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**NON-LINEAR FINITE ELEMENTS ANALYSIS  
OF REINFORCED CONCRETE FRAMES  
WITH MASONRY FILLER WALLS**

**THESIS**

**Faik Kivanç**

BOĞAZİÇİ UNIVERSITY  
Civil Engineering Department  
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Faik Kivanç

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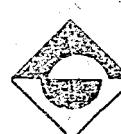
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## ABSTRACT

An incremental non-linear finite element program taking into consideration the nonlinear behaviour and failure of mortar joints in masonry is developed and applied to the analysis of reinforced concrete frames with masonry filler walls. Tension cracks in reinforced concrete frame and brick elements are taken into consideration also. A failure criterion is adopted for mortar joint elements to simulate their failure. Several failure criteria, material properties of mortar and loading types are used to study their effects. Resulting crack patterns and load-deflection curves are presented.

## ÖZET

Tuğla duvarlarda ki harçların doğrusal olmayan davranışını ve çatlamalarını dikkate alan doğrusal olmayan bir sonlu elemanlar programı geliştirilmiş ve betonarme çerçeveye içinde ki tuğla dolgu duvarlara uygulanmıştır. Betonarme çerçevede ki ve tuğlalarda ki gerilme çatlakları da dikkate alınmıştır. Harç elemanlarının kırılmalarını simüle etmek Üzere bir kırılma kriteri kullanılmıştır. Değişik etkileri incelemek amacıyla çeşitli kırılma kriterleri, harç malzeme özellikleri ve yükleme tipleri uygulanmıştır. Sonuç olarak elde edilen çatlama şıkları ve yük-sehim eşrileri gösterilmiştir.

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## LIST OF SYMBOLS

- $\{u\}$  : Displacements within the finite element  
 $\{d\}$  : Nodal displacements  
 $[N]$  : Shape functions  
 $[\varepsilon]$  : Strain vector  
 $[\Delta]$  : Linear operator  
 $[G]$  : Strain matrix  
 $\{\sigma\}$  : Stress vector  
 $[D]$  : Elasticity matrix  
 $[S]$  : Stress matrix  
 $\Pi$  : Potential energy  
 $U$  : Strain energy  
 $W$  : Work done by external loads  
 $\{p_b\}$  : Body force vector  
 $\{p_s\}$  : Distributed force vector  
 $\{p_N\}$  : Nodal force vector  
 $[k]$  : Element stiffness matrix  
 $\{\sigma_0\}$  : Initial stresses  
 $\{\varepsilon_0\}$  : Initial strains  
 $\{p\}$  : Load vector of the system  
 $\{w\}$  : Relative displacements of joint element  
 $[K]$  : System stiffness matrix  
 $\{F\}$  : Unit force vector  
 $[k_u]$  : Joint unit property matrix  
 $k_s$  : Unit shear stiffness of joint element  
 $k_n$  : Unit normal stiffness of joint element  
 $t_m$  : Joint thickness

- T : Wall thickness
- [T] : Transformation matrix
- $\xi$  : Dimensionless coordinate in x-direction
- $\eta$  : Dimensionless coordinate in y-direction
- $\sigma_1$  : Maximum principal stress
- $\sigma_2$  : Minimum principal stress
- $\{S_c\}$  : Stress vector of reinforced concrete element
- $\{S_{cp}\}$  : Principal stress vector of reinforced concrete element
- $\{S_{ccp}\}$  : Principal stress vector of cracked reinforced concrete
- $\theta$  : Angle of principal direction
- $\delta$  : A sufficiently small number

## 1. INTRODUCTION

### 1.1 GENERAL

In structures walls and partitions are created by infilling frames with construction materials such as bricks or concrete blocks. Although it is common practise that these masonry infills are not included in design calculations of framed structures, they obviously have some effect on the overall behaviour of the structure. Unless they are separated from the frame, their interaction with the structure has to be into account in design calculations. Overall stiffness, energy absorbtion capacity and shear distribution throughout the structure may then be predicted more realistically.

At low stress levels masonry can be considered as an assemblage of brick and mortar joints with isotropic and linear elastic behaviour. At higher stress levels, however, behaviour of mortar joints are nonlinear. Due to this fact and also due to the cracking of some mortar joints and bricks at certain areas stress redistributions occur, which can not be neglected. Methods such as using equivalent struts to represent the action of the infill panel may be useful in an approximate analysis at low stress levels, but at higher stress levels, especially near failure of the infill, a more sophisticated method accounting for the nonlineari-

ties and cracks in masonry should be used.

## 1.2 FORMER STUDIES ON MASONRY INFILL PANELS AND THEIR INTERACTION WITH FRAMES

Behaviour of masonry itself and its interaction with frames has been a subject of interest for a long time. Benjamin and Williams<sup>(4)</sup> performed a set of tests on one-storey reinforced concrete frames with brick masonry infills under lateral loading. Main variables in these tests were wall dimensions, mortar properties and scale of the structure. Results were expressed in load-deflection curves for various types of walls. Smith and Carter<sup>(15)</sup> examined the behaviour of multistorey infilled frames under the effect of lateral loading. Lateral strength was examined and empirical formulas and design graphs were given to predict the cracking and crushing strength of concrete and brickwork.

Yakel and Fattal<sup>(16)</sup> made various studies about the load capacities of clay masonry walls subjected to a diagonal compressive load combined with a compressive edge load acting in the plane of the wall and normal to the direction of mortar bed joint. A failure hypothesis was also developed accounting for the observed failure modes. Tests were made by Meli<sup>(17)</sup> on full scale masonry panels subjected to lateral loads. Walls encased in concrete frames, walls with concrete tie columns and interiorly reinforced walls were included in this study. Strength, stiffness, modes of failure and postcracking behaviour of the walls were discussed. Umemura et al.<sup>(7)</sup> performed a series of tests on plain brick walls of one quarter size model with cement mortar and lime mortar, with and without frames. The purpose of these tests

were to observe the behaviour of plain brick walls under the action of a combination of lateral and vertical forces. An analytical approach to the behaviour of masonry as deep beams was made by Page<sup>(3)</sup>. In his study Page used the finite element method to predict the cracking patterns of mortar joints in brick masonry deep beams, where he considered the nonlinear mortar joint deformation characteristics also. He made use of a failure criterion for joint elements, which he developed as a result of tests performed on masonry panels. Stress-strain curves of mortar joints, again resulting from tests, were presented also. Effects of infill panels on overall seismic response of structures were investigated by Dowrick<sup>(20)</sup>. Mayes et al.<sup>(8)</sup> presented in their study a summary of works on the evaluation of the seismic design section of the 1972, 1973, 1974 and 1976 'Uniform Building Codes', and the recommended 'Comprehensive Seismic Design Provisions for Buildings' prepared by the Applied Technology Council.

