Modelling for design

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Summary

In spite of continuous evolution and updating of International and Regional Design Codes for concrete structures in producing physical models able to describe structural concrete behaviour, the design of a concrete structure cannot be considered a blind application of code specifications. On the contrary, the Designer is required to carry out, on the same time, both the processes of analysis and synthesis in selecting the relevant physical models and in adapting them to the peculiar structural problem to be solved. In fact the main aspect of design process is still the creativity, sustained by the code provisions in producing more and more enhanced concrete structures.

Keywords: Concrete Structures, Design Codes, Physical Models

1. Introduction

During the last 25 years the common task of most recognized International and Regional Design Codes for concrete structures was the definition of physical models able to describe structural concrete behaviour in presence of internal actions and their combinations, evaluated as effects of direct/indirect/environmental actions. Despite the presence of some residual lacks in the definition of a few resisting mechanisms, both for serviceability and for ultimate conditions, that are yet covered by empirical rules, the Designer is today supported by a large set of code specifications in conceiving more and more enhanced structures. On the contrary the idea that the design process is today only a blind application of code rules is absolutely misleading, because the main aspect of design is the creativity and design models available within the Codes should be considered only as a tool to conceive the structures and to verify the achievement of required performance criteria and safety level.

For a better underlining of this concept, in the following a set of typical design models and related code provisions will be analysed, putting in evidence the necessity to apply further adapted analysis for their actual application to solve design problems. Just as simplification the presentation will be divided in linear and two-dimensional concrete elements.

2. Linear elements

In this field a very interesting design model is suggested in CEB-FIP Model Code 1990 [1] for columns subjected to axial loads, bending and shear (fig. 1) in which compression may result more or less dominant. In the first case, in which compression is so high that no longitudinal tension reinforcement is required by the design, it should be checked only wheter or not inclined cracking is to be expected. Then, being σ_{c2} and σ_{c1} respectively the principal stresses in compression and in tension, inclined cracking can be avoided if:

$$\sigma_{c1} \leq f_{ctk,min} / 1.5 \qquad \text{for} \qquad \sigma_{c2} < f_{cd} / 3 \qquad (1)$$

$$\sigma_{c1} \leq f_{ctk,min} (1 - \sigma_{c2}/f_{cd}) \qquad \text{for} \qquad \sigma_{c2} \geq f_{cd} / 3 \qquad (2)$$

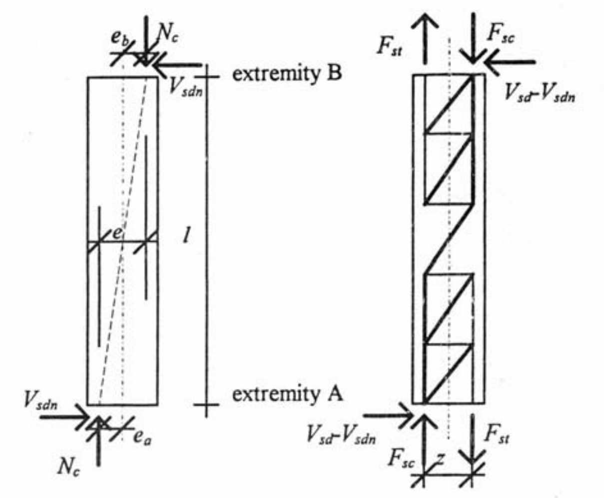

Fig. 1 – Acting action effects and moment diagram in a pier

In the second case, in which compression is less dominant, so that longitudinal tension reinforcement is required by the design, the models 1 and 2 of fig. 1 may be superimposed. In model 1 the column is subjected to the resultant of compression in concrete N_C deriving by the design of extremity sections and to the percentage V_{Sdn} of acting shear V_{Sd}, able to affect the strut geometry, so that its ends are located at the point of application of N_c.

