M\left(\frac{G}{E_x} + \frac{\omega^2}{21}\right)}} \qquad (36)$$

Il n'y a plus de relation simple entre $\omega$ et $k_s$ et il n'est plus possible d'obtenir une formule générale équivalente à (25). Dans le cas d'un matériau isotrope, pour lequel $k_s = 2{,}6$, on a:

$$\psi = \frac{1}{1 + \frac{1}{12}k_s\frac{pB^2}{M} - 0{,}235\sqrt{k_s}\,\frac{\Delta T \cdot B}{M}} \qquad (37)$$

puisque $B = 2\,b$ *et* $\Delta T = -P$.

5. *Conclusions générales*

L'expression de la largeur efficace réduite s'écrit, en toute généralité, sous la forme simple suivante:

$$\psi = \frac{1}{1 - \alpha\sqrt{k_s}\,\frac{B\Delta T}{M} + \frac{k_s}{12}\frac{pB^2}{M}} \qquad (38)$$

où $k_s$ est un coefficient d'orthotropie (pour plus de détails, consulter [2], [3]), qui dans le cas d'une semelle isotrope se réduit au rapport $E/G$.

En développant successivement l'approche «exacte» ($r$) pour le matériau transversalement rigide [3], puis l'approche cinématique ($c$) et l'approche statique ($s$), on trouve que le coefficient $\alpha$ correspondant prend les valeurs ci-après:

$\alpha_r = 0{,}270 \qquad \alpha_c = 0{,}323 \qquad \alpha_{st} = 0{,}235$

Cette dernière valeur $\alpha_{st}$ n'est valable que pour un matériau isotrope; si la valeur de $k_s$ augmente, elle se rapproche de $\alpha_r$.

Les figures 3 et 4 fournissent respectivement les résultats obtenus pour la largeur efficace d'une part, à mi-portée d'une poutre sur deux appuis, chargée d'une force concentrée dans cette même section, et d'autre part, sur l'appui intérieur d'une poutre continue à deux travées égales, chargée uniformément. On observera que l'approche basée sur l'hypothèse du matériau transversalement rigide est sensiblement la moyenne des deux autres.

Tout en gardant à l'esprit que, en toute rigueur, les solutions proposées ne donnent des bornes que pour l'énergie, le faible écart entre les courbes obtenues donne à penser que l'approche préconisée fournit des résultats fort voisins de la solution exacte.

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# Ultimate Load Behaviour of Longitudinally and Transversally Web Plates Loaded in Shear

by. Michele Mele, Rome and Roberto Puhali, Trieste

*Summary*

The paper briefly reports a general method for designing stiffened webs loaded in shear, which allows for any kind of stiffening. The cases of compact and stiffened flanges are separately dealt with. The theoretical procedures, the experimental and numerical research programmes carried out during the last ten years, as well as the comparison with other design methods are described.

## 1. Introduction

In more recent years a lot of experimental and theoretical work has been devoted to the study of stiffened web panels loaded beyond the critical load until the point of collapse. Among the design methods which have been developed as a result of these research works the one by Rockey, Evans and Porter [1][1] named Cardiff Method, is well known and has already been introduced into some codes. Another design method, named Trieste Method [2], was proposed by the authors [3], [4] and was introduced into the Italian Code in 1973. Its main features are to

[1] See references at the end.

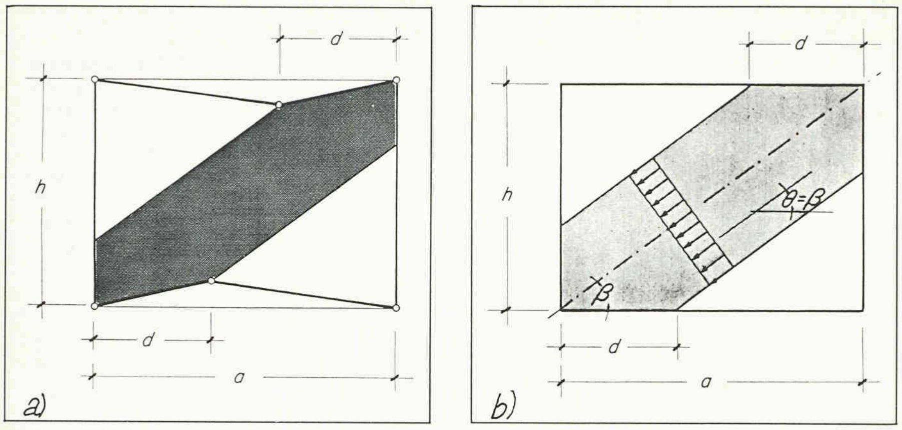



*Fig. 1. — Collapse mechanism.*

point out a new design philosophy which requires ultimate and serviceability checks and to allow for the most general web stiffening; in fact transverse ribs and longitudinal ones as well may be considered.

