APPLICATION OF THE FINITE ELEMENT METHOD TO THE ANALYSIS OF REINFORCED CONCRETE PANELS

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ABSTRACT

A description is given of a model for the mechanical behaviour of reinforced concrete in plane state of stress, including the initial elastic behaviour, the cracking phenomenon, and plastic yielding of concrete and reinforcement. This model has been applied in the finite element analysis of reinforced concrete panels. A comparative study has been made of the results obtained from two loading cases: 1) a panel in uniaxial tension, 2) a cantilever panel under a concentrated load, with the test results available, and fairly close agreement is observable between the numerical and experimental values.

1. INTRODUCTION

Reinforced concrete displays very complicated mechanical behaviour, including cracking, bond slip, plastic yielding and creep. It is thus intuitively clear, and has also been confirmed by tests, that analysis based upon the assumption of linear elastic behaviour cannot give a reliable prediction of the displacements and stress distribution of reinforced concrete structures. Limit analysis which is in many cases capable of estimating the carrying capacity of reinforced concrete structures, has the drawback that it can not provide information on the displacements.

As a consequence of the complicated mechanical behaviour, analytical solutions for reinforced concrete structures are almost impossible of achievement. However, this difficulty can be overcome in numerical solution procedures on digital computers. In particular, the finite element method has been found a very promising means of deriving more accurate solutions for reinforced concrete structures [1]...[6].

Naturally enough, very great difficulties are involved, despite application of the finite element method and digital computer for solution of the numerical problem. The main obstacle encountered is that of finding a model that will give a sufficiently accurate description of the substantial features of reinforced concrete behaviour, and still be simple enough for use in calculations. Such models have been explained in [1], [3], [4], [5] and [8].

This paper reports on some studies made with the model developed in [1]. It describes initial elastic behaviour, cracking and plastic yield, but does not account for bond slip, dowel action or creep. A finite element programme has been developed for the solution of problems in a plane state of stress. The continued research programme will also include bending action, with applications to reinforced slabs and shells.

2. MODEL OF MATERIAL BEHAVIOUR OF REINFORCED CONCRETE

No more than a brief description is given here of the model for the mechanical behaviour of reinforced concrete, since a more detailed representation can be found elsewhere [1]. The model includes initial elastic behaviour before cracking, the cracking criterion, elastic and plastic behaviour in a cracked state, and plastic behaviour in biaxial (uncracked) compression.

2.1 Concrete

The two main reasons for choice of the octahedral shearing stress criterion as the failure criterion of plain concrete are:
1) its simple mathematical form and invariant character,
2) experimental data seem to support this criterion at least as well as any other simple criterion. The criterion is taken in the form

$$\tau_{oct} = a - bp \tag{1}$$

where

$$\tau_{oct} = (1/3)[(\sigma_{11} - \sigma_{22})^2 + (\sigma_{22} - \sigma_{33})^2 + (\sigma_{33} - \sigma_{11})^2 + 6(\sigma_{12}^2 + \sigma_{23}^2 + \sigma_{31}^2)]^{1/2}$$

$$p = (1/3)(\sigma_{11} + \sigma_{22} + \sigma_{33})$$

In a biaxial state of stress where $\sigma_{33} = \sigma_{13} = \sigma_{23} \equiv 0$

$$\tau_{oct} = (\sqrt{2}/3)(\sigma_{11}^2 - \sigma_{11}\sigma_{22} + \sigma_{22}^2 + 3\sigma_{12}^2)^{1/2}$$

$$p = (1/3)(\sigma_{11} + \sigma_{22})$$

To acquire a good match with experimental results available for a plane state of stress, two expressions of form (1) are needed, one valid in biaxial compression

$$\tau_{oct} + [\sqrt{2}(\beta - 1)/(2\beta - 1)]p - [\sqrt{2}\beta/3(2\beta - 1)]\, f_c = 0 \quad (2a)$$

$(\sigma_1 < 0$ and $\sigma_2 < 0)$

and the other in states of compression - tension or biaxial tension

$$\tau_{oct} + [\sqrt{2}(1 - \alpha)/(1 + \alpha)]p - [2\sqrt{2}\alpha/3(1 + \alpha)]\, f_c = 0 \quad (2b)$$

