

NUMERICAL ANALYSIS  
ORIENTED BIAXIAL STRESS-STRAIN RELATION AND  
FAILURE CRITERION OF PLAIN CONCRETE

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### SUMMARY

For a numerical analysis, e.g. by Finite Differences or Finite Element Method, there is still a lack of knowledge on the level of incorporating a material behaviour law for concrete beyond the linear elastic range.

In the presented paper, a biaxial stress-strain relation and failure criterion is proposed, which is applicable to the above mentioned structural analysis methods.

The formulation of material behaviour of plain concrete in biaxial stress-state was developed on the basis of test results, given by Kupfer/Hilsdorf/Rüsch. A nonlinear elastic, anisotropic stress-strain relation was derived with two moduli of elasticity,  $E_1$ ,  $E_2$ , and Poisson's ratios,  $\nu_1$ ,  $\nu_2$ , which depend on the prevailing biaxial stress state. The stress-strain relation is valid in the whole biaxial stress field, that means with a smooth transition between the domains of tension/tension, tension/compression and compression/compression. The stress-dependent moduli  $E_1$ ,  $E_2$  and the Poisson's ratios  $\nu_1$ ,  $\nu_2$  are approximated by polynomials, trigonometrical and exponential functions. They were established not only as secant-, but also as tangent-functions for arbitrary use in the numerical procedure.

A failure criterion was defined by approximating the test results of the biaxial ultimate concrete strength with a 7th degree polynomial, which is also valid in the whole biaxial stress domain. The definition of the state of failure is given as a function of stresses as well as strains.

Initial parameters of the formulation of the biaxial material behaviour are the uniaxial cylindrical strength of concrete and the initial values of Young's modulus and Poisson's ratio. A simple expansion of this formulation makes it applicable not only to normal but also to lightweight concrete.

Comparison of numerically calculated stress-strain curves up to the ultimate biaxial stresses which indicate the failure criteria with those obtained from tests show a very good agreement. It is shown, that the biaxial stress-strain relation can be extended for use in cases of triaxial tension/tension/compression stress state. Numerical examples of analysis of concrete slabs show the importance of incorporation of a realistic material behaviour for better safety estimations.

### 1. Introduction

In the numerical analysis of reinforced concrete structures mainly two ways of implementing a material behaviour law are known, both on the phenomenological level of material characterization. One is based on an integrated description of the material behaviour of both concrete and reinforcement, the other considers the respective material characteristics of concrete and reinforced steel bars separately. This paper refers to the latter way and is restricted to the material behaviour of plain concrete only.

In the last few years numerous papers had been published on this subject, some of which are cited later. Up to now most of the proposed failure criteria for biaxially loaded concrete consist in a set of functions, each valid only in parts of the biaxial stress-strain domain. The load dependence was always chosen in terms of stress only. Except for the quadrilinear stress-strain relation of Romstad et al. [1], in all other formulations of biaxial deformation behaviour the differential representation of the non-linear stress-strain relations is preferred, being a more adequate way to fit test data and to take account of the incremental numerical procedure.

Berg [2] used a modified representation of Kupfer's [3] test data and calculated the tangent values of the elasticity matrix by interpolation, in dependence of the prevailing stress- and strain-increments. Nelissen [4] and Wegner [5] developed a strain-stress, respectively stress-strain relation, dependent on the current state of stresses, respectively strains, and several further parameters or functions. Stress-strain relations in octahedral form were proposed by Girijavallabhan/Metha [6], Khan et al. [7], Grünberg [8] and Kupfer/Gerstle [9]. They all presented tangent values of the shear modulus, while Kupfer/Gerstle [9] derived tangent as well as secant values of both shear and bulk moduli. It was stated by most of the above mentioned authors, that it is sufficient to introduce a stress or strain dependent shear modulus and to regard the bulk modulus as a constant value. The above cited formulations of deformation behaviour are isotropic ones, while the stress-strain behaviour of biaxially loaded concrete indicates stress- or strain-induced anisotropy beyond the linear elastic range. The anisotropy results from the nonlinear stress-strain response in the two directions considered and leads in general to a non-symmetrical elasticity matrix containing the appropriate material properties. Liu et al. [10] and Darwin [11] suggested anisotropic stress-strain relations, and employed special assumptions to obtain a symmetrical elasticity matrix.