In model 2 the residual portion of axial force FSc, FSt, related to the reinforcement and the remaining percentage of the shear (VSd – VSdn) are transferred across the column by means of a classical truss model. The stress fields coming from the two models may be superimposed, on the safety side, then it should be verified that:

$$\frac{N_c}{b_w\ z}\sqrt{\frac{l^2+e^2}{l^2}}+\frac{V_{Sd}-V_{Sdn}}{b_w\cdot z\cdot\sin\vartheta\cdot\cos\vartheta}\le f_{cd2} \tag{3}$$

$$F_{st}=V_{Sd}-V_{Sdn}\le\frac{A_{sw}\cdot f_{ywd}}{s}z\cot\vartheta \tag{4}$$

It should be recognized that this interesting physical model may result very onerous in the design, when several different load cases are to be considered; but this difficulty may be overcome, with a typical design approach, by the identification of the limitations imposed to the M, N interaction diagrams by the presence of high shear. In practice, following the same approach of those models and expressing the equilibrium conditions in non-dimensional terms, it is possible to draw M, N interaction diagrams, showing the safety domain restricted by the shear effect, for an assigned shear span a/l and column slenderless l/h ratio [2]; fig. 2 illustrates such result for a/l=0.5 and l/h=3÷9. The same approach can be used for boxed rectangular bridge piers, for which case fig. 3 illustrates the influence of shear on M, N behaviour for a/l=1 and l/h=3÷9.

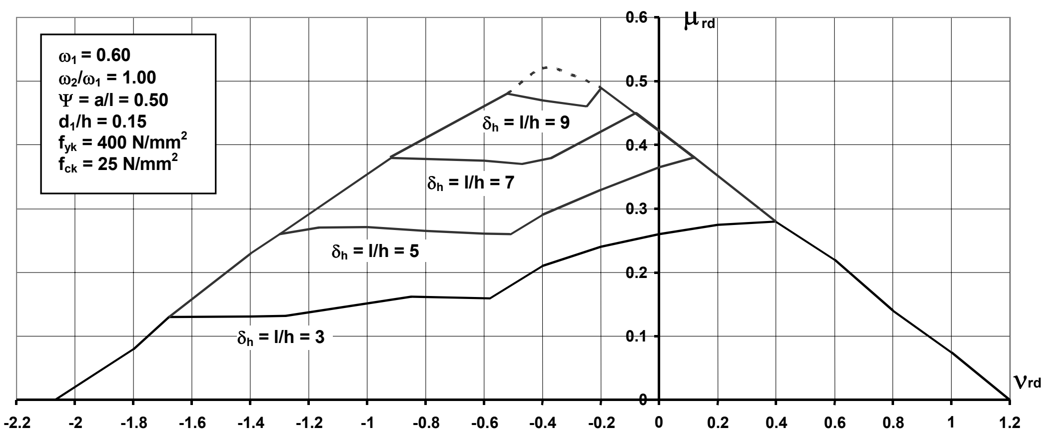

Fig. 2 – M,N,V interaction diagram for solid rectangular piers, plotted for ω = 0.60, Ψ=0.50 and f_{ck}= 25 MPa