$(\sigma_1 < 0$ or $\sigma_2 < 0)$

Here, the following constants have been introduced

$-f_c$ uniaxial compressive strength $(f_c > 0)$

$f_t = \alpha f_c$ uniaxial tensile strength, $\alpha \approx 0.1$

$-\beta f_c$ biaxial compressive strength in state $\sigma_1 = \sigma_2$, $\beta \approx 1.16$

Expressions (2a) and (2b) can be interpreted in 3-dimensional $(\sigma_1, \sigma_2, \sigma_3)$ space as coaxial cones intersecting along the plane $\sigma_1 + \sigma_2 + \sigma_3 = -f_c$. The intersection along the (σ_1, σ_2) plane is presented in Fig. 1. In a uniaxial state of stress, the behaviour of concrete is elastic, perfectly plastic in compression, and elastic - brittle in tension (Fig. 2).

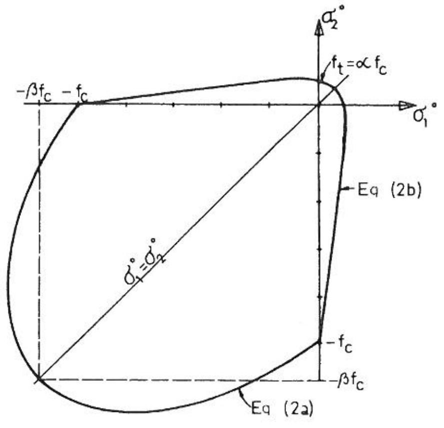


Fig. 1. Yield and cracking criterion of concrete in biazialstate of stress

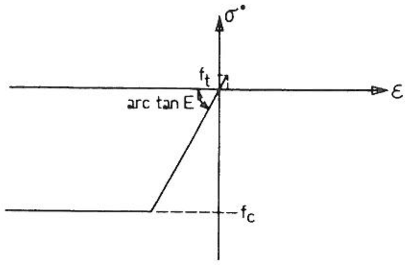


Fig. 2. Stress-strain relationship of concrete

2.2 Reinforcement

For steel bars the uniaxial elastic, perfectly plastic idealization is used

$$\sigma = \begin{cases} E_{st}\,\varepsilon & |\varepsilon| < \sigma_p/E_{st} \quad \text{or} \quad |\varepsilon| \geqslant \sigma_p/E_{st} \quad \text{and} \quad \sigma d\sigma < 0 \\ \sigma_p \,\text{sign}\,\varepsilon, & |\varepsilon| \geqslant \sigma_p/E_{st} \quad \text{and} \quad \sigma d\sigma \geqslant 0 \end{cases}$$

2.3 Reinforced concrete

In the construction of a model for description of the mechanical behaviour of reinforced concrete, the following assumptions are made

1) the deformation is uniform in an uncracked state, i.e. concrete and reinforcement have the same strains, and full bond is maintained
2) cracking occurs when the cracking criterion (2b) is satisfied; after cracking, concrete has no tensile strength in the direction perpendicular to the crack. In the crack direction, concrete behaves uniaxially. It is assumed that no bond slip occurs, even over the finite region of the material which corresponds to the region of cracking
3) yielding of concrete occurs when the yield criterion (2a) is satisfied. The yielding of concrete is not affected by reinforcement.

2.3.1 Initial elastic behaviour

Consideration is given to a concrete plate of thickness t. This plate contains two systems of reinforcing steel denc and R_2, with directions that make angles ϕ_1 and ϕ_2 with respect to the x_1-axis. On denoting the steel area per unit of width of section as A, the relative amounts of reinforcing steel are $\mu_1 = A_1/t$ and $\mu_2 = A_2/t$. The relative amount of concrete is then $\mu_o = 1 - \mu_1 - \mu_2$. The stresses in various constituents of the plate are denoted by $\sigma^0_{\alpha\beta}$, $\sigma^1_{\alpha\beta}$, and $\sigma^2_{\alpha\beta}$, respectively. Thus the average stress or pseudostress through the thickness of the plate is

$$\{\sigma\} = \mu_o\{\sigma^0\} + \mu_1\{\sigma^1\} + \mu_2\{\sigma^2\} \tag{4}$$