### 1.3 OBJECT AND SCOPE

This study deals with masonry panels encased in reinforced concrete frames subjected to lateral loading or a combination of lateral and vertical loading. The main object of the study is to develop an analytical model which predicts the type and degree of cracking of the masonry panel at various load levels and to study the effect of masonry infills on the behaviour of the reinforced concrete frames.

An incremental finite element program modeling

- a) Nonlinear behaviour of mortar joints
- b) Tensile splitting in bricks

c) Effect of tensile cracks in reinforced concrete frame has been developed. The model allows progressive joint failure to occur.

## 2. FINITE ELEMENT FORMULATION

### 2.1 INTRODUCTION

With the advances in digital computers, the finite element method became a very popular technique in handling complicated engineering problems. By this method, a continuum is discretized and problems can generally be solved readily even for very complicated boundary conditions. In this chapter, after a general formulation of the finite element method, element details used in the structural model are presented.

### 2.2 GENERAL FINITE ELEMENT FORMULATION

The stress analysis of a continuous system can be performed by discretizing the system into a gridwork of finite sized, two dimensional elements interconnected at their corners. To avoid conceptual difficulties the problem is illustrated with a very simple example of plane stress analysis of a thin slice, shown in Fig. 1.

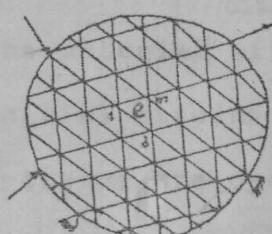


Fig. 1: Finite Element Discretization of a Plane Stress Region

A typical finite element,  $e$ , is defined by its nodes  $i, j, m$  and straight line boundaries. Let the displacements  $u$  at any point within the element be expressed as a column vector,  $\{u\}$ .  $\{u\}$  can be written as a function of the nodal displacements as

$$\{u\} [N_i, N_j, \dots] \begin{bmatrix} d_i \\ d_j \\ \vdots \\ \vdots \end{bmatrix} = [N] \{d\} \quad (2.1)$$

Here,  $\{d\}$  represents the nodal displacements for a particular element.  $[N]$  is the vector of shape functions and has to be so chosen as to give appropriate nodal displacements, when the coordinates of the appropriate nodes are inserted in equation (2.1).

With displacements known at all points within the element, strains at any point can be determined by the relation:

$$\{\epsilon\} = [\Delta] \{u\}$$

where  $[\Delta]$  is a suitable linear operator. Using equation (2.1), the above equation can be expressed as:

$$\{\epsilon\} = [G] \{d\} \quad (2.2)$$

are  $[G] = [\Delta][N]$  is called the strain matrix

For a plain stress case, strains are defined in terms of displacements by well-known relations<sup>(1)</sup> which define  $[\Delta]$ :

$$\{\epsilon\} = \begin{bmatrix} \partial u / \partial x \\ \partial v / \partial y \\ \partial u / \partial y + \partial v / \partial x \end{bmatrix} = \begin{bmatrix} \partial / \partial x & 0 \\ 0 & \partial / \partial y \\ \partial / \partial y & \partial / \partial x \end{bmatrix} \begin{bmatrix} u \\ v \end{bmatrix}$$

In general, the material within the element boundaries may be subjected to initial strains due to temperature changes, shrinkage etc.. If such strains are denoted by  $\{\epsilon_0\}$ , then stresses will be caused by the difference between the actual and initial strain

In addition, it is convenient to assume that at the beginning of the analysis the body is stressed with initial stresses  $\{\sigma_0\}$ . Thus, assuming general linear elastic behaviour, the relationship between stresses and strains will be of the form:

$$\{\sigma\} = [D] (\{\epsilon\} - \{\epsilon_0\}) + \{\sigma_0\} \quad (2.3)$$

where

$$[D] = \frac{E}{1-\nu^2} \begin{bmatrix} 1 & \nu & 0 \\ \nu & 1 & 0 \\ 0 & 0 & \frac{1-\nu}{2} \end{bmatrix}$$

is the elasticity matrix.

$$\{\sigma\} = [D] ([G]\{d\} - \{\epsilon_0\}) + \{\sigma_0\}$$

$$[\sigma]\{d\} - [D]\{\epsilon_0\} + \{\sigma_0\} \quad (2.4)$$

in which  $[\sigma] = [D][G]$  is called the stress matrix.

Because the displacement models are separately assumed for each element of the continuum with interelement compatibility maintained to the necessary degree, total potential energy of the continuum,  $\Pi$ , can be thought to be equal to the sum of the potential energies of individual elements:

$$\Pi = \sum \Pi_e \quad (2.5)$$

The potential energy functional  $\Pi_e$  of an element is:

$$\Pi_e = \underbrace{\frac{1}{2} \int_U \{\epsilon\}^T \{\sigma\} dV}_{U} - \left[ \underbrace{\int_V \{u\}^T \{p_B\} dV}_{W} + \underbrace{\int_W \{u\}^T \{p_s\} ds}_{W} + \{d\}^T \{p_N\} \right]$$

where

$U$ : strain energy

$W$ : work done by external loads

$\{p_B\}$ : body force vector

$\{p_s\}$ : distributed force vector (surface tractions)

$\{p_N\}$ : nodal force vector

Using equations (2.2) and (2.3)  $\Pi_e$  is obtained as:

$$\begin{aligned}\Pi_e = & \frac{1}{2} \int \{d\}^T [G]^T [D] [G] \{d\} dV - \frac{1}{2} \int \{d\}^T [G]^T [D] \{\epsilon_0\} dV + \\ & + \frac{1}{2} \int \{d\}^T [G]^T \{\epsilon_0\} dV - \int \{d\}^T [N]^T \{p_B\} dV - \int \{d\}^T [N]^T \{p_s\} ds - \\ & - \{d\}^T \{p_N\}\end{aligned}$$

By the principle of minimum potential energy, of all the displacement configurations satisfying kinematic and geometric boundary conditions, the configuration which makes the potential energy minimum satisfies the equilibrium conditions. For the potential energy to be minimum, its first variation must vanish.

$$\delta \Pi_e = \delta U - \delta W = 0$$

Thus,

$$\begin{aligned}\delta \Pi_e = 0 = & [\delta d]^T \left[ \left( \int [G]^T [D] [G] dV \right) \{d\} - \frac{1}{2} \left( \int [G]^T dV \right) [D] \{\epsilon_0\} + \right. \\ & \left. + \frac{1}{2} \left( \int [G]^T dV \right) \{\epsilon_0\} - \int [N]^T \{p_B\} dV - \int [N]^T \{p_s\} ds - \{p_N\} \right]\end{aligned}$$