The herein proposed formulation of biaxial deformation behaviour provides a stress-induced anisotropical presentation too. Symmetry of the material matrix is achieved by assumption f., listed below. Main assumptions incorporated in the material law in order to obtain characteristic values on the basis of test results and to describe the biaxial, nonlinear stress-strain behaviour of concrete are as follows:

- a. plane stress state ( $\sigma_{33} = 0$ ); (extension to  $\sigma_{33} \neq 0$  see chapter 4)
- b. restriction to plain concrete, reinforcement regarded separately
- c. homogeneous, quasi-isotropic material, with stress-induced anisotropy
- d. short-term loading
- e. hyperelasticity
- f. existence of an elastic potential for each stress- and strain-increment.

The dependence of all material characteristic values on biaxial loading, incorporated in the developed formulations, relies on the test results obtained by Kupfer/Hilsdorf/Rüsch [3], [9] and Stegbauer/Linse [12], with tests performed on concrete and lightweight concrete specimens.

2. Stress-strain relations for concrete in biaxial loading

Considering the above listed assumptions, it is possible to reduce the yet unknown material characteristic values in the elasticity matrix to a total of four. These four values may be interpreted as two moduli of elasticity,  $E_1$  and  $E_2$ , the index indicating principal directions of biaxial load action, and two corresponding Poisson's ratios,  $\nu_1$  and  $\nu_2$ . In order to satisfy assumption f., eq. (1) is obtained, which shows that now only three material

$$\frac{\nu_1}{E_1} = \frac{\nu_2}{E_2} \tag{1}$$

characteristic values, dependent on the current stress-strain state, are necessary at all. Eq. (2) shows the elasticity matrix valid for principal directions 1 and 2 and for tangent values of the material parameters being

$$\Delta \sigma^{(1,2)} = D^{(1,2)} \cdot \Delta \epsilon^{(1,2)} \tag{2}$$

$$D^{(1,2)} = \frac{1}{1-\nu_1 \nu_2} \begin{bmatrix} E_1 & \nu_1 E_2 & 0 \\ \nu_2 E_1 & E_2 & 0 \\ 0 & 0 & \frac{E_1(1-\nu_1 \nu_2)}{1+2\nu_1+\nu_1/\nu_2} \end{bmatrix} = \frac{E_1}{1-\nu_1 \nu_2} \begin{bmatrix} 1 & \nu_2 & 0 \\ \nu_2 & \frac{\nu_2}{\nu_1} & 0 \\ 0 & 0 & \frac{1-\nu_1 \nu_2}{1+2\nu_1+\nu_1/\nu_2} \end{bmatrix}$$

constant inside each increment. Incremental stress-strain and strain-stress relations for arbitrary directions x and y, x inclined with an angle  $\alpha$  to the 1-axis, are obtained with eqs. (3) and (4) by simple transformation using the abbreviations indicated above.

Eqs. (2) to (4) hold for secant values of the material characteristic parameters too, if the total stresses and strains and not only their increments  $\Delta$ , are inserted and if the secant values  $E_1^S$  and  $\nu_1^S$ ,  $i = 1, 2$ , are used instead of the tangent values  $E_i$  and  $\nu_i$ .

$$\begin{aligned}
 s &= \sin \alpha & s_2 &= \sin^2 \alpha & s_3 &= \sin^3 \alpha \\
 c &= \cos \alpha & c_2 &= \cos^2 \alpha & c_3 &= \cos^3 \alpha \\
 A_1 &= 1 + 2\nu_1 + \nu_1/\nu_2 & A_3 &= -1 + \nu_1 - \nu_2 + \nu_1/\nu_2 \\
 A_2 &= 2 - \nu_1/\nu_2 - \nu_2/\nu_1 & A_4 &= 1 + \nu_1 - \nu_2 - \nu_2/\nu_1
 \end{aligned}$$

$$\Delta \sigma^{(x,y)} = \mathbf{D}^{(x,y)} \cdot \Delta \epsilon^{(x,y)} \tag{3}$$