The assumed uniform strain in both concrete and reinforcement gives a basis for relation of the pseudostress to the strains by formula

$$\{\sigma\} = [C]\{\varepsilon\} \tag{5}$$

where the elasticity matrix is

$$[C] = \mu_o[C^0] + \mu_1[C^1] + \mu_2[C^2] \tag{6}$$

and the stress and strain vectors are

$$\{\sigma\}^T = \{\sigma_{11}\sigma_{22}\sigma_{12}\}, \qquad \{\varepsilon\}^T = \{\varepsilon_{11}\varepsilon_{22}\gamma_{12}\} \tag{7}$$

The material property matrices of the individual constituents are

$$[C^0] = \frac{E}{1-\nu^2}\begin{bmatrix} 1 & \nu & 0 \\ \nu & 1 & 0 \\ 0 & 0 & (1-\nu)/2 \end{bmatrix} \tag{8}$$

$$[C^i] = [\bar{T}]_i \begin{bmatrix} E_{st} & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 0 \end{bmatrix} [\bar{T}]_i^T \qquad i = 1,2 \tag{9}$$

$[\bar{T}]_i$ denotes the transformation matrix

$$[\bar{T}] = \begin{bmatrix} \cos^2\phi & \sin^2\phi & -2\cos\phi\sin\phi \\ \sin^2\phi & \cos^2\phi & 2\cos\phi\sin\phi \\ \cos\phi\sin\phi & -\cos\phi\sin\phi & \cos^2\phi-\sin^2\phi \end{bmatrix} \tag{10}$$

evaluated at values $\phi = \phi_1$ and $\phi = \phi_2$, respectively.

Equations (4)...(10) establish a model for the anisotropic elastic behaviour of reinforced concrete.

2.3.2 Cracked concrete

Cracking will occur if one or both of the principal stresses of concrete σ_1^o and σ_2^o are positive, and the failure criterion (2b) is satisfied. The direction of the crack is taken to be perpendicular to the direction of maximum tensile stress. After cracking, concrete behaves uniaxially in the crack direction, of which the angle with respect to the x_1-axis is denoted by ϕ_c. Hence, the elasticity matrix of concrete after cracking is

$$[C^o]_c = [\bar{T}]_c \begin{bmatrix} E & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 0 \end{bmatrix} [\bar{T}]_c^T \qquad (11)$$

where $[\bar{T}]_c$ is the transformation matrix (10) evaluated at $\phi = \phi_c$. The elastic behaviour after cracking is determined by equation (6), in which $[C^o]_c$ is now substituted for the isotropic properties of equation (8).

It is possible that concrete could be stressed into a second cracking system. This occurs if the concrete stress in the ϕ_c-direction attains the limiting value $f_t = \alpha f_c$ i.e.

$$\sigma_c^o = \sigma_{11}^o \cos^2\phi_c + \sigma_{22}^o \sin^2\phi_c + 2\sigma_{12}^o \cos\phi_c \sin\phi_c = \alpha f_c \qquad (12)$$

After the formation of a second crack, only the reinforcement is effective; in this case, the elasticity matrix is given by equation

(6), by omitting the part of concrete $\mu_o[C^o]$. In the case of orthogonal reinforcement, it is observable that no shearing resistance is left in the material.

In regard to the plastic behaviour of cracked concrete, the following possibilities exist:

1) Concrete yields in uniaxial compression, which will occur if

$$-\sigma_c^o - f_c = -(\sigma_{11}^o \cos^2\phi_c + \sigma_{22}^o \sin^2\phi_c + 2\sigma_{12}^o \cos\phi_c \sin\phi_c) - f_c$$

and (13)

$$-(d\varepsilon_{11} \cos^2\phi_c + d\varepsilon_{22} \sin^2\phi_c + d\gamma_{12} \cos\phi_c \sin\phi_c) \geqslant 0$$

(loading criterion)

2) Reinforcement 1 or 2 yields in tension or compression, which will occur if

$$\pm\sigma^i - \sigma_p = \pm(\sigma_{11}^i \cos^2\phi_i + \sigma_{22}^i \sin^2\phi_i + 2\sigma_{12}^i \cos\phi_i \sin\phi_i) -$$
$$- \sigma_p = 0, \quad i = 1,2$$

and (14)

$$\pm(d\varepsilon_{11} \cos^2\phi_i + d\varepsilon_{22} \sin^2\phi_i + d\gamma_{12} \cos\phi_i \sin\phi_i) \geqslant 0$$

(loading criterion)

The upper sign corresponds to the yield in tension, the lower sign to the yield in compression.