Since the variations of the nodal displacements  $\{\delta d\}$  are arbitrary, the expression in the brackets must vanish. This gives the equilibrium equations for the element:

$$[k] \{d\} = \{p_N\} - (\{f\}_{\epsilon_0} - \{f\}_{\epsilon_0} - \{f\}_{p_B} + \{f\}_{p_s}) \quad (2.5)$$

with

$$\begin{aligned}[k] &= \int [G]^T [D] [G] dV \\ \{f\}_{\epsilon_0} &= - \left( \frac{1}{2} \int [G]^T dV \right) [D] \{\epsilon_0\} \\ \{f\}_{\epsilon_0} &= - \left( \frac{1}{2} \int [G]^T dV \right) \{\epsilon_0\} \\ \{f\}_{p_B} &= - \int [N]^T \{p_B\} dV \\ \{f\}_{p_s} &= - \int [N]^T \{p_s\} ds\end{aligned}$$

Here  $[k]$  is called the stiffness matrix of the element.

The next step is to form the system stiffness matrix  $[K]$  and system load vector  $\{P\}$ . An efficient way of doing this is the code number technique<sup>(10)</sup>. Thus equilibrium equations for the system takes the form:

$$[K]\{d\}_{sys} \{P\} \quad (2.6)$$

where  $\{d\}_{sys}$  is vector of system nodal displacements.

After solving equation (2.6) for  $\{d\}_{sys}$ , displacements for each element can be obtained from  $\{d\}_{sys}$ , and stresses in each element can be calculated using equation (2.4).

## 2.3 STRUCTURAL MODEL

The system under consideration is a brick masonry panel encased in a reinforced concrete frame. The inplane behaviour of masonry is modeled using an elastic continuum of plane stress brick elements with superimposed linkage elements simulating the mortar joints. For reinforced concrete frame, again plane stress elements are used taking into consideration the nonhomogeneity caused by the reinforcing steel bars.

## 2.4 ELEMENT DETAILS

### 2.4.1 JOINT ELEMENTS

In modeling the mortar joint elements between brick elements, a one dimensional element capable of undergoing relative displacements is used. This element type was developed by Goodman et al.<sup>(2)</sup> in their study of rock-joints, but it has been adopted

to masonry also<sup>(3)</sup>. Element geometry in local coordinate system is shown below:

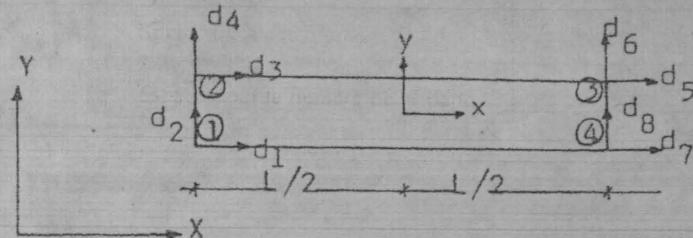


Fig.2. Joint Element (After Page<sup>(3)</sup>)

Since the joint elements are extremely thin, pairs of nodes (1,2) and (3,4) in Fig.2 are specified by the same coordinates. Thus as far as geometry is concerned the thickness of the element is zero. However, a thickness  $t_m$  is used in computing joint element properties.

It is assumed that normal and shear displacements along the element vary linearly, and that the one dimensional element has zero thickness, as mentioned above. Since the joint element can deform only in normal and shear directions, the relative displacement vector  $\{w\}$  at any point along the joint is given by:

$$\{w\} = \begin{bmatrix} w_s^{\text{top}} & -w_s^{\text{bottom}} \\ w_n^{\text{top}} & w_n^{\text{bottom}} \end{bmatrix} \quad (2.7)$$

Subscripts s and n denote shear and normal (x and y), respectively. If the vector of forces per unit length of joint element is taken as

$$\{F\} = \begin{bmatrix} F_s \\ F_n \end{bmatrix}$$

it can be expressed in terms of the element relative displacements

$$\{F\} = [k_u] \{w\}$$

where  $[k_u]$  is a diagonal material property matrix expressing joint stiffness per unit length in shear and normal directions:

$$[k_u] = \begin{bmatrix} k_s & 0 \\ 0 & k_n \end{bmatrix}$$

For the shear direction, substituting  $A=T \cdot L$  and  $L=t_m$  (where  $T$  represents wall thickness) into the formula  $\delta = \frac{PL}{AG}$ :

$$\delta = \frac{Pt}{T \cdot L \cdot G}$$

solving for  $P$

$$P = \delta \frac{G \cdot T}{t_m \cdot L}$$

$k_s$  is found as

$$k_s = \frac{G \cdot T}{t_m}$$

Similarly, for the normal direction with  $\delta = \frac{P \cdot L'}{AE}$ ,  $A=T \cdot L$  and  $L'=t_m$

$k_n$  is determined as

$$k_n = \frac{E \cdot T}{t_m}$$

Carrying out the matrix multiplication yields

$G$  and  $E$  are instantaneous shear and elastic moduli at the particular shear and normal stress levels and can be determined from stress-strain curves for mortar.

The strain energy of the joint element is

$$U = -\frac{1}{2} \int_{-\frac{L}{2}}^{\frac{L}{2}} \{w\}^T \{F\} dx = -\frac{1}{2} \int_{-\frac{L}{2}}^{\frac{L}{2}} \{w\}^T [k_u] \{w\} dx \quad (2.8)$$

The relative displacements  $\{w\}$  can be expressed in terms of the nodal displacements  $\{d\}$  through linear displacement formulas

$$w_s^{\text{bottom}} = \left(-\frac{1}{2} - \frac{x}{L}\right)d_1 + \left(\frac{1}{2} + \frac{x}{L}\right)d_7$$

In equation (2.11) the only term varying along the section are  $w_s^{\text{bottom}}$  and  $w_n^{\text{bottom}}$

$$w_n^{\text{bottom}} = \left(-\frac{1}{2} - \frac{x}{L}\right)d_2 + \left(\frac{1}{2} + \frac{x}{L}\right)d_8$$

$$w_s^{\text{top}} = \left(-\frac{1}{2} - \frac{x}{L}\right)d_3 + \left(\frac{1}{2} + \frac{x}{L}\right)d_5$$

$$w_n^{\text{top}} = \left(-\frac{1}{2} - \frac{x}{L}\right)d_4 + \left(\frac{1}{2} + \frac{x}{L}\right)d_6$$