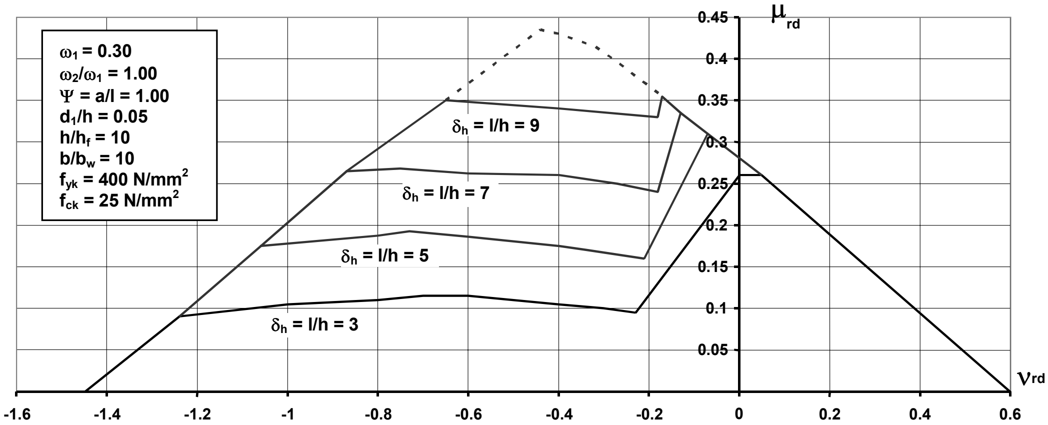

Fig. 3 – M,N,V interaction diagram for a boxed rectangular pier, plotted for $\omega = 0.30$, Ψ=1.00 and f_{ck}= 25 MPa

Within the beams field, the classical resisting model for shear may be used for the definition of interaction bending moment-shear, useful to design intermediate support regions in continuous beams; considering the combination of stress fields within the chords and the web, with their actual extension derived from the equilibrium conditions, and applying a lower bound approach to characterize the model [3], [4], interaction diagrams bending moment – shear can be drawn (fig. 4,5) for predefined longitudinal reinforcement ratio r=As/A's and web reinforcement mechanical ratio

$$\omega_w = \frac{A_{sw} f_{ywd}}{\left(b_w \cdot s \cdot \sin\alpha \cdot f_{cd2}\right)} \left(m = \frac{M}{f_{cd1} \cdot b_w \cdot h^2},\ v = \frac{V}{(h-4c) b_w f_{cd2}},\ c = \mathrm{cov}\,er \right).$$

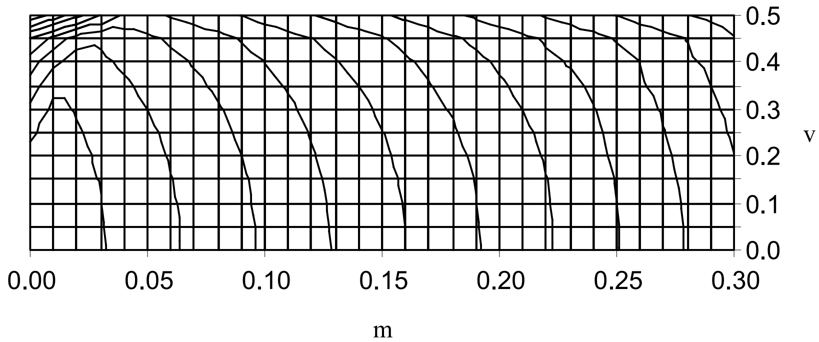

Fig. 4 – Interaction diagram for r=0.25, ω_w = 1

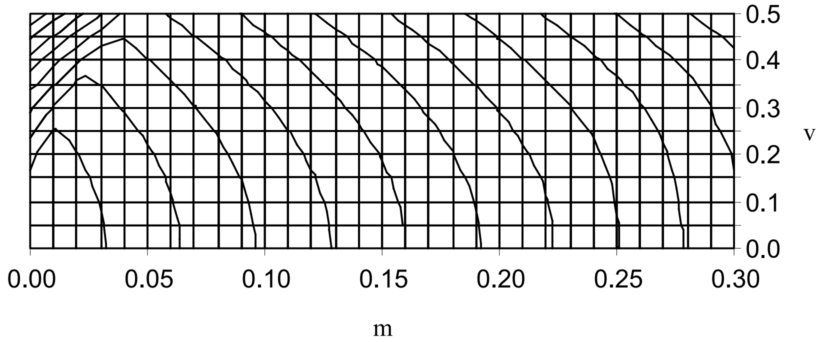

Fig. 5 - Interaction diagram for r = 0.25, ωw = 0.5

A further typical example of the need of engineering adaptation in physical models to cover current design cases is the behaviour in combination of bending moment, shear and torsion of precast segmental continuous decks prestressed with external tendons: a typical combination of internal actions that can be reached in proximity of internal supports of continuous decks. Considering firstly the combination of shear and bending moment one can remark that the classical resisting model for shear should be adapted to take into account the opening of the joints (fig. 6).

