DISCUSSION

fib Commission 2, 'Safety and Performance Concepts': mission and activities

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Structural deformations have to be calculated in ultimate limit state design with accuracy only in cases where they directly influence the action effects, for example stability checks based on second-order theory, soil–structure interaction, indirect actions due to restraint deformations, and rotation in plastic regions. For determining the moment distribution in statically indeterminate structures applying first-order theory only the ratios of the stiffnesses or curvatures between hogging and sagging regions have to be taken appropriately into account. Their absolute values are in most cases irrelevant. Due simplifications are acceptable.

When preparing Chapter 6.6 of the CEB–FIP Model Code 1990 (1993) on ultimate limit state of buckling the commission concerned decided that it was absolutely necessary to calculate the deformations based on mean material properties. Otherwise the results do not correspond to widely existing experience, for example with slender reinforced and prestressed concrete masts, bridge piers and towers. The new approach to calculate the deformations as well as the cross-section bearing capacities with the same modified strengths f_{cR} and f_{yR} for concrete and steel as given in equation (4) and to use a global safety factor $\gamma_R = 1.3$ for determining the design value of the bearing capacity $R_{d,R}$ from the bearing capacity R is not useful in cases where the deformations directly influence the action effects. This can be realized by comparing the equations for the bearing capacities following both alternative approaches. Lengthy investigations and comparative calculations are not necessary.

The stress resultants for the cross-section capacity are formulated by

$$R_d = \begin{Bmatrix} N_{d,d} \\ M_{d,d,y} \\ M_{d,d,z} \end{Bmatrix} = \int \begin{Bmatrix} 1 \\ z \\ -y \end{Bmatrix} \sigma_d[\varepsilon(y, z)] dA = \int a \sigma_d[\varepsilon(y, z)] dA \quad (1)$$

Using the short form

$$a = \begin{Bmatrix} 1 \\ z \\ -y \end{Bmatrix} \quad (2)$$

and introducing the design values for the concrete and the steel strength the final equation is obtained:

$$R_d = \int a [-f_{cd} \leq \sigma_{cd}(\varepsilon, f_{cd}) \leq 0, -f_{yd} \leq E_s \varepsilon \leq f_{yd}] dA \quad (3)$$

The new approach using the equation

$$f_{cR} = 0.85\alpha f_{ck}; \quad f_{yR} = 1.1f_{yk} \quad (4)$$

gives the design value for the bearing capacity as shown in equation (6):

$$R_{d,R} = \frac{R}{\gamma_R} = \frac{1}{\gamma_R} \int a \sigma_R[\varepsilon(y, z)] dA \quad (5)$$

$$R_{d,R} = \frac{1}{\gamma_R} \int a [-f_{cR} \leq \sigma_{cR}(\varepsilon, f_{cR}) \leq 0, -f_{yR} \leq \sigma_{sR}(\varepsilon) \leq f_{yR}] dA \quad (6)$$

Modifying equation (6) and replacing the steel stress $\sigma_{sR}(\varepsilon)$ by $E_s \varepsilon$ results in

$$R_{d,R} = \int a \left(\frac{-f_{cR}}{\gamma_R} \leq \frac{\sigma_{cR}(\varepsilon, f_{cR})}{\gamma_R} \leq 0, \frac{-f_{yR}}{\gamma_R} \leq \frac{E_s \varepsilon}{\gamma_R} \leq \frac{f_{yR}}{\gamma_R} \right) dA \quad (7)$$

Introducing the proposed figures from equation (4) and for γ_R , the following equations are obtained:

$$\frac{f_{cR}}{\gamma_R} = \frac{0.85\alpha f_{ck}}{1.3} = \frac{\alpha f_{ck}}{1.529} = 0.98 \frac{\alpha f_{ck}}{1.5} = 0.98 f_{cd} \cong f_{cd} \quad (8)$$

$$\frac{f_{yR}}{\gamma_R} = \frac{1.1f_{yk}}{1.3} = \frac{f_{yk}}{1.182} = 0.97 \frac{f_{yk}}{1.15} = 0.97 f_{yd} \cong f_{yd} \quad (9)$$

They allow equation (7) finally to be written as

$$R_{d,R} \cong \int a \left(-f_{cd} \leq \sigma_{cR}(\varepsilon, f_{cd}) \leq 0, -f_{yd} \leq \frac{E_s}{\gamma_R} \varepsilon \leq f_{yd} \right) dA \quad (10)$$

The new approach reduces the modulus of elasticity for steel by $\gamma_R = 1.3$. In the main this does not result in significant differences compared with the old bearing capacity according to equation (3) if complete use of the design values of strengths is made. Differences only occur for cross-sections with significant amounts of steel only being elastically stressed, as in cross-sections with significant amounts of steel distributed along their sides. The great disadvantage of the new approach is that due to the reduction of

E_s the effective stiffness $dM/d(1/r)$ is even less than when calculating it with design values as in equation (3). Deformations calculated by this method are significantly different from those based on the mean material properties. Therefore the new approach is not acceptable in cases where action effects are directly influenced by deformations. In cases where only the ratios

of deformations have to be met it is acceptable also to determine the deformations with the design values used for cross-section design. These stiffnesses are closer to the mean material behaviour than when calculated using the new approach. Summing up, what can be realized directly by comparing equation (3) with equation (10) is that the new approach is completely superfluous.