Substituting into equation (2.7)

$$\{w\} = \begin{Bmatrix} w_s^{\text{top}} & -w_s^{\text{bottom}} \\ w_n^{\text{top}} & -w_n^{\text{bottom}} \end{Bmatrix} = -\frac{1}{2} \begin{bmatrix} -A & 0 & A & 0 & B & 0 & -B & 0 \\ 0 & -A & 0 & A & 0 & B & 0 & -B \end{bmatrix} \begin{Bmatrix} d_1 \\ d_2 \\ d_3 \\ d_4 \\ d_5 \\ d_6 \\ d_7 \\ d_8 \end{Bmatrix}$$

where

$$A = 1 - \frac{2x}{L} ; \quad B = 1 + \frac{2x}{L}$$

This equation can be written as

$$\{w\} = [G] \{d\} \quad (2.9)$$

with

$$G = -\frac{1}{2} \begin{bmatrix} -A & 0 & A & 0 & B & 0 & -B & 0 \\ 0 & -A & 0 & A & 0 & B & 0 & -B \end{bmatrix}$$

Substituting (2.9) into equation (2.8)

$$\Pi = \frac{1}{2} \int_{-\frac{L}{2}}^{\frac{L}{2}} \{d\}^T [G]^T [k_u] [G] \{d\} dx \quad (2.10)$$

Carrying out the triple matrix multiplication yields

$$[G]^T [k_u] [G] = \begin{bmatrix} A^2 k_s & 0 & -A^2 k_s & 0 & -ABk_s & 0 & ABk_s & 0 \\ 0 & A^2 k_n & 0 & -A^2 k_n & 0 & -ABk_n & 0 & ABk_n \\ -A^2 k_s & 0 & A^2 k_s & 0 & ABk_s & 0 & -ABk_s & 0 \\ 0 & -A^2 k_n & 0 & A^2 k_n & 0 & ABk_n & 0 & -ABk_n \\ -ABk_s & 0 & ABk_s & 0 & B^2 k_s & 0 & -B^2 k_s & 0 \\ 0 & -ABk_n & 0 & ABk_n & 0 & B^2 k_n & 0 & -B^2 k_n \\ ABk_s & 0 & -ABk_s & 0 & -B^2 k_s & 0 & B^2 k_s & 0 \\ 0 & ABk_n & 0 & -ABk_n & 0 & -B^2 k_n & 0 & B^2 k_n \end{bmatrix} \cdot \frac{1}{4} \quad (2.11)$$

In equation (2.11) the only terms varying along the x-direction are  $A^2$ ,  $B^2$  and  $AB$ , that is,  $(1 - \frac{2x}{L})^2$ ,  $(1 + \frac{2x}{L})^2$  and  $(1 - \frac{2x}{L})(1 + \frac{2x}{L})$ .

There are thus three types of integrals to be evaluated:

$$\int_{-\frac{L}{2}}^{\frac{L}{2}} (1 - \frac{2x}{L})^2 dx = \frac{4}{3} L$$

$$\int_{-\frac{L}{2}}^{\frac{L}{2}} \left(1 + \frac{2x}{L}\right)^2 dx = \frac{4}{3} L$$

$$\int_{-\frac{L}{2}}^{\frac{L}{2}} \left(1 - \frac{2x}{L}\right) \left(1 + \frac{2x}{L}\right) dx = -\frac{2}{3} L$$

Substituting the resulting integrals into equation (2.10) and taking respective derivatives of  $U$  with respect to  $d_i$ , joint element stiffness matrix  $[k]$  is obtained:

$$[k] = \begin{bmatrix} 2k_s & 0 & -2k_s & 0 & -k_s & 0 & k_s & 0 \\ 0 & 2k_n & 0 & -2k_n & 0 & -k_n & 0 & k_n \\ -2k_s & 0 & 2k_s & 0 & k_s & 0 & -k_s & 0 \\ 0 & -2k_n & 0 & 2k_n & 0 & k_n & 0 & -k_n \\ -k_s & 0 & k_s & 0 & 2k_s & 0 & -2k_s & 0 \\ 0 & -k_n & 0 & k_n & 0 & 2k_n & 0 & -2k_n \\ k_s & 0 & -k_s & 0 & -2k_s & 0 & 2k_s & 0 \\ 0 & k_n & 0 & -k_n & 0 & -2k_n & 0 & 2k_n \end{bmatrix} \cdot \frac{L}{6}$$

For horizontal joint elements local coordinates  $(x, y)$  and global coordinates  $(X, Y)$  coincide. However, to obtain the stiffness matrix of vertical joint elements in global coordinates a transformation from local to global coordinates is necessary. The transformation matrix  $[T]$  is generated as follows:

$$[t] = \begin{bmatrix} \cos\theta & \sin\theta \\ -\sin\theta & \cos\theta \end{bmatrix}$$

and for  $\theta = 90^\circ$

$$[t] = \begin{bmatrix} 0 & 1 \\ -1 & 0 \end{bmatrix}$$

where  $[t]$  is a matrix of direction cosines.

$[T]$  has the form:

$$[T] = \begin{bmatrix} [t] & 0 & 0 & 0 \\ 0 & [t] & 0 & 0 \\ 0 & 0 & [t] & 0 \\ 0 & 0 & 0 & [t] \end{bmatrix}$$

and the stiffness matrix in global coordinates is obtained as:

$$[k]_{XY} = [T]^T [k]_{XY} [T]$$

The triple matrix multiplication gives:

$$[k]_{XY} = \begin{bmatrix} 2k_n & 0 & -2k_n & 0 & -k_n & 0 & k_n & 0 \\ 0 & 2k_s & 0 & -2k_s & 0 & -k_s & 0 & k_s \\ -2k_n & 0 & 2k_n & 0 & k_n & 0 & -k_n & 0 \\ 0 & -2k_s & 0 & 2k_s & 0 & k_s & 0 & -k_s \\ -k_n & 0 & k_n & 0 & 2k_n & 0 & -2k_n & 0 \\ 0 & -k_s & 0 & k_s & 0 & 2k_s & 0 & -2k_s \\ k_n & 0 & -k_n & 0 & -2k_n & 0 & 2k_n & 0 \\ 0 & k_s & 0 & -k_s & 0 & -2k_s & 0 & 2k_s \end{bmatrix} \cdot \frac{L}{6}$$

In horizontal joint elements, element displacements in global and member axes are identical, but in vertical joint elements a transformation has to be applied to obtain element displacements in member axes, after the system of equations (2.6) are solved and element displacements are obtained in global axes. The transformation is done by using the matrix  $[T]$  again:

$$\{d\}_{XY} = [T] \{d\}_{XX}$$

This process is illustrated in Fig.3.