The contribution of a yielding reinforcement to the state of stress is determined by equations

$$\{\sigma^i\}^T = \pm\sigma_p\{\cos^2\phi_i \ \sin^2\phi_i \ \cos\phi_i \sin\phi_i\}, \quad \{d\sigma^i\} = \{0\}, \ i = 1.2$$

(15)

The corresponding relationship for concrete is formed in an analogous manner.

2.3.3 Plastic behaviour in biaxial compression

It is assumed that concrete yields if the yield criterion (2a) is satisfied, as well as the loading criterion

$$\{\frac{\partial f}{\partial\{\sigma\}}\}^T \{d\sigma\} = 0 \quad \text{with } f = \tau_{oct} + \sqrt{2}\,(\beta - 1)p/(2\beta - 1)$$

The stress and strain increments are related by the normality rule of the theory of plasticity. The 3-dimensional relationship can be presented in the form (cf. [1])

$$\{d\sigma\} = [A]\,\{d\varepsilon\} \qquad (16)$$

where

$$\{d\sigma\}^T = \{d\sigma_{11}\ d\sigma_{22}\ d\sigma_{33}\ d\sigma_{12}\ d\sigma_{23}\ d\sigma_{31}\}$$

$$\{d\varepsilon\}^T = \{d\varepsilon_{11}\ d\varepsilon_{22}\ d\varepsilon_{33}\ d\gamma_{12}\ d\gamma_{23}\ d\gamma_{31}\}$$

and

$$[A] = [C] - \frac{[C]\,\{\frac{\partial f}{\partial\{\sigma\}}\}\,\{\frac{\partial f}{\partial\{\sigma\}}\}^T[C]}{\{\frac{\partial f}{\partial\{\sigma\}}\}^T[C]\,\{\frac{\partial f}{\partial\{\sigma\}}\}}$$

[C] represents Hooke's law for isotropic material. From equation (16), the form suitable for plane state of stress

$$\{d\sigma^o\} = [A^*]\{d\varepsilon\} \qquad (17)$$

is obtainable if it is noticed that in this case $d\sigma_{33} = d\gamma_{13} = d\gamma_{23} \equiv 0$. Thus, the elements of the matrix $[A^*]$ are

$$A^*_{ik} = A_{ik} - \frac{A_{i3}A_{3k}}{A_{33}} \qquad i,k = 1,2,4 \qquad (18)$$

The loading criterion can be manipulated into the form

$$\{\frac{\partial f}{\partial\{\sigma\}}\}^T\,[C^o]\,\{d\varepsilon\} \begin{cases} \geqslant 0 & \text{loading} \\ < 0 & \text{unloading} \end{cases} \qquad (19)$$

Here,

$$\{\frac{\partial f}{\partial\{\sigma\}}\}^T = (1/3)\{\frac{\sigma^o_{11} - p^o}{\tau^o_{oct}} + \frac{\sqrt{2}(\beta - 1)}{2\beta - 1} \quad \frac{\sigma^o_{22} - p^o}{\tau^o_{oct}} + \frac{\sqrt{2}(\beta - 1)}{2\beta - 1} \quad \frac{2\sigma^o_{12}}{\tau^o_{oct}}\} \quad (20)$$

In actual computation, finite increments of stress and strain are taken into account. Consequently, it is necessary to return the stress vector on to the yield surface. This is achievable by formula

$$\{\sigma^o\}_{corrected} = \{\sigma^o\}/[\tau^o_{oct} + \sqrt{2}(\beta - 1)p^o/(2\beta - 1)] \qquad (20)$$

2.3.4 Unloading

Two different cases of unloading can occur:

1) unloading after plastic yielding. In this case, the elastic relations are applicable.

2) closing of cracks. This will occur if compressive stresses tend to develop perpendicularly to the crack direction.

Consequently, if cracking occurs in one direction only, the crack will close if

$$\varepsilon_{\perp} + \nu\varepsilon_{=} \leqslant 0 \qquad (21)$$

where $\varepsilon_{\perp}$ denotes the strain perpendicular, and $\varepsilon_{=}$ the strain parallel to the crack. If cracks exist in two (perpendicular) directions, then one crack will close if

$$\varepsilon_{\perp} \leqslant 0 \qquad (22)$$

After a crack has closed the behaviour of concrete is similar to that before the opening of that particular crack. Of course, the tensile strength perpendicular to a crack is lost; this should be observed in the case of this crack reopening.