All these methods can be applied to design webs in plate girders built up with compact flanges; so they cannot be employed when "no compact" stiffenend flanges are used. The last case is typical of steel bridges, when orthotropic plates are …ed deck or bottom flange of box girders.

This paper reviews the results of an extensive theoretical and experimental research project the authors have been performing for ten years in order to extend the design method to bridge girders characterised by:

- low values of the aspect ratio $\alpha$,
- longitudinal stiffening,
- stiffened flanges.

## 2. Plate Girders with Compact Flanges

As already pointed out the proposed method requires checking at the critical and ultimate state as well.

The critical state is considered as a serviceability limit state and the check is performed by limiting the shear stress in web panels and the relative rigidity of transverse stiffenings:

$$\tau \leqq \tau_{cr} \qquad \gamma \geqq \gamma^*. \qquad (1)$$

Such a control is justified by the fact that critical load is the threshold of sensitivity to deformations for perfect or near perfect structures [3]. Also for fatigue prevention as well as for psychological motives, the deformations must be limited under service loads.

The collapse mechanism adopted for calculating the ultimate load (fig. 1) ignores the frame contribution to shear resistance and assumes for the slope of the tensile stress field in the panel an angle equal to the one of the geometrical diagonal if the aspect ratio $\alpha$ is higher than one, otherwise this angle must be taken as equal to 45°. It is further assumed that the diagonal stresses will take the shape of a strip and that plastic hinges be located where the strip joins the flanges. The presence of longitudinal stiffenings is taken into account by fictitious values for the web thickness.

The transverse stiffeners, which are loaded by an axial force and a bending moment, are checked by the simple formula [5]:

$$\frac{P}{P_{cr}} + \frac{M}{M_p\,(1 - P/P_E)} \leqq 1. \qquad (2)$$

The critical load ($P_{cr}$) and the Euler one ($P_E$) are calculated assuming an affective length of the strut equal to half of the length of the stiffener; this is to allow for the load variability along its length and the actual constraint conditions. The transverse resistant section of the stiffener is defined by taking into account a collaborating width of web equal to [4], [6]:

$$b_w = 70\ t_w\ \left(1 - \frac{\sqrt{3}\ \tau_{cr}}{f_{yw}}\right) \qquad (3)$$

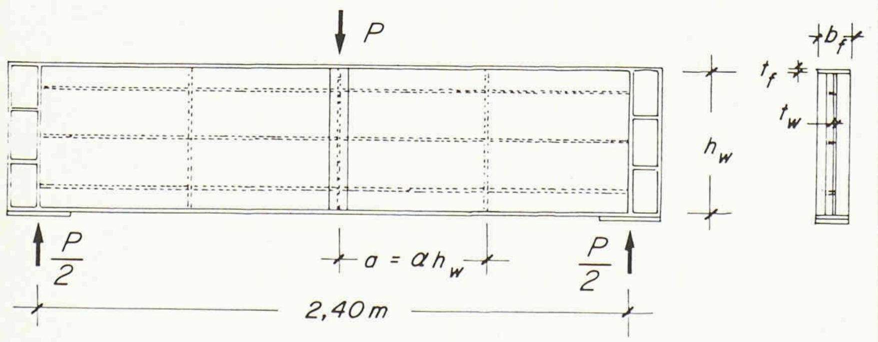
The longitudinal stiffeners are designed assuming that their main job is to increase the critical load of the web panel between two adjacent transverse ribs; so only the flexural rigidity is to be checked.

The reliability of this simple design method has been controlled by an extensive research programme carried out by experimental and numerical tests [4], [5], [6], [7].

The experimental research programme was carried out by testing twentyseven beams. They were built up like the three basic models illustrated in figure 2, but they differed one from each other in the web stiffening. During the tests strains in the flanges and web, vertical deflections at midspan of the beam and transverse deflections of the web were recorded.