$$\begin{bmatrix} \Delta \sigma_{xx} \\ \Delta \sigma_{yy} \\ \Delta \sigma_{xy} \end{bmatrix} = \frac{E_1 / A_1}{1 - \nu_1 \nu_2} \begin{bmatrix} A_1 \left( \frac{\nu_2}{\nu_1} s_2 + c_2 \right) + \nu_2 A_1 - A_2 s_2 c_2 & A_3 s^3 c_3 + A_4 s_3 c \\ \text{symm.} & A_1 \left( s_2 + \frac{\nu_2}{\nu_1} c_2 \right) + A_2 s_2 c_2 & A_3 s_3 c + A_4 s c_3 \\ & & (1 - \nu_1 \nu_2) - A_2 s_2 c_2 \end{bmatrix} \begin{bmatrix} \Delta \epsilon_{xx} \\ \Delta \epsilon_{yy} \\ 2\Delta \epsilon_{xy} \end{bmatrix}$$

$$\Delta \epsilon^{(x,y)} = [ \mathbf{D}^{(x,y)} ]^{-1} \cdot \Delta \sigma^{(x,y)} \tag{4}$$

$$\begin{bmatrix} \Delta \epsilon_{xx} \\ \Delta \epsilon_{yy} \\ 2\Delta \epsilon_{xy} \end{bmatrix} = \frac{1}{E_1} \begin{bmatrix} \frac{\nu_1}{\nu_2} s_2 + c_2 & -\nu_1 & \left(1 - \frac{\nu_1}{\nu_2}\right) s c \\ \text{symm.} & s_2 + \frac{\nu_1}{\nu_2} c_2 & \left(1 - \frac{\nu_1}{\nu_2}\right) s c \\ & & 1 + 2\nu_1 + \frac{\nu_1}{\nu_2} \end{bmatrix} \begin{bmatrix} \Delta \sigma_{xx} \\ \Delta \sigma_{yy} \\ \Delta \sigma_{xy} \end{bmatrix}$$

$$\Delta \epsilon_{zz} = \frac{1}{E_1} [ -\nu_1 \quad -\nu_1 \quad 0 ] \cdot \Delta \sigma^{(x,y)}$$

By close-fitting approximation of Kupfer's [3], [9] parameter representation of the secant values  $E_1^S$  and  $\nu_1^S$ , dependent on the two principal stresses,  $\sigma_{11}$  and  $\sigma_{22}$ , related to the uniaxial cylindrical strength of concrete  $f_c$ , the functions  $E_2^S$  ( $\hat{=} FEM2$ ),  $\nu_1^S$  ( $\hat{=} FQU1$ ) and  $\nu_2^S$  ( $\hat{=} FQU2$ ) were developed (see table I, a - c). The numerical procedure may be more efficient, when secant as well as tangent values of the material characteristic functions are available in the computer program. The use of tangent values is more convenient during incremental loading, but secant values are advantageous when partial unloading effects, e.g. cracking of concrete under tension stresses, have to be considered. A parameter representation of the tangent values was derived similar to Kupfer's secant one. The functions of the tangent values  $E_1$  ( $\hat{=} FET1$ ),  $E_2$  ( $\hat{=} FET2$ ) and  $\nu_1$  ( $\hat{=} FQT1$ ), resulting of approximation by least squares, are presented in table I, d - f.

The respective fourth material characteristic value, missing in table I, can be obtained by application of eq.(1).

The whole formulation of the nonlinear stress-strain behaviour is presented in dependence of the biaxial related to  $f_c$  principal stresses in form of FORTRAN - function subroutines.

Figs. 1 and 2 point out, that the functions are valid in the entire biaxial stress domain and show a smooth transition between the compression/compression, compression/tension and tension/tension zones. They are limited by the failure criteria described in chapter 3.

