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SERVICEABILITY LIMIT STATE OF THE WEBS OF STEEL GIRDERS.

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Summary.

The aim of the paper is to discuss the serviceability limit state of the webs of steel girders. Simple formulae are given so as to make it possible:

i) To analyse the limit state of serviceability of the plate elements encountered in steel buildings and bridges ,

ii) To determine that limiting value of the depth-to-thickness ratio which enables the designer to disregard entirely the point of view of serviceability.

1. Introduction.

The first and second authors proposed, jointly with Ch. MASSONNET (MAQUOI et al, 1981) and in one case also with Ph. JETTEUR (JETTEUR et al, 1983), a new approach to the ultimate limit state (ULS) of the plate elements of structural steelwork. Let us briefly recall here that the analysis was governed by two criteria :

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(i) the criterion of plate buckling ,
(ii) that of local yielding.

For example, in the case of a web panel subject to combined compression, bending and shear, the analysis via the first criterion leads to the relationship :

$$(\sigma_c/S_c f_y) + (\sigma_b/S_b f_y)^2 + (\tau\sqrt{3}/S_s f_y)^2 \leq 1, \quad (1)$$

where σ_c, σ_b and τ denote the average loads acting on the panel considered, f_y is the yield stress of the web material, and S_c, S_b and S_s are so-called buckling factors (for whose formulae the reader is referred to the aforementioned references) which - taking account of (i) postcritical behaviour, (ii) the effect of initial imperfections - determine the ultimate load of the panel under consideration.

The other criterion then furnishes the relationship :

$$\{[\sigma_c + (\sigma_b/n)]/f_y\}^2 + (\tau\sqrt{3}/f_y)^2 \leq 1, \quad (2)$$

σ_c, σ_b and τ being the stress peaks at the most loaded point of the panel studied, and n a factor (with n = 1.3 for buildings and n = 1.15 for bridges) that allows for a certain adaptability of the bending stress peaks due to edge yielding.

2. Factors Determining the Serviceability Limit State of Webs.

While the ULS corresponds to factored loads (i.e. actual loading multiplied by a certain load factor > 1), the serviceability limit state (SLS) is related to working loads, i.e. to loading to which the structure is subjected (and almost every day of its life) under service.

Then it is in the nature of things that the rules governing the SLS can be given by other considerations than those defining the ULS ;nevertheless, it is appropriate to have some conditions and criteria which can safeguard all aspects of satisfactory performance of webs under service.

Which are these conditions ?

In the authors'view, the main of them can be formulated as follows :

(i) The magnitude of web deflection, since a large deflection (and particularly so when this can be compared with the straight line of the edge of a flange, stiffener and the like) can have a disturbing effect on observing persons.

(ii) Pronounced "breathing" of the web under ordinary cyclic loading, when the structure is loaded and unloaded in very frequent cycles, which may make the observing person feel that the structure is not strong enough.

(iii) Snap-through phenomena, which occur in the buckled pattern of the web under the aforesaid cyclic loading, and which (despite the fact that they almost never have anything in common with a limitation of some important functions of the structure) can induce in public the feeling that the structure is in danger.

Thus we can see that the limit state of serviceability is governed rather by aesthetic or psychological considerations than by those connected with strength.

It has been observed that a SLS analysis can be conducted advantageously with the aid of critical load yielding from linear buckling theory. Thus K.C. ROCKEY concluded, after studying during an extended period of time the reaction of his students to various magnitudes of the buckled surfaces of the webs of his test girders, that web buckling was practically unobservable up to a load equal to 1.5 times the critical load (ROCKEY, 1958). Then it follows from theoretical investigations that, in the case of initially imperfect webs, snap-through phenomena occur under loads which are a little higher than the critical load of the corresponding "ideal" web.

Therefore, the authors propose that the analysis of the SLS be carried out by means of the following condition :

$$(\bar{\sigma}_c/\sigma^o_{c,cr}) + (\bar{\sigma}_b/K\sigma^o_{b,cr})^2 + (\bar{\tau}/K\tau^o_{cr})^2 \leq 1, \qquad (3)$$

where $\bar{\sigma}_c$, $\bar{\sigma}_b$ and $\bar{\tau}$ are average stresses acting on the web panel under the working loads, $\sigma^o_{c,cr}$, $\sigma^o_{b,cr}$ and τ^o_{cr} are the critical loads of the same panel (when loaded by (i) compression, (ii) bending, (iii) shear), and the factor K = 1.5 for buildings and K = 1.1 for bridges.

Of course, the critical stresses $\sigma^o_{c,cr}$, $\sigma^o_{b,cr}$ and τ^o_{cr} are calculated for the actual boundary conditions of the panel under consideration. When we have not enough data to proceed in this way, the values of $\sigma^o_{c,cr}$, $\sigma^o_{b,cr}$ and τ^o_{cr} are evaluated at least approximately using this simple formula for the buckling coefficient k :

$$k = \sqrt{k_{s,s}\, k_f}\,, \qquad (4)$$

where $k_{s,s}$, k_f are the coefficients for (i) a simply supported panel, (ii) a fixed panel, respectively. The formulae for them are listed in the following table, where $\alpha = a/d_i$ is the aspect ratio of the web panel or subpanel.

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A full description of the obtained results and conclusions (the results were plotted in charts similar to those shown in Fig. 3, which gives five curves : one for the plate buckling ULS, two for the local yielding ULS and two for the SLS) will be presented in a more comprehensive publication (JASPART et al, 1986) that is going to appear shortly. Here, only some of the results regarding the analysis of (i) webs fitted with a longitudinal rib at 1/5 of the web depth, (ii) longitudinally unstiffened webs are given. As in this analysis the webs are regarded as being parts of bridge structures, K = 1.1 and η = 1.15.