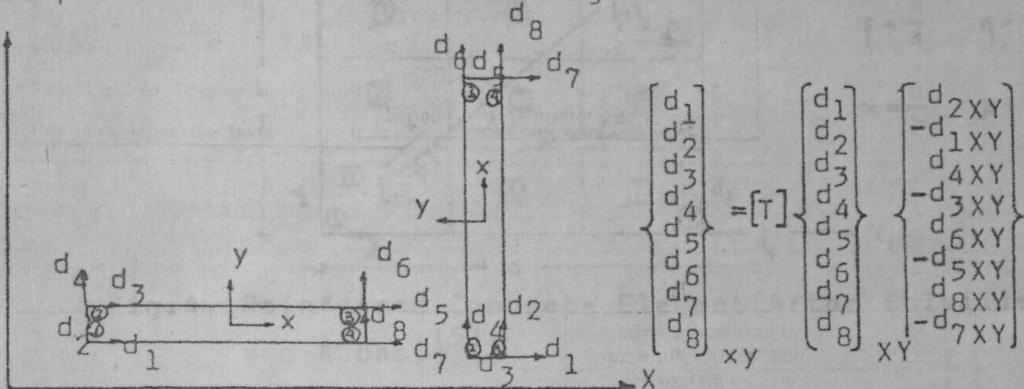


Fig.3. Coordinate Transformation For Joint Element

The element displacements are then used to calculate the stresses in the joint elements. Stresses are calculated at the middle of the joint elements, i.e. at  $x=0$ , as follows:

$$\{\sigma\} = \frac{1}{T} \left[ k_u \right] \{w\} = \frac{1}{T} \left[ k_u \right] \left[ G \right] \{d\}_{xy} \quad (2.12)$$

Since  $A=B=1$  at the middle of the element:

$$\begin{Bmatrix} \sigma \\ \epsilon \end{Bmatrix} = \frac{1}{2T} \begin{bmatrix} k_s & 0 \\ 0 & k_n \end{bmatrix} \begin{bmatrix} -1 & 0 & 1 & 0 & 1 & 0 & -1 & 0 \\ 0 & -1 & 0 & 1 & 0 & 1 & 0 & -1 \end{bmatrix} \begin{Bmatrix} d_1 \\ d_2 \\ d_3 \\ d_4 \\ d_5 \\ d_6 \\ d_7 \\ d_8 \end{Bmatrix}$$

## 2.2.2 REINFORCED CONCRETE ELEMENTS

A rectangular element developed by Colville and Abbasi (5) is used for modeling reinforced concrete. Linear edge displacements are assumed and non-dimensional coordinates are used in the derivation of element properties. This type of an element is shown below:

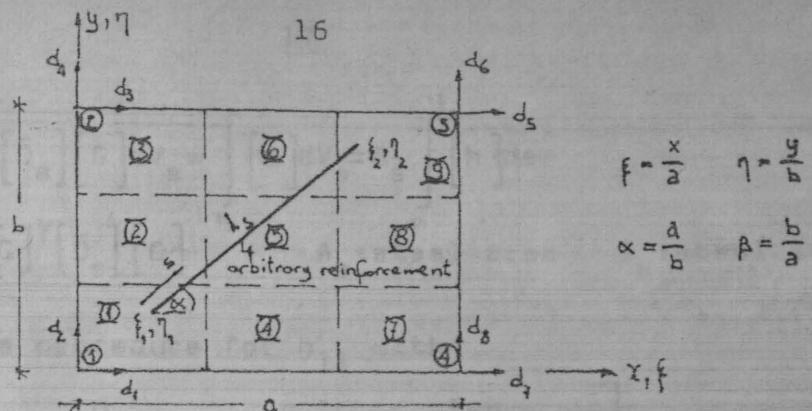


Fig.4. Reinforced Concrete Element (After Colville and Abbasi (5))

Since the stresses are not constant over the element area, the element is divided into 9 subregions and stresses are computed at the centroids of these subregions.

Stiffness matrix of the reinforced concrete element,  $[k_{c,s}]$ , must be computed as the sum of stiffnesses of steel and concrete components. Considering concrete as a linearly elastic, isotropic and homogenous material,

$$\text{Let } [k_c] = \int [G]^T [D_c] [G] dV = T \int [G]^T [D_c] [G] dA$$

Explicit form of  $[k_c]$  is given in Appendix 1.

$$[k_{c,s}] = [k_c] + \sum_{l=1}^m [k_{sl}] \quad (2.13)$$

where

$$[k_s] = \int [G]^T [\bar{D}_s] [G] dV_s$$

with

$$[\bar{D}_s] = [D_s] - [D_c]$$

$m$ : number of reinforcing steel bars in the element

$D_s$  : elasticity matrix of steel

$D_c$  : elasticity matrix of concrete

Thus  $[k_s]$  is obtained by taking the line integral over the volume of steel contained in the element:

$$[k_s] = \int_{V_s} [G]^T [\bar{D}_s] [G] dV_s = \int_{V_s} [h] dV_s = A_s \int_0^{l_s} [h] ds$$

where from  $h = [G]^T [\bar{D}_s] [G]$   $A_s$  : steel area  $l_s$  : steel length

Forcing bar in the element stiffness is obtained using section illustrating the procedure for  $h_{11}$  with

$$[G] = \begin{bmatrix} -\frac{1-\eta}{a} & 0 & -\frac{\eta}{a} & 0 & \frac{\eta}{a} & 0 & \frac{1-\eta}{a} & 0 \\ 0 & -\frac{1-\xi}{b} & 0 & \frac{1-\xi}{b} & 0 & \frac{\xi}{b} & 0 & -\frac{\xi}{b} \\ -\frac{1-\xi}{b} & -\frac{1-\eta}{a} & \frac{1-\xi}{b} & -\frac{\eta}{a} & \frac{\xi}{b} & \frac{\eta}{a} & -\frac{\xi}{b} & \frac{1-\eta}{a} \end{bmatrix}$$

and using the notation from Fig.4

$$h_{11} = G_{11} \bar{D}_{s11} G_{11} - G_{31} \bar{D}_{s33} G_{31} = \frac{(1-\eta)^2}{a^2} \bar{D}_{s11} - \frac{(1-\xi)^2}{b^2} \bar{D}_{s33}$$

given in explicit form in Appendix

$$h_{11} = (1-\eta_1 - sS)^2 \frac{\bar{D}_{s11}}{a^2} - (1-\xi_1 - sC)^2 \frac{\bar{D}_{s33}}{b^2}$$