The proposed stress-strain relation presented in table I is applicable with good agreement to test results of normal concrete of various strengths when the proper initial parameters of Young's modulus,  $E_0$  ( $\hat{=}$  EMO), and Poisson's ratio,  $\nu_0$  ( $\hat{=}$  QUO) are inserted. Fig. 3 shows the good agreement of calculated stress-strain curves with those obtained in the experiment. Eq. (5) indicates the extension of the stress-strain relation obtained of normal concrete

$$\begin{aligned}
 E_i^{S,LC} &= E_0^{LC} \left[ 0.7 + 0.3 \left( \frac{E_i^{S,NC}}{E_0} \right) \right] \\
 \nu_i^{S,LC} &= \nu_0^{LC} \left[ 0.7 + 0.3 \left( \frac{\nu_i^{S,NC}}{\nu_0} \right) \right] \\
 E_i^{LC} &= E_0^{LC} \left[ 0.6 + 0.4 \left( \frac{E_i^{NC}}{E_0} \right) \right] \\
 \nu_i^{LC} &= \nu_0^{LC} \left( \frac{\nu_i^{NC}}{\nu_0} \right)
 \end{aligned} \tag{5}$$

with  $i=1,2$  and material values in round brackets according to table I

(NC) to lightweight concrete (LC). The extension was performed by simple curve-fitting of the above outlined dependence of the material characteristic values on biaxial loading related to those reported in the test results of Stegbauer/Linse [12]. Calculated stress-strain curves for lightweight concrete (Fig. 4) show good correspondence with the test results.

### 3. Failure criteria for concrete in biaxial loading

Since the failure limit is symmetrical to the equisectrix ( $\sigma_{11} = \sigma_{22}$ , or  $\epsilon_{11} = \epsilon_{22}$ ), the formulation of a failure criterion may be restricted to one half of the biaxial stress (or strain) domain. This is realized by approximating the test results of ultimate concrete strength in a X-Y, respectively U-V coordinate system inclined to the  $\sigma_{11}/f_c$ -, respectively  $\epsilon_{11}/\epsilon_c$ -, U-axis with an angle of  $45^\circ$ . Using polynomials, as stated in eqs. (6) and (7), the coefficients compiled in table II are obtained. They refer to the three

$$Y = \sum_{n=0}^{n=7} A_n X^n + A_8 \left( 0.594 \sin \frac{\pi X}{1.7} - 0.00192 \tan \frac{\pi X}{3.3} \right) \tag{6}$$

$$V = \sum_{n=0}^{n=6} B_n U^n \tag{7}$$

different concrete strengths used in Kupfer's tests [3], [9], to the lightweight concrete investigated by Stegbauer/Linse [12] and to an average failure curve, valid for all strengths of normal concrete. Failure criteria for normal and lightweight concretes, according to eq. (6) and dependent on biaxial principal stresses related to the uniaxial strength, are compared with several test curves in Fig. 5. The related to  $f_c$  stress dependent failure curves of different concrete strength, e.g. Kupfer's test results included in Fig. 5, only differ scarcely. On the other hand strong dependence on concrete strength is obvious through Fig. 6 by the failure criteria as functions of strains, related to the uniaxial ultimate strain, received from eq. (7). This evident dependence on concrete strength gave rise to the stress-dependent formulation of the material characteristic values outlined in chapter 2. It is the formulation of biaxial material behaviour in terms of stresses that allows for this advantageous, uniformly defined material law to be applied for all concrete strengths in the numerical procedure.

4. Extension of the stress-strain relation proposed in chapter 2 to special triaxial stress states

The stress-strain behaviour of concrete, subjected to pure tension load, is almost linear. The material characteristic functions, given in table I, confirm this linear elastic behaviour in the biaxial tension/tension domain yielding material characteristic values nearby the initial parameters  $E_0$  and  $\nu_0$ . An extension of the proposed biaxial stress-strain relation to a triaxial one, may therefore be easily obtained in the compr./compr./tens., compr./tens./tens. and tens./tens./tension domain. Since stresses and strains are algebraically arranged the material characteristic functions according to table I are related to the principal directions 1 and 2. The pertinent principal direction 3 remains in tension loading throughout all the above mentioned special triaxial stress states. Therefore it becomes obvious to apply the initial parameters  $E_0$  and  $\nu_0$  to principal direction 3 upon derivation of a special triaxial stress-strain relation. The obtained strain-