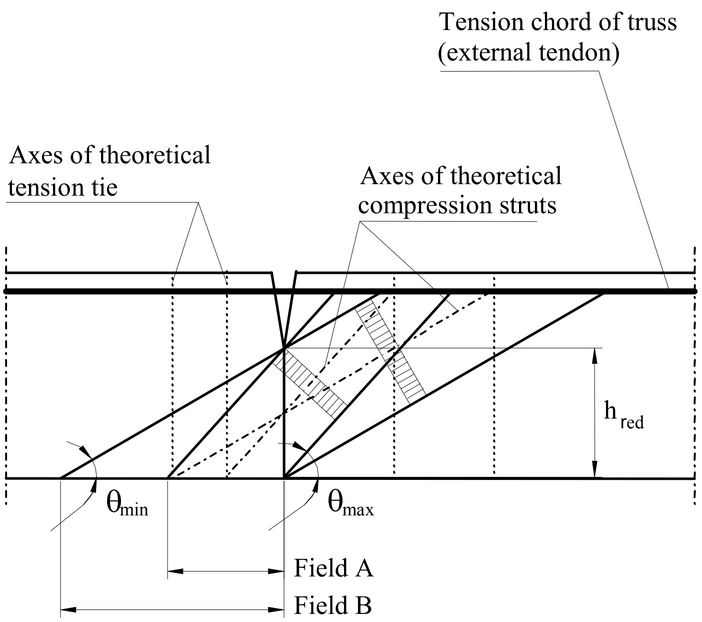

Fig. 6 – Diagonal Stress fields across the joint in the web

First of all one have to ensure that the shear can be transmitted between adjacent segments, supposed to behave like rigid blocks, by means of residual compressed region of the joint

and shear keys effect [5], [6], [7], [8]. The corresponding shear resistance may be expressed as:

$$V_{rd} = \left(N_{sd} \tan\varphi + \sum_i A_{ki} \left(c + \sigma_i \tan\psi \right) + \sum_j A_{kj} c \right) \Big/ \gamma_{rd} \tag{5}$$

where:

- $N_{sd} \tan\varphi$ is the contribution of residual compression region (in presence of the action level corresponding to the beginning of segments slip;
- $\sum_i A_{ki} \left(c + \sigma_i \tan\psi \right)$ is the contribution of shear keys included within the compressed region

(A_{ki} = shear area of "i" key, $c = 0.50 f_{ctd}$, $\sigma_i \le 0.05 f_{cd}$ = local compression residual stress, $\tan\psi = 0.50 f_{cd}^{1/3}$);

- $\sum_j A_{kj} c$ is the contribution of the shear keys included within the open region of the joint;
- γ_{rd} is the model uncertainty coefficient ($\gamma_{rd} \cong 1{,}3$)

If the rigid blocks shear transfer is ensured with the above approach, the resistance of both web tensile and compressive stress fields should be checked. In particular for compression stress field within the web it should be verified that:

$$\sigma_c = \frac{V_{sd}}{b_w h_{red} \sin\theta \cos\theta} \le f_{cd2} \tag{6}$$

that is, having chosen the θ angle, the inequality (6) may be used to derive the maximum opening of the joint ($h-h_{red}$) able to satisfy the required safety level in web concrete stress field.

Tensile resistance of tensed web stress field may be ensured if, in agreement with [9], [10], the necessary suspension stirrups are included within the relevant stress field extension determined by the θ angle. This procedure obviously implies an increase of prestressing force necessary to maintain closed the joint at an extension not smaller than h_{red}, with respect to the required one by a purely design for the acting bending moment (fig.7).