$$\text{Let } \bar{\xi} = 1 - \xi_1 \quad \bar{\eta} = 1 - \eta_1 \quad D_{11} = \frac{\bar{D}_{s11}}{a^2} \quad D_{33} = \frac{\bar{D}_{s33}}{b^2}$$

$$h_{11} = (\bar{\eta} - sS)^2 D_{11} + (\bar{\xi} - sC)^2 D_{33}$$

$$\begin{aligned} k_{11} &= \int_{V_s} h_{11} dV_s = A_s \int_0^{l_s} h_{11} ds \\ &= A_s \int_0^{l_s} (\bar{\eta}^2 D_{11} - 2\bar{\eta} s S D_{11} + s^2 S^2 D_{11} + \bar{\xi}^2 D_{33} - 2\bar{\xi} s C D_{33} + s^2 C^2 D_{33}) ds = \\ &= (\bar{\eta}^2 D_{11} + \bar{\xi}^2 D_{33}) l_s + (-\bar{\eta} s D_{11} - \bar{\xi} s C D_{33}) l_s^2 + (s^2 D_{11} + C^2 D_{33}) \frac{l_s^3}{3} = A_s = \end{aligned}$$

$$k_{11} = V_s \left[ (\bar{\eta}^2 D_{11} + \bar{\xi}^2 D_{33}) + l_s (-\bar{\eta} s D_{11} - \bar{\xi} s C D_{33}) + \frac{l_s^2}{3} (s^2 D_{11} + C^2 D_{33}) \right]$$

$$k_{11} = V_s (\psi' - \psi'' - \psi''')$$

$$\text{with } \psi' = \bar{\eta}^2 D_{11} + \bar{\xi}^2 D_{33} ; \quad \psi'' = (-\bar{\eta} s D_{11} - \bar{\xi} s C D_{33}) l_s ; \quad \psi''' = \frac{1}{3} (s^2 D_{11} + C^2 D_{33}) l_s^2$$

$$\text{and } V_s = A_s l_s$$

Each item of  $[k_s]$  is in the form  $V_s (\psi + \psi'' + \psi'')$ , and the expressions for  $\psi$ ,  $\psi''$  and  $\psi'''$  are given in Appendix 2.

Repeating the procedure for evaluating  $[k_s]$  for each reinforcing bar in the element,  $[k_{c,s}]$  is obtained using equation (2.13)

#### 2.4.3 BRICK ELEMENTS

Bricks are modeled using conventional eight-parameter rectangular plane stress elements with isotropic and elastic properties shown in Fig.5.

The strain matrix  $[G]$  and the stiffness matrix  $[k]$  are given in explicit form in Appendix 1.

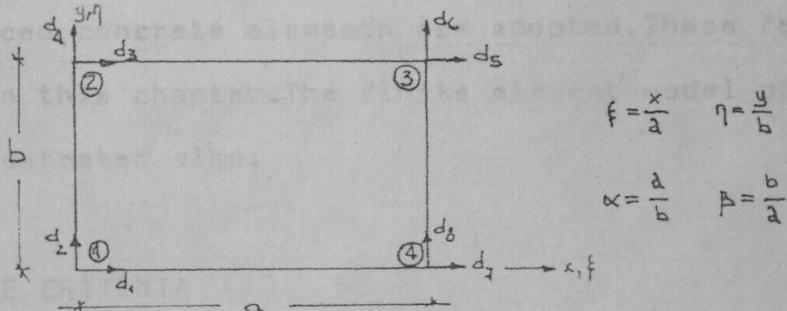


Fig.5. Brick Element

#### 3.2.1 FAILURE CRITERION FOR JOINT ELEMENTS

A failure criterion involving joint failure characteristics under various types of stress compositions has been derived by (3). This type of criterion is given in Fig.6. It has been obtained by plotting the test results in terms of ultimate shear and ultimate normal stresses. A linear best-fit curve has been used for simplicity in the compressive stress range. The characteristic slope corresponds to a change in the failure mode from bond load failure to a combined joint-brick failure.

### 3. ESSENTIAL ASPECTS OF SOLUTION PROCEDURE

#### 3.1 INTRODUCTION

Nonlinearities in material behaviour and local failures necessitate a nonlinear analysis procedure. Therefore an incremental step-iteration method and failure criteria for joint, brick and reinforced concrete elements are adopted. These features are presented in this chapter. The finite element model of the structure is illustrated also.

#### 3.2 FAILURE CRITERIA

##### 3.2.1 FAILURE CRITERION FOR JOINT ELEMENTS

Failure criterion simulating joint failure characteristics under various types of stress combinations has been derived by <sup>(3)</sup>. This type of criterion is given in Fig.6. It has been obtained by plotting the test results in terms of ultimate shear and ultimate normal stresses. Two linear best fit curves have been used for simplicity in the compressive stress region. The change in slope corresponds to a change in the failure modes from pure bond failure to a combined joint-brick failure.

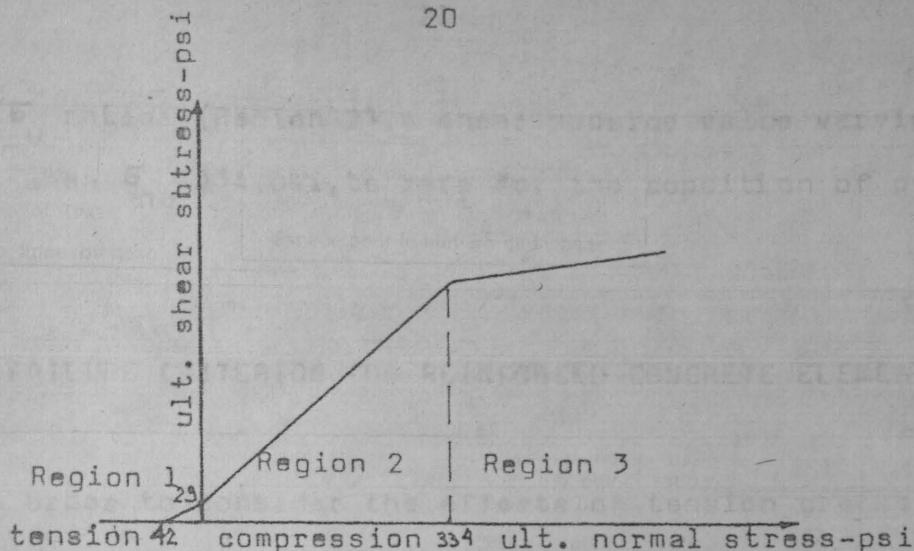


Fig.6. Failure Criterion for Joint Elements (After Page<sup>(3)</sup>)