$$\Delta \epsilon^{(1,3)} = [ D^{(1,3)} ]^{-1} \Delta \sigma^{(1,3)} \quad (8)$$

$$[ D^{(1,3)} ]^{-1} = \frac{1}{E_1} \begin{bmatrix} 1 & -\nu_1 & -\nu_1 & 0 & 0 & 0 \\ & \nu_1/\nu_2 & -\nu_1 & 0 & 0 & 0 \\ & & E_1/E_0 & 0 & 0 & 0 \\ & & & 1+2\nu_1+\frac{\nu_1}{\nu_2} & 0 & 0 \\ \text{symm.} & & & & 2(1+\nu_0)\frac{E_1}{E_0} & 0 \\ & & & & & 2(1+\nu_0)\frac{E_1}{E_0} \end{bmatrix}$$

stress relation, valid for principal directions and applicable to the above defined special triaxial domain, is given in eq. (8). Matrix-inversion of eq. (8) yields the elasticity matrix stated in eq. (9). Comparison of

$$\Delta \sigma^{(1,3)} = \mathbf{D}^{(1,3)} \cdot \Delta \epsilon^{(1,3)} \quad (9)$$

$$\mathbf{D}^{(1,3)} = \frac{E_1}{A_5} \begin{bmatrix} \frac{\nu_1 E_1}{\nu_2 E_0} - \nu_1^2 & \nu_1 \frac{E_1}{E_0} + \nu_1^2 & \nu_1 \frac{\nu_1 + \nu_2^2}{\nu_2} & 0 & 0 & 0 \\ & \frac{E_1}{E_0} - \nu_1^2 & \nu_1 (1 + \nu_1) & 0 & 0 & 0 \\ & & \frac{\nu_1}{\nu_2} - \nu_1^2 & 0 & 0 & 0 \\ \text{symm.} & & & \frac{A_5}{1 + 2\nu_1 + \nu_1/\nu_2} & 0 & 0 \\ & & & & \frac{A_5 E_0/E_1}{2(1 + \nu_0)} & 0 \\ & & & & & \frac{A_5 E_0/E_1}{2(1 + \nu_0)} \end{bmatrix}$$

with:  $A_5 = \left(\frac{\nu_1}{\nu_2} - \nu_1^2\right) \frac{E_1}{E_0} - \nu_1^2 (1 + 2\nu_1 + \frac{\nu_1}{\nu_2})$

$$[\Delta \sigma^{(1,3)}]^T = [\Delta \sigma_{11} \quad \Delta \sigma_{22} \quad \Delta \sigma_{33} \quad \Delta \sigma_{12} \quad \Delta \sigma_{13} \quad \Delta \sigma_{23}]$$

$$[\Delta \epsilon^{(1,3)}]^T = [\Delta \epsilon_{11} \quad \Delta \epsilon_{22} \quad \Delta \epsilon_{33} \quad 2\Delta \epsilon_{12} \quad 2\Delta \epsilon_{13} \quad 2\Delta \epsilon_{23}]$$

calculated stress-strain curves with those obtained in tests will be presented at the conference.

5. Finite element analysis of a reinforced concrete slab

The influence of different assumptions concerning the stress-strain and failure behaviour of concrete was investigated with the numerical analysis of a reinforced concrete slab. Fig. 7 shows the calculated load deflection curves compared with test results obtained by Franz [13]. The comparison of various assumptions, described in Fig. 7, indicates the importance especially for the ultimate load range of incorporating a realistic formulation of biaxial concrete behaviour. Not only the ultimate load, but also the deflections near ultimate load are influenced by different assumptions of concrete behaviour. More detailed results of the numerical analysis are comprised elsewhere [14].