3. THE FINITE ELEMENT SOLUTION

In the elastic range, the finite element procedure follows the common pattern [9], so that repetition of the necessary steps can be omitted. It is adequate to mention that triangular constant strain elements were used in the solution of a plate in a planar state of stress, the particular problem studied here.

The problem becomes highly nonlinear when the stresses reach the cracking or/and yielding stage. The solution is achievable by employment of the "initial stress" method described in [10]. In this method, the part of a calculated stress increment that exceeds the actual stress increment corresponding to the current state of the material properties is converted into a pseudo-load located at the pertinent node. Pseudo-loads give rise to new strain and stress increments, and the iteration can be continued until the desired level of accuracy has been attained.

In the case of cracking, a pseudo-load vector for a cracked element

$$\{\Delta P_c\}_e = t\, A_e [B]_e^T \{\sigma_c\}_e \qquad \text{(for CST-element)} \tag{23}$$

is evaluated, in which

$$\{\sigma_c\}_e^T = \mu_o \sigma_1^o \{\sin^2\phi_c \;\; \cos^2\phi_c \;\; -\cos\phi_c \sin\phi_c\} \tag{24}$$

is the stress released in cracking. σ_1^o is the maximum tensile stress of concrete, and angle ϕ_c defines the crack direction. For cracked elements, the elasticity matrix (8) is replaced by that of cracked concrete (11). In the case of a second crack, the concrete stiffness is zero.

In plastic yielding, the pseudo-load vector is

$$\{\Delta P_p\}_e = t\, A_e [B]_e^T \{\Delta\sigma_p\}_e \tag{25}$$

where

$$\{\Delta\sigma_p\}_e = \mu_o \{\Delta\sigma_p\}_e + \mu_1 \{\Delta\sigma_p^1\}_e + \mu_2 \{\Delta\sigma_p^2\}_e \tag{26}$$

is the difference of elastic and actual stress increments

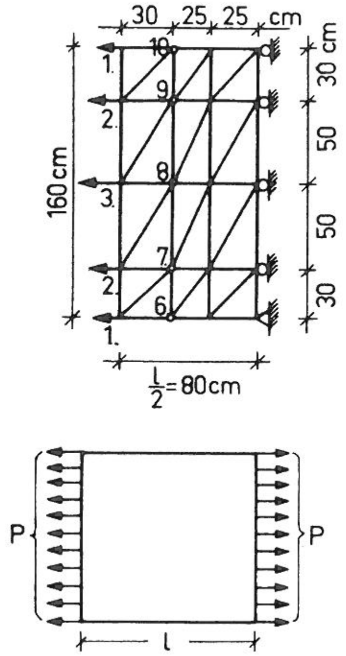
$$\{\Delta\sigma_p^i\}_e = \{\Delta\sigma_e^i\}_e - \{\Delta\sigma^i\}_e \qquad i = 0,1,2$$

The elastic stress increments are given trough elastic stress-strain relations, and the actual stress increments by equation (17) (concrete).

4. APPLICATIONS

Peter [7] has reported on an extensive series of tests on panels under uniform tension, and with different directions of reinforcement. The method described above was applied to analysis of some of the panels, and by use of the data given in Peter's thesis. The finite element arrangement is illustrated in Fig. 3. The data given by Peter, and the cracking and yield loads obtained by finite element calculation, are listed in Table 1. The calculated cracking loads are considerably higher than the values observed. This discrepancy is attributable to a number of reasons, including neglect of the bond slip, nonhomogeneity, the test arrangements, and so on which can not be accounted for by the idealized model. Agreement is better between the calculated yield loads and the observed failure loads. The excess of observed over calculated loads probably arises from bond and strain hardening of steel bars. The load-displacement relationships are indicated in Fig. 4. The abrupt changes of stiffness are visible in the calculated relationships, while the observed curves are smoother.