The relations adopted for the criterion are:

$$\text{Region 1 : } \tau_u = -0.66 \frac{\sigma}{\sigma_u} - 29$$

$$\text{Region 2 : } \tau_u = -0.87 \frac{\sigma}{\sigma_u} - 29$$

$$\text{Region 3 : } \tau_u = -0.11 \frac{\sigma}{\sigma_u} - 277$$

When used in the analytical model, this criterion allows progressive joint failure to occur. If the failure criterion is violated for a joint element, element properties are modified and the problem solved again. The residual properties allocated depend upon the stress state present. If the criterion of Region 1 is violated tensile bond failure is assumed to occur, and no residual capacity is assigned to that element ( $E=G=0$ ). If failure occurs under a combination of compressive and shear stress (Regions 2 and 3) a shear bond failure is simulated. The stiffness of the joint element in the normal direction is assumed to remain unchanged, and reduced shear stiffness is allocated depending upon the magnitude of compressive stress present. When the normal stress is high, some frictional shear capacity remains in the joint after failure, which will diminish as the compressive stress on the joint decreases. Consequently for low  $\frac{\tau}{\sigma}$  ratios (Region 3), a constant residual value for shear modulus  $G$  of 3630 psi is allocated. For

high  $\tau/\sigma_u$  ratios (Region 2), a shear modulus value varying from 3630 psi when  $\sigma_u = 334$  psi, to zero for the condition of pure shear is used.

### 3.2.2 FAILURE CRITERION FOR REINFORCED CONCRETE ELEMENTS

In order to consider the effects of tension cracks in reinforced concrete elements, it is necessary to establish a criterion for the occurrence of tension cracks. According to the criterion used by Zienkiewicz<sup>(6)</sup> regarding rock type materials and which can be employed for concrete also, crack in an element occur perpendicular to the principal directions of the stress tensor, when the value of the principal stress exceeds the uniaxial tensile strength,  $\sigma_{tc}$ , of concrete. Procedure is as follows:

$$\begin{bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{bmatrix} = \{S_c\} = [D_c][G]\{d\}$$

Principal stress vector  $\{S_{cp}\}$  is computed as:

$$\sigma_{1,2} = \frac{\sigma_x + \sigma_y}{2} \pm \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 - \tau_{xy}^2}$$

$$\{S_{cp}\} = \begin{cases} \sigma_1 \text{ (max)} \\ \sigma_2 \text{ (min)} \\ 0 \end{cases}$$

$\theta$ , angle defining the plane of the maximum or minimum normal stress is given by:

$$\tan 2\theta = \frac{2\tau_{xy}}{(\sigma_x - \sigma_y)}$$

In order to find which one of the principal stresses act on the

plane defined by  $\theta, \theta$  is substituted into the equation

$$\sigma' = \frac{\sigma_x - \sigma_y}{2} + \frac{\sigma_x - \sigma_y}{2} \cos 2\theta + \tau_{xy} \sin 2\theta$$

If  $\sigma' = \sigma_1$ , then  $\sigma_1$  acts on the plane defined by  $\theta$ , but

if  $\sigma' = \sigma_2$ , then  $\sigma_1$  acts on the plane defined by  $\theta + 90^\circ$

After finding  $\theta, \sigma_1$  and  $\sigma_2$  are compared with allowable tensile stress  $\sigma_{tc}$ :

If  $\sigma_2 < \sigma_1 < \sigma_{tc}$  no tension crack (case 1)

$\sigma_2 < \sigma_{tc} < \sigma_1$  cracking in one direction (case 2)

$\sigma_{tc} < \sigma_2 < \sigma_1$  cracking in both directions (case 3)

In case that any crack occurs (case 2 or 3) a pseudoload vector due to this crack has to be evaluated and added to the original load vector of the system.

Case 2: Element subdivision cracks only due to  $\sigma_1$  and it is assumed that no stress is taken anymore in that direction. So vector  $\{S_{ccp}\}$  has the form  $\{S_{ccp}\} = \begin{pmatrix} \sigma_1 \\ 0 \\ 0 \end{pmatrix}$ ,  $\{S_{ccp}\}$  representing the released stresses.

Case 3: Element subdivision cracks due to both  $\sigma_1$  and  $\sigma_2$  and it is assumed that no stresses are taken in both principal directions anymore. So vector  $\{S_{ccp}\}$  takes the form  $\{S_{ccp}\} = \begin{pmatrix} \sigma_1 \\ \sigma_2 \\ 0 \end{pmatrix}$ .

In the next step  $\{S_{ccp}\}$  is transformed into global coordinates:

$$\{S_{cc}\} = \begin{pmatrix} S_{ccx} \\ S_{ccy} \\ S_{ccxy} \end{pmatrix} = \begin{bmatrix} \cos^2 \theta & \sin^2 \theta & -2\sin \theta \cos \theta \\ \sin^2 \theta & \cos^2 \theta & 2\sin \theta \cos \theta \\ \sin \theta \cos \theta & -\sin \theta \cos \theta & \cos^2 \theta - \sin^2 \theta \end{bmatrix} \{S_{ccp}\}$$

These stresses are treated as a case of initial stresses at the next iteration. Thus from equation (2.5) the unbalanced forces at nodes adjacent to that element (pseudoloads) due to cracking are calculated as:

$$\{pq_{cc}\} = V [G]^T \{s_{cc}\} \quad (2.14)$$

where

$\{pq_{cc}\}$ : pseudoload vector

$V$  : volume of element subdivision, equal to  $\frac{a \cdot b \cdot T}{9}$

$T$  : wall thickness

### 3.2.3 FAILURE CRITERION FOR BRICK ELEMENTS

Same principles as derived for reinforced concrete elements in chapter 3.2.2 are applicable to brick elements also.

### 3.3 THE NONLINEAR ANALYSIS PROCEDURE

Element types given in chapter 3.1 are incorporated into an incremental finite element program. The procedure used is a step-iteration utilizing a combination of incremental and iterative schemes. The load is applied incrementally, but after each increment successive iterations are performed.

At the  $i^{\text{th}}$  increment, incremental load  $\{\Delta P\}_i$  is applied and the equation of the system

$$[k]_{i-1} \{d\}_i = \{\Delta P\}_i \quad (3.1)$$

is solved for  $\{d\}_i$ , where  $[k]_{i-1}$  is the system stiffness matrix from the previous increment. But because of the nonlinear behaviour of mortar joints and probable tension cracks in reinforced concrete and bricks, the system has not reached an equilibrium under  $\{d\}_i$  yet, that is the applied force  $\{\Delta P\}_i$  is not completely equilibrated due to the nonlinearity of mortar joint elements and due to the unbalanced forces in reinforced concrete and brick

elements in case of cracking.