The main conclusions drawn from the tests are:

- the ultimate loads predicted by the proposed design method are very close to the experimental ones, and in actual fact an average error of 2,1% with a standard deviation of 7,3% was obtained;
- in the case of aspect ratios ($\alpha$) less than one, the results are more realistic then those obtained by applying the Cardiff Method, this is expecially true when "smeared" longitudinal stiffenings are used;
- the same collapse mechanism is capable of describing the ultimate behaviour of webs longitudinally stiffened or not (figs. 3, 4, 5);
- longitudinal stiffeners, if there are only one or two, can only increase the critical load. As a matter of fact when their flexural rigidity is higher than the * value no significant differences were observed in the case of $\gamma = \gamma^*$, as far as either the pre-critical or the post-critical behaviour of the beam is concerned;
- when the number of the longitudinal stiffeners is more than two they can be considered "smeared" along the




| | $h_w$ (mm) | $b_f$ (mm) | $t_f$ (mm) | $t_w$ (mm) |
|---|---|---|---|---|
| GIRDER 1 | 600 | 90 | 8 | 2 |
| GIRDER 2 | 600 | 150 | 8 | 2 |
| GIRDER 3 | 900 | 150 | 8 | 2 |

*Fig. 2. — Test set up; Dimensions of the three basic models.*

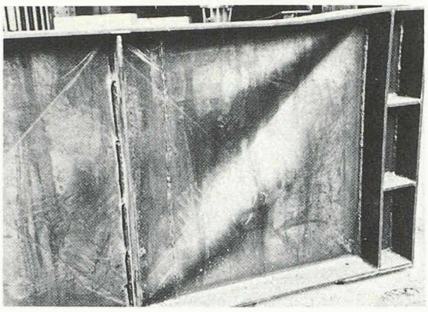


*Fig. 3. — Typical collapse of a girder without longitudinal stiffener.*

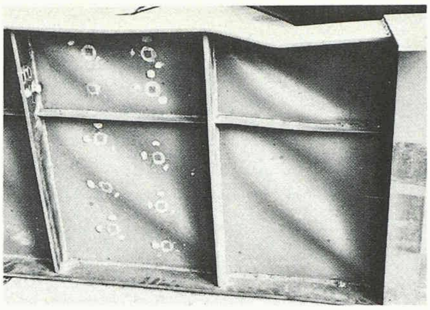


*Fig. 4. — Typical collapse of a girder with one longitudinal stiffener.*

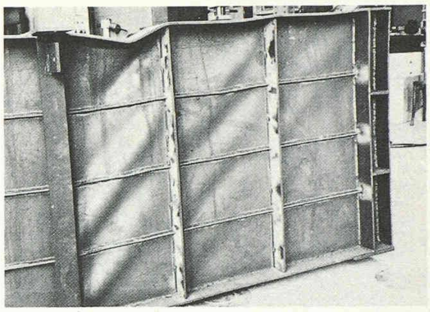


*Fig. 5. — Typical collapse of a girder with three longitudinal stiffeners.*

web depth: then fictitious values for the web thickness can be determined and the same formulas valid for non longitudinally stiffened webs are still valid.

Furthermore the numerical tests demonstrated that the results obtainable from the proposed method for checking the transverse stiffeners are very close to those achieved throught the approach proposed in [8] by Rockey, Valtinat and Tang, who supported their thesis by careful experimental work. But the present method may offer certain advantages, particularly in terms of practical simplicity and conceptual clarity, as well as being closer to the general lines of most current codes. The dimensioning of stiffeners through the B.S. 5400 method, as already pointed out in [9], would seem to be overprudent, leading to uselessly uneconomic design.

Initial imperfections in the stiffeners seem to have very little influence, and the same may be said of the destabilising effects due to states of stress, whether normal or tangential, acting on the web. However, in particular cases, the method proposed here could easily be refined: geometrical imperfections could be taken into account, or the normal stress acting on the stiffener could be fictitiously increased in the usual way, in order to evaluate particularly serious stress states in the web that might occur, for example, in continuous beams close to the intermediate supports, or in stayed-cable beams.