6. Notations, References

$\sigma_{11}, \sigma_{22}, \sigma_{33}$  : principal stresses, arranged algebraically  
(but: in the biaxial domain:  $\sigma_{33}=0$ )

$\epsilon_{11}, \epsilon_{22}, \epsilon_{33}$  : corresponding principal strains

$\alpha$  : angle between arbitrary x-y, and principal 1-2 coordinate system; counter-clockwise positive

$f_c$  : uniaxial, compressive cylindrical concrete strength  
 $\epsilon_{c,U}$  : ultimate strain corresponding to uniaxial cyl. strength  $f_c$   
 $E_0, \nu_0$  : initial values (at beginning of uniaxial loading) of Young's modulus and Poisson's ratio  
 $E_1, E_2, \nu_1, \nu_2$  : stress-dependent elasticity moduli and Poisson's ratios of the anisotropic stress-strain formulation, the principal directions 1 and 2 being the axes of stress-induced anisotropy (tangent value, superscript's stands for secant one)

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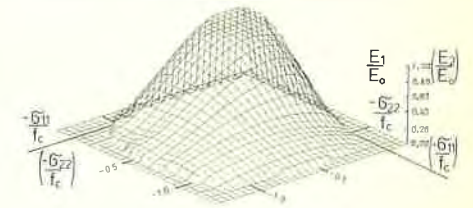
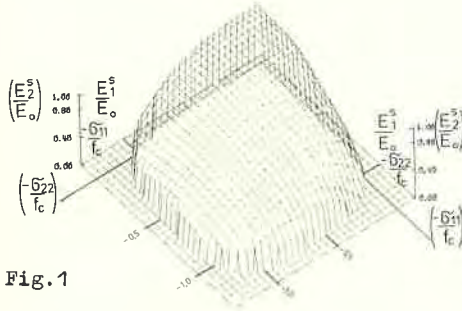
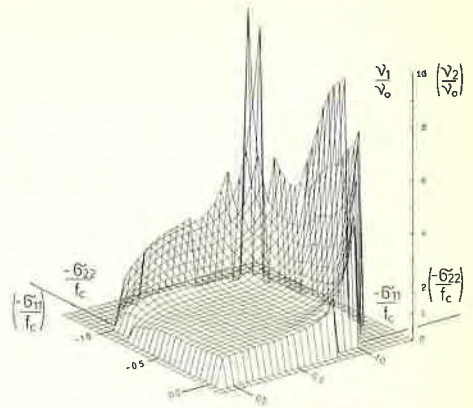
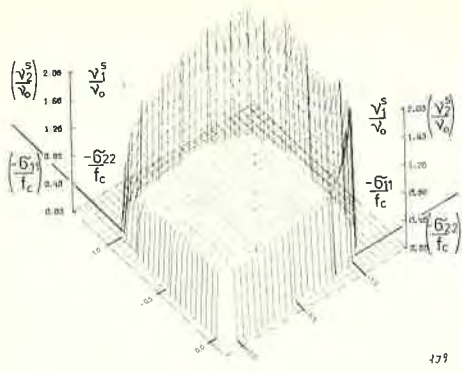