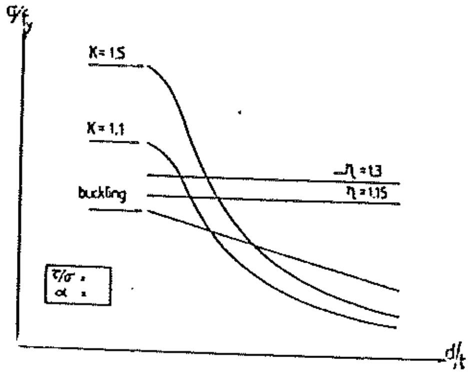

Figure 3

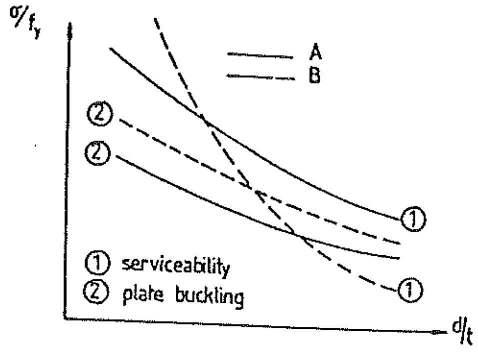

Figure 4

3.1. A Web Fitted with a Longitudinal Rib at One Fifth of the Web Depth.

The longitudinal rib is assumed so strong that it remains rigid up to collapse of the web ; thus only the behaviour of both subpanels needs to be considered. The obtained results can be summed up as follows in the range d/t<500 :

(a) The influence of the magnitude of the yielding factor η is only weak when τ/σ = 0, and tends to disappear completely as this ratio grows.

(b) For the subpanel A (d_1/d = 0.2), the criterion of local yielding governs the analysis only when small values of τ/σ are combined with very small values of d_1/t ; in the case of the other subpanel B (d_2/d = 0.8), the point of view of local yielding prevails only for τ/σ inferior to about 0.5.

(c) For the values studied of d/t and τ/σ , the decisive criterion for the subpanel A is always that of plate buckling. For the subpanel B, local yielding and serviceability successively govern as the d/t ratio grows ; when τ/σ remains small ($\leq \sim$ 0.5), only local yielding and serviceability are successively decisive.

In the case considered, the design criterion is defined by the lower envelope with respect to the behaviour of both subpanels (Fig. 4).

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3.2. A Web without Longitudinal Ribs.

In this case the results of the authors' parametric study can be summarized as follows on the range $100 < d/t < 300$:

(a) Similarly to the above case of a longitudinally stiffened web, the effect of the magnitude of the yielding factor η is very little significant for $\tau/\sigma = 0$, and very quickly becomes entirely insignificant as the value of τ/σ grows.

(b) The criterion of local yielding is never decisive.

(c) The design is at first governed by plate buckling failure, and only when the web depth-to-thickness ratio is sufficiently great, serviceability takes over.

(d) The limiting value, $(d/t)_{lim}$, of the depth-to-thickness ratio is dependent on (i) the ratio τ/σ and (ii) the aspect ratio α.

The values of $(d/t)_{lim}$ are plotted in Fig. 6. An inspection of the figure reveals that, for a given value of τ/σ, $(d/t)_{lim}$ grows as the aspect ratio α diminishes ; for a fixed value of α, the limiting slenderness increases or decreases when τ/σ goes up, and fairly quickly converges to a bound. It can also be seen in the figure that for $\alpha = 1.2$ the $(d/t)_{lim}$ ratio is insensitive to a variation of τ/σ.

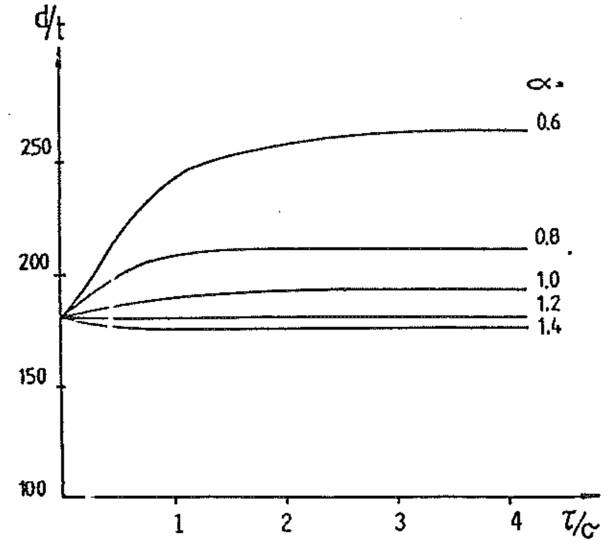

Figure 6

By simulating the curves obtained via a family of bilinear funct
ted so as to get simple and safe results, we come to the followi

$$(d/t)_{lim} = \text{MAX}\ [180 + g(\alpha)\ \tau/\sigma\ ;\ 52\,\alpha + 136/\alpha]\ \text{if}\ g(\alpha) < 0\ ,$$

$$= \text{MIN}\ [180 + g(\alpha)\ \tau/\sigma\ ;\ 52\,\alpha + 136/\alpha]\ \text{if}\ g(\alpha) > 0\ ,$$

where

$$g(\alpha) = 76\ (\alpha^2 - 3\alpha + 2.1) \qquad (0.6 \leq \alpha \leq 1.4).$$

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