The second application is a cantilever panel, in accordance with Fig. 5. This case was studied by Cervenka [3], both theoretically by means of the finite element method, and experimentally. The



Panel 160 × 160 × 8 cm^3

total load P

1. $= \frac{15}{160} P$

2. $= \frac{40}{160} P$

3. $= \frac{50}{160} P$

Concrete:

$E = 250000\ kp/cm^2$

$\nu = 0.14$

Reinforcement:

$\mu_1 = \mu_2 = 0.00666$

$E_{st} = 2080000\ kp/cm^2$

$\sigma_p = 4220\ kp/cm^2$

Fig. 3. Panel in uniform tension.

Table 1. Panel in uniform tension.

Data [7]				FEM-calculation		
sign	concrete tensile strength [kp/cm²]	cracking load [Mp]	failure load [Mp]	cracking load [Mp]	crack direction	yield load [Mp]
S 2r 0	18.5	19.0	40.0	24.64	90.0	36.0-36.8
S 2r 10	23.4	19.5	35.1	31.07	89.4	34.4-35.2
S 2r 20	23.1	16.6	39.4	30.44	89.1	31.2-32.0
S 2r 30	20.9	19.6	38.8	27.31	89.2	28.8-29.6
S 2r 40	27.6	20.0	40.8	35.85	-	< 35.8

data were chosen to correspond to Cervenka's test specimen. The load-displacement relationship has been plotted in Fig. 6, which further contains re-drawing of the curves obtained by Cervenka. Another panel, which was considered to correspond more closely to the test arrange-

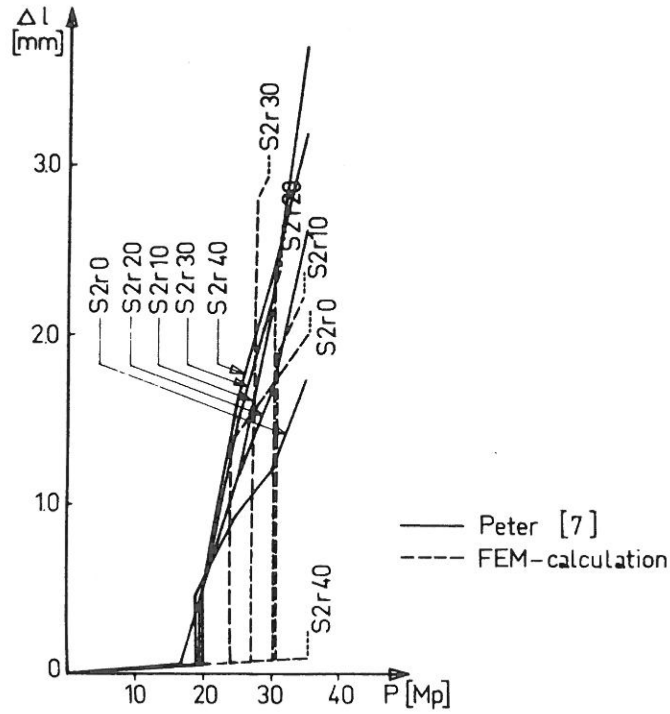
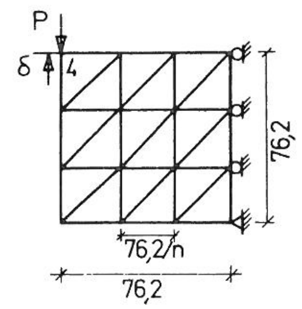


Fig. 4. Load-displacement relationships for panels in uniform tension.



Panel: 76,2 × 76,2 × 5,08 cm

Concrete:
$E = 203000$ kp/cm^2
$\nu = 0{,}14$
$f_c = 280$ kp/cm^2
$\alpha = 0{,}075$
$\beta = 1{,}2$

Reinforcement:
$\mu_1 = \mu_2 = 0{,}01$
$\phi_1 = 0°$, $\phi_2 = 90°$
$E_{st} = 2100\,000$ kp/cm^2
$\sigma_p = 2800$ kp/cm^2

Fig. 5. Cantilever panel.

ment, was also studied (Fig. 7). The load-displacement relationships for this example, together with Cervenka's experimental curve, are illustrated in Fig. 8. The agreement is qualitatively good, though for reasons of convergence, the finite element calculation could not be continued as far as the test.