Fig.1

Fig.2

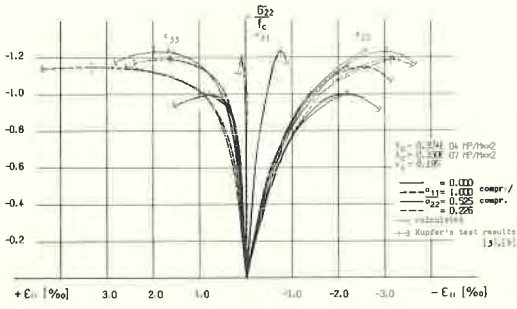


Fig.3

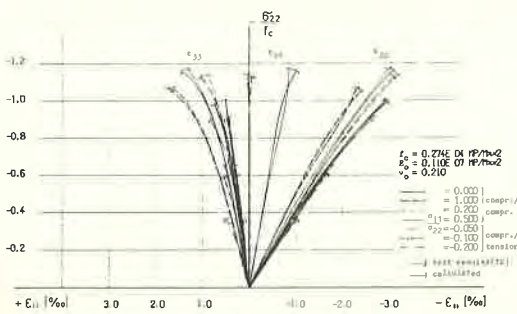


Fig.4

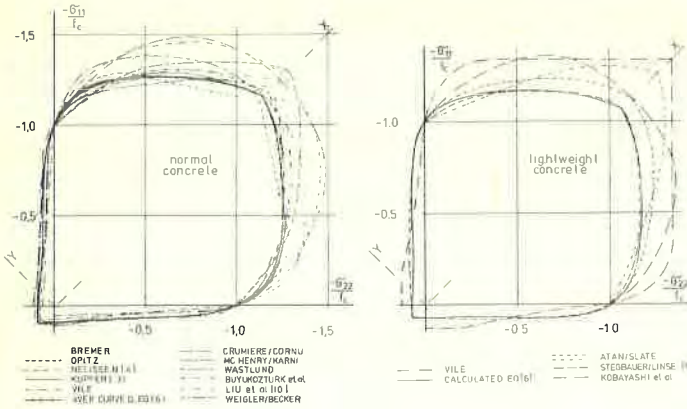


Fig.5

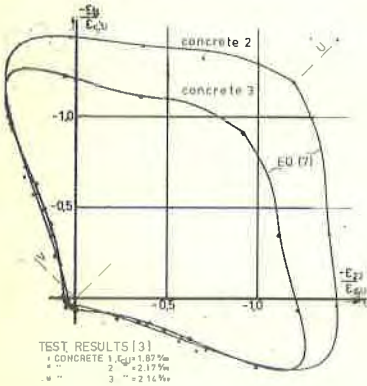


Fig.6

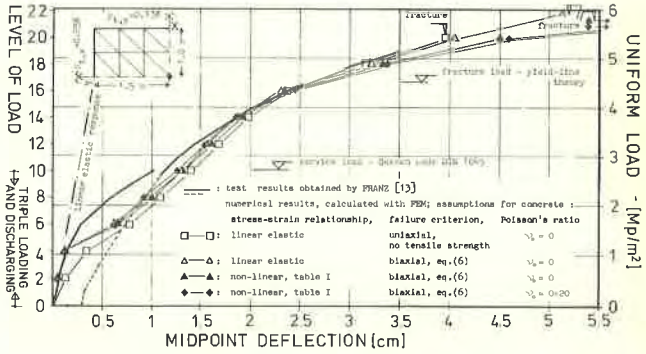


Fig.7

POLYNOMIAL COEFFICIENTS $A_1$ ACCORDING TO EQ. (6)										TABLE II
type	$f_0$ [k <sub>sp</sub> /cm <sup>2</sup> ]	$A_0$	$A_1$	$A_2$	$A_3$	$A_4$	$A_5$	$A_6$	$A_7$	
concrete 1	192	0.1535	1.0556	-0.3081	-1.7391	6.5294	-9.1747	5.2937	-1.0879	0
concrete 2	324	0.1253	0.9309	-0.2515	0.4045	2.1693	-6.6915	5.3193	-1.3357	0
concrete 3	619	0.1196	0.8414	-1.0010	4.7454	-6.2908	1.4994	1.4218	-0.6206	0
lightweight	274	0.09915	0.9406	-0.3003	3.2897	-6.7550	3.5832	0.1611	-0.3952	0
average curve I		0.1231	0.8335	0.2191	2.0055	-7.1955	7.9627	-4.0104	0.7742	0
average curve II		0.1203	0.3682	-0.1910	2.8430	-5.4875	3.7567	-1.0154	0.06664	0.4335

POLYNOMIAL COEFFICIENTS $B_1$ ACCORDING TO EQ. (7)								
type	$\epsilon_{c,U}$ [‰]	$B_0$	$B_1$	$B_2$	$B_3$	$B_4$	$B_5$	$B_6$
concrete 2	2.17	0.07118	1.6077	2.2264	-1.7707	-5.0898	5.4652	-1.4778
concrete 3	2.14	0.09937	1.2405	-1.6506	27.2072	-66.968	57.244	16.5738

# TABLE I

FORTRAN-function subroutines of material characteristic values (secant and tangent functions of principal stresses)

Notations:

- $SQ11 = \sigma_{11}/f_c$      $SQ22 = \sigma_{22}/f_c$      $EMO = E_0$      $QUO = \nu_0$
- $E_2^2 =$  FUNCTION FEM2(EMO, SQ11, SQ22)
- $\nu_1^2 =$  FUNCTION FVM1(QUO, SQ11, SQ22)
- $E_1 =$  FUNCTION FEM1(EMO, SQ11, SQ22)
- $\nu_1 =$  FUNCTION FVM1(QUO, SQ11, SQ22)

