English translation of part of the text of pages 13, 14 and 15 of the book: Beulwerte ausgesteifter Rechteckplatten, von K. Klöppel und K.H. Möller

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Englische Übersetzung eines Teils des Textes der Seiten 13, 14 und 15 des Buches: Beulwerte ausgesteifter Rechteckplatten, Bd. II von K. Klöppel und K.H. Möller, Verlag W. Ernst und Sohn, Berlin 1968 (Mit Genehmigung von Prof. K. Klöppel)

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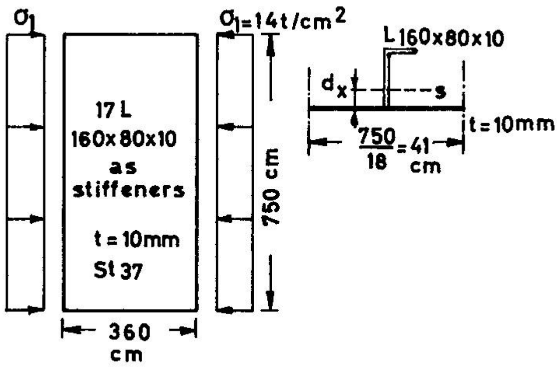
In the use of tables for uniformly distributed stiffening, it is necessary exert cautisuness about two questions :
First, about the calculations of the effective moment of inertia, secondly, about the safety required against buckling.
The Standard DIN 4114 Ri 18.13. accepts, for eccentric stiffeners, to relate the moment of inertia to the upper edge of the plate. The basis of this rule is an assumption about the effective width of a corresponding strip of the buckled plate. But when the stiffeners are regularly spaced, the effective width cannot be larger than the distance between two stiffeners.
If the moment of inertia of such a plate strip is calculated, then values significantly smaller than according to DIN 4114 are usually obtained. For closely spaced stiffeners, it is therefore always recommended to calculate the moment of inertia both according to the Standard and also for a plate strip composed of a stiffener and a web strip having as width the distance between two stiffeners, the example figure 11 should serve an illustration. According to DIN 4114, the moment of inertia with respect to the upper edge of the web is

$$J_{DIN} = J_L + F_L\ (a - e_x)^2 = 6\ 11 + 23{,}2 \times (16 - 5{,}63)^2 = 3115\ \text{cm}^4$$



With the largest value for the effective width, namely the distance between two stiffeners, the distance of the controïd to the upper edge of the web is found to be, according to fig. 11 :

$$d_x = \frac{F_L\ (a + \frac{t}{2} - e_x)}{F_L + F_{web}} = \frac{23,2\ (16 + 0,5 - 5,63)}{23,2 + 41} = 3,92$$

which yields

$$J = J_2 = F_L\ (a + \frac{t}{2} - e_x - d_x)^2 + F_{web}\ d^2 = 2360\ cm^4$$

It is seen therefore that J_{DIN} does not apply for the calculation of the buckling stress.

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As already mentioned in the Introduction, one has to distinquist between the linear buckling, connected with idealized structural and geometrical assumptions, with the buckling load as limit load, which is the basis for the specification DIN 4114 and for the present book, and a second postcritical non linear buckling, discribed by non linear differential equations to which is associated a limit load which is called ultimate load. This load is reqularly higher than the buckling load, because additional membrane stresses come into play. On this theoretically more complicated way, the influences of unavoidable initial deflections may also, for example, be taken in consideration. The ultimate load above the buckling load can be determined either by the entry into the plastic range or by inacceptably large deformations.

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It is necessary to come back to the question whither by plate buckling a second equilibrium state establishes itself or not. Let us imagine the plate to be studied by linear buckling theory in the buckled state. In addition to the membrane stresses existing already before buckling, we have now flexural stresses, that may be considered to be produced by the fictition transverse load

$$q = \sigma_x t \frac{\sigma^2 w}{\sigma_x^2} + \sigma_y t \frac{\sigma^2 w}{\sigma_y^2} + \tau t \frac{\sigma^2 w}{\sigma_x \sigma_y}$$

where σ_x, σ_y and τ are the edge stresses of the plate. If the buckling shape is not known, one shall imagine that the plate is subjected to a uniform transverse load. A large limit load is then to await, when these fictitions transverse loads are supported in all directions nearly on the same way, in other words when we have an extended plate action. If, on the contrary, these imagined transverse loads are supported only in the direction of the greatest compressive stresses, then we can as a good approximation to reality imagine the plate to be cut in strips by cuts parallel to the largest compressive stresses. The individual beamlike strip wall then behave like a buckled stult.

It is recommended now, for plates which support the fictitions transverse loads nearly uniquely in the direction of the large compressive stresses, to require larger safeties than $\nu_b = 1,25$ or $\nu_h = 1,35$.

According to the opinion of the authers, which is supported by sections 17.5 and 17.6 of the DIN 4114, safeties are to be required which in the limit case coïncide with those for the compressed bar. Numerical comparisons have led to following formule - which cannot be demonstrated, which should give the required safety

$$\varphi_{\beta}^{k} = \frac{\varphi_k + 100\,\bar{\alpha}^2\,\varphi_\beta}{1 + 100\,\alpha^{-2}}$$

The side ratio $\bar{\alpha}$ is obtained through the differential equation of the orthotropic plate, under the assumption of a definite torsional rigidity, by a coordinate transformation. $\bar{\alpha}$ is the side ratio of an unstiffened comparison plate with the same buckling shape as the stiffened plate :

$$\bar{\alpha} = \alpha \sqrt[4]{\frac{1 + \Sigma\gamma_Q}{1 + \Sigma\gamma_2}}$$

An unstiffened plate with the side ratio α behave therefore similarly as an unstiffened plate with the side ratio $\bar{\alpha}$.

For the above example, (fig.11), γ_L = 615 and

$$\bar{\alpha} = 0{,}48\sqrt[4]{\frac{1}{616}} = 0{,}096$$

According to Table 7 of DIN 4114, we have for loading case 1

$$\varphi_k = 2{,}18$$

and one obtains

$$\varphi_{\beta}^{k} = \frac{\varphi_k + 100\,\bar{\alpha}^2\varphi_\beta}{1 + 100\,\bar{\alpha}^2} = 1{,}69$$

This safety against buckling at least should be required, in this example, in the opinion of the Authers.

In the choice of the stiffener section, it must be in addition taken care that stiffeners do not buckle before reaching the plate buckling load.

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