5. CONCLUSION

The results obtained above lead to the conclusion that the model used in calculations is capable of repreduction, at least qualitatively, of some features of the behaviour of reinforced concrete. It is to be hoped that improvement of the model, especially with respect to its shear stiffness after cracking, can be made. Attempts are currently being made to use more refined finite elements, and to analyse more complex structures, such as laterally loaded plates and shells.

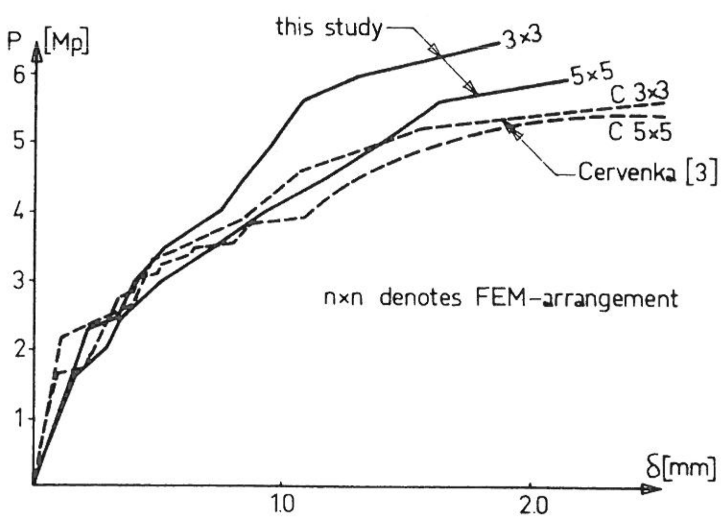
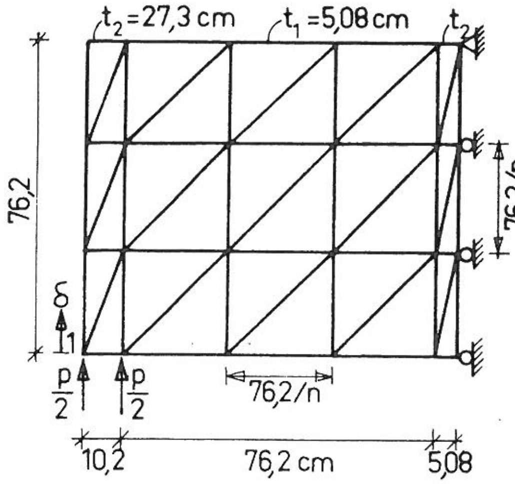


Fig. 6. Load-displacement relationships for cantilever panel.



Concrete:

$E = 203000\ kp/cm^2$

$\nu = 0{,}14$

$f_c = 237\ kp/cm^2$

$f_t = \alpha f_c = 0{,}1426 \cdot 237 = 33{,}8\ kp/cm^2$

$\beta = 1{,}2$

Reinforcement:

$\mu_1 = 0{,}00785,\ \phi_1 = 0^\circ$

$\mu_2 = 0,\ \phi_2 = 90^\circ$

$E_{st} = 1673000\ kp/cm^2$

$\sigma_p = 4690\ kp/cm^2$

Fig. 7. Cantilever panel.

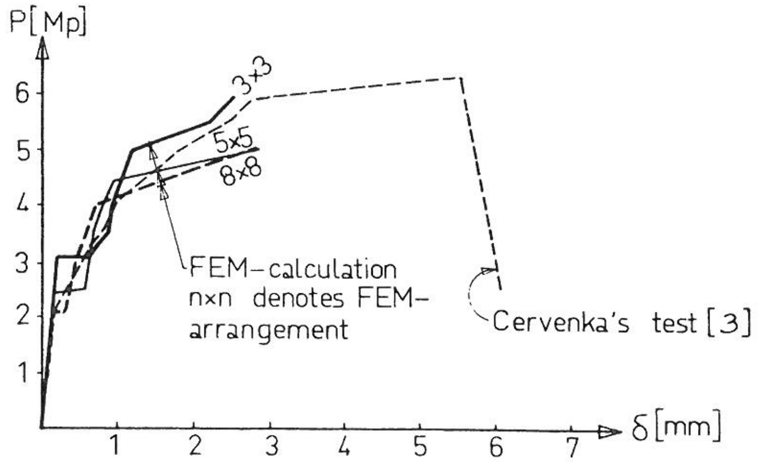


Fig. 8. Load-displacement relationship for cantilever panel.

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