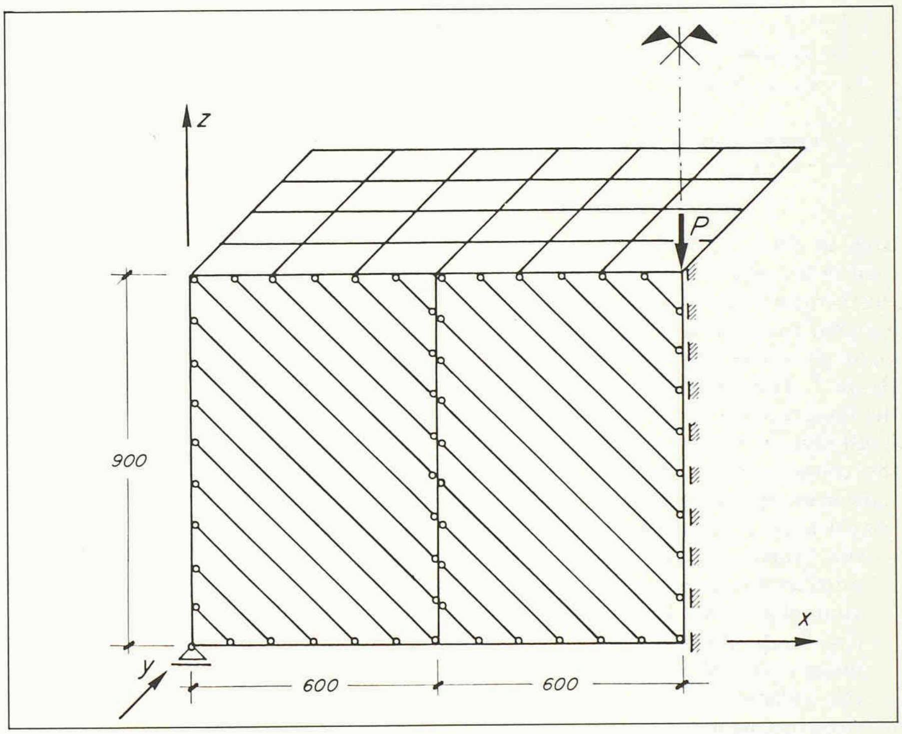



*Fig. 6. — Analytical model for box girders with stiffened flanges*

## 3. Plate or Box Girders with Stiffened Flanges

Neither the Cardiff nor the Trieste Method can be applied to plate or box girders built up with stiffened flanges, because they only allow for compact flanges; nevertheless it seems to be too conservative to neglect completely the contribution of stiffened flanges to the ultimate load capacity. In order to enlight the correct way to allow for the rigidity and strength of the flanges a simple numerical model has been proposed [10], [11]. The analysis is performed in the post-critical range only by beam and truss finite elements; the former ones simulating flanges and stiffeners and the latter ones

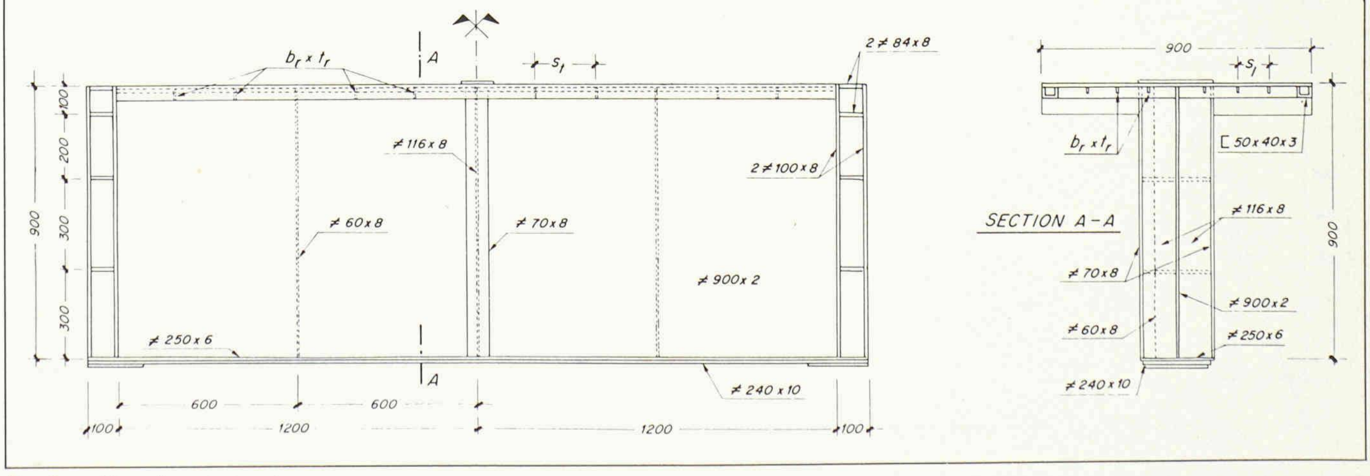
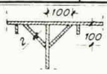