## I b

```

0002  FUNCTION FVM1(QUO, SQ11, SQ22)
0003  * LETZTE AENDERUNG VDM., 17. JAN. 1975
0004  DATA Z1 /13.25, 50.57, 71.73, 46.57, 13.105, 0.954, 0.0867, 0.1999/
0005  Z2 /13.1, 1.7, 4.75, 9.999, 15.870, 1.387, 9.72, 45.4, 7087.4/
0006  Z3 /12.98, 4.27, 69.61, 42, 76.7, 789, 2.998, 1.675, 1.077, 4/
0007  Z4 /50.5, 23.28, 28.45, 35.58, 49.81, 0.11, 1.768, 1.526, 0.33/
0008  Z5 /2.50, 45.82471, 1.4462, 8.9, 102.87, 1.967, 2.9, 801.6, 25.55/
0009  Z6 /2.867, 28.45, 37.32, 20.25, 5.384, 0.3857, 0.1919, 0.14/
0010  CALL ERSET(207, 999, 1, 1, 1, 209)
0011  PUL1=0.
0012  PUL2=0.
0013  PUL3=0.
0014  PUL4=0.
0015  PUL5=0.
0016  PUL6=0.
0017  PUL7=1.
0018  PUL8=21.
0019  PUL9=21.
0020  PUL10=21.
0021  PUL11=21.
0022  PUL12=21.
0023  PUL13=21.
0024  PUL14=21.
0025  PUL15=21.
0026  ARG=28.
0027  ARG2=(POL3*(POL5+SQ11)+POL6)
0028  ARG3=(POL4*(POL5+SQ11)+POL6)
0029  ARG4=(POL5*(POL5+SQ11)+POL6)
0030  ARG5=(POL6*(POL5+SQ11)+POL6)
0031  ARG6=100.
0032  ARG7=(SQ11*(0.33+0.25*CUSH(ARG6)/SINH(ARG6)))
0033  ARG8=(1.029*APDL1+0.03*(0.8*COSH(ARG7)/SINH(ARG7))+0.999*APDL4+
0034  (1.0*CUSH(ARG7)/SINH(ARG7))-0.9, 4.0*(1.0*SINH(ARG7)/COSH(ARG7))
0035  (0.028+0.0686*(0.894*COSH(ARG3)+0.493*COSH(ARG4)+0.201*COB
0036  (3*ARG2)+0.0826*COSH(ARG5)))/0.0404*(1.0*STANH(ARG8))
0037  FVM1=ARG8*ARG9
0038  RETURN
0039  END

```

## I c

```

0002  FUNCTION FOM2(QUO, SQ11, SQ22)
... as FUNCTION FOM1, but SQ11 and SQ22 interchanged

```

## I d

```

0002  FUNCTION FEM2(F=0, SQ11, SQ22)
* LETZTE AENDERUNG VDM., 17. JAN. 1975
0003  DATA Z1 /-2.4873, -20.0868, -51.5789, -59.667, -34.5181,
0004  -9.769, -1.4205, 0.1/
0005  CALL ERSET(207, 999, 1, 1, 1, 209)
0006  PUL5=0.
0007  PUL6=0.
0008  PUL7=1.
0009  PUL8=(POL5+21.0)*SQ11
0010  PUL9=(POL5+21.0)*SQ22
0011  PUL10=0.
0012  PUL11=0.
0013  PUL12=0.
0014  PUL13=0.
0015  PUL14=0.
0016  PUL15=0.
0017  PUL16=0.
0018  PUL17=0.
0019  PUL18=0.
0020  PUL19=0.
0021  PUL20=0.
0022  PUL21=0.
0023  PUL22=0.
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0025  PUL24=0.
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0027  PUL26=0.
0028  PUL27=0.
0029  PUL28=0.
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0031  PUL30=0.
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0089  PUL88=0.
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0095  PUL94=0.
0096  PUL95=0.
0097  PUL96=0.
0098  PUL97=0.
0099  PUL98=0.
0100  PUL99=0.
0101  PUL100=0.
0102  PUL101=0.
0103  PUL102=0.
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