*Fig. 7. — Size of one test series.*

TAB. I: *Geometrical properties of test girders*

| GIRDER NUMBER | PARAMETERS OF STIFFENED FLANGE RIBS (FOR EACH WEB PANEL) | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | LONGITUDINAL | | | | TRANSVERSE | | | |
| | N° | $s_l$ [mm] | $b_r$ [mm] | $t_r$ [mm] | N° | $s_t$ [mm] | $b_r$ [mm] | $t_r$ [mm] |
| 1 * | 3+3 | 100 | 20 | 2 | 2 | 200 | 40 | 4 |
| 2 | 3+3 | 100 | 20 | 2 | 1 | 300 | 40 | 4 |
| 3 | 2+2 | 133 | 30 | 3 | 2 | 200 | 50 | 5 |
| 4 * | 2+2 | 133 | 30 | 3 | 1 | 300 | 50 | 5 |
| 5 (2) | 3+3 | 100 | 20 | 2 | 2 | 200 | 40 | 4 |
| 6 (2) | 2+2 | 133 | 30 | 3 | 2 | 200 | 50 | 5 |
| 7 (1) | 3+3 | 100 | 20 | 2 | 2 | 200 | 40 | 4 |
| 8 (1) | 2+2 | 133 | 30 | 3 | 2 | 200 | 50 | 5 |

(1) GIRDERS 7 AND 8 HAVE A "WINE GLASS" FLANGE




(2) GIRDERS 5 AND 6 HAVE THE BOTTOM FLANGE STIFFENED

used to allow for the diagonal tension field in the web (fig. 6).

Furthermore an experimental programme has been performed by testing eight girders of the type illustrated in figure 7. The geometrical properties of the flanges are reported in table I.

Until now extensive numerical tests and the complete experimental programme have been carried out and the following conclusions can be drawn:

- the proposed analytical model can satisfactorily predict the real behaviour of girders with stiffened flanges (fig. 8), the higher percentage differences observed were less then 5%;
- the collapse mechanism is characterized by diagonal web strips of tensile stresses anchored both to vertical stiffeners and flanges, with the strip width anchored to the flanges depending on the rigidity of the flanges;
- the ultimate loads are considerably higher than the values predicted by the "true Basler solution" with an improvement in carrying capacity provided by the stiffened flanges which averages 20% in the usual cases of orthotropic plates;
- a not negligible increase in the collapse load (more then 15%) can be achieved by employing "wine glass" connections between web and flanges;
- particular care must be paid to the design of the end-posts in order to avoid their premature failure, particularly in the case of not-very-stiff flanges.

## 4. Conclusions

The simple method proposed by the authors (Trieste Method) can be safely used to design webs of plate girders built up with compact flanges in the presence of any kind of web stiffening by checking both the serviceability and the ultimate limit states.

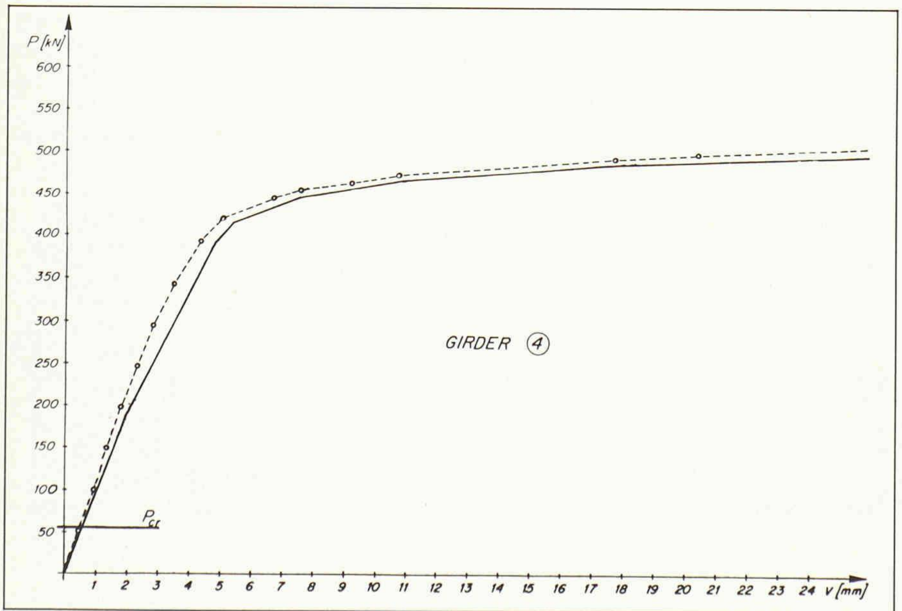



*Fig. 8. — Numerical model versus test results.*

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When orthotropic plates are used as flanges, the behaviour of the girder can be closely predicted by a numerical method using linear finite elements which allow for the contribution of the rigidity of the flanges.

At present further numerical investigations are in progress in order to achieve simple formulas which would enable us to extend the "Trieste Method" to the most general case of stiffened flanges avoiding the use of the finite elements method in practice.

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