Regarding mortar joints, stresses in each element are calculated as:

$$\{\epsilon\}_i = \{\epsilon\}_{i-1} + \frac{1}{T} [k_u]_{i-1} [G] \{d\}_i$$

according to equation (2.12). Corrected unit joint stiffness matrix  $[k_u]_i$  corresponding to the stress state is obtained.

The stresses in each reinforced concrete and brick element subdivision is calculated as:

$$\{\epsilon\}_i = [D][G] \left( \sum_{j=1}^i \{d\}_j \right)$$

according to equation (2.4)

If, after checking for cracks, any tension cracks are detected, corresponding pseudoload vector  $\{pq_{cc}\}$  is calculated according to equation (2.14) as:

$$\{pq_{cc}\} = \int [G]^T \{\epsilon\}_i dV \quad (3.2)$$

and this element subdivision is not checked for cracks in the following iterations of this increment.

The system stiffness matrix is rearranged due to the changes in  $[k]_i$ 's of joint elements to obtain  $[K]_i$ . The equilibrated part of  $\{\Delta P\}_i$  can be represented as:

$$\{\Delta P\}_{ib} = [K]_i \{d\}_i \quad (3.3)$$

After placing the pseudoload vectors of individual reinforced concrete and brick elements to their corresponding locations in the system by means of code number technique<sup>(10)</sup>, equation (3.1) can be written as the new equation of the system as follows:

$$[K]_i \{\Delta d\}_i = \underbrace{\{\Delta P\}_i - \{\Delta P\}_{ib}}_I + \underbrace{\{pq_{cc}\}}_{II}$$

the increment of the system. For a detailed description of the solution procedure see Chapter 3.2.1.

$$\underbrace{\{\Delta P\}_i - [K]_i \{d\}_i}_{\text{part I}} + \underbrace{\{p_{qc}\}_i}_{\text{part II}} = 0 \quad (3.4)$$

where part I represents the unbalanced force vector due to material nonlinearities in joint elements and part II represents the unbalanced forces due to cracking in reinforced concrete and brick elements.

After the necessary arrangement of equation (3.4), it takes the form

$$[K]_i (\{d\}_i + \{\Delta d\}_i) = \{\Delta P\}_i + \{p_{qc}\}_i$$

which means solving equation (3.1) with corrected system stiffness matrix and considering the effects of tension cracks, to obtain a new  $\{d\}_i$ . This process continues until

$$\frac{\|\{d\}_{i,n} - \{d\}_{i,n-1}\|}{\|\{d\}_{i,n}\|} < \delta$$

where  $\delta$  is a sufficiently small number and  $\|\cdot\|$  indicates a suitable norm of vectors.

At this stage joint elements are checked for cracks and residual stiffnesses are allocated as described in chapter 3.2.1 and the procedure upto this point is repeated until no more cracks in joint elements occur. Now the load on the system can be increased by one more increment. At the beginning of every new increment, increased stresses are calculated in already cracked element subdivisions of brick and reinforced concrete elements also, due to the incremented load on the system. In following iterations, however, these element subdivisions shall be skipped, as mentioned before. By this way the already existing pseudoload vectors are corrected according to the increased stresses resulting from

the increased load on the system. (For a detailed flowchart of the solution procedure see Appendix 3.)

### 3.4 FINITE ELEMENT MODEL OF FRAME WITH MASONRY INFILL

The system under consideration is brick masonry panel encased in a reinforced concrete frame, finite element subdivision of which is shown in Fig.7 and Fig.8.

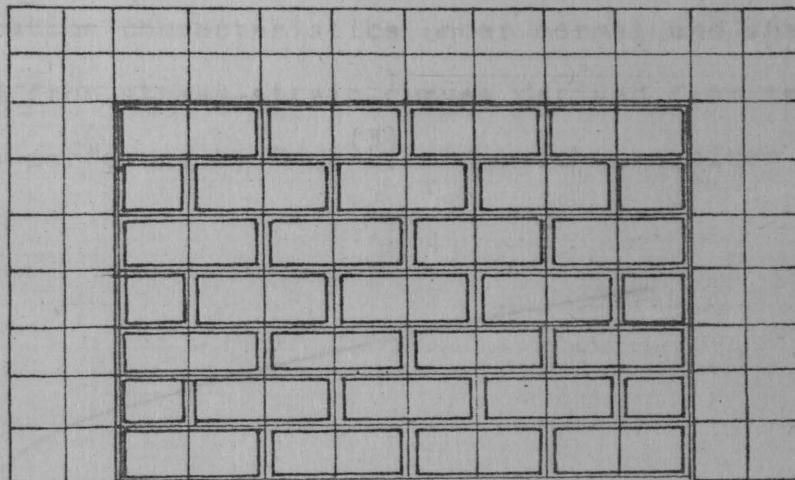


Fig.7. Finite Element Model of Frame-Masonry Panel

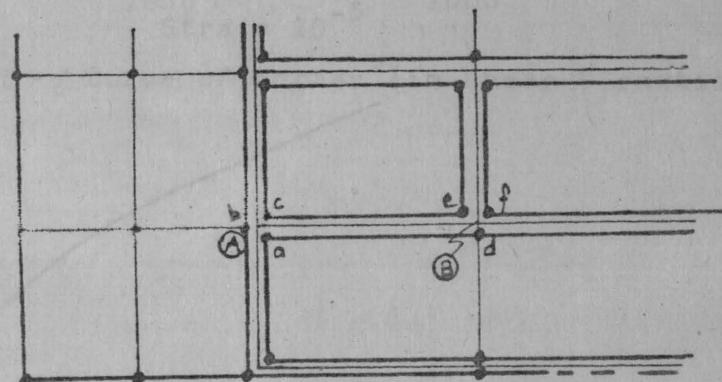


Fig.8. Close-up View of Lower Left Corner of Fig.7

It is assumed that at point A in Fig.8 nodes a,b and c coincide and so do nodes d,e and f at point B, although they are apart by a finite distance in reality. Consequently nodes of joint element AB

are a-c-e-d.

### 3.5 MATERIAL PROPERTIES

Bricks are assumed isotropic, inherent variability of brick properties and small degree of anisotropy is neglected. Brick and reinforced concrete body in equilibrium is elastic only for the uncracked part of the body, and perfect bond exists between the steel and concrete. Joints are assumed to behave nonlinearly and joint deformation characteristics under normal and shear forces are obtained from stress-strain curves derived from tests on masonry panels performed by Page<sup>(3)</sup>, and which are given in Fig.9 and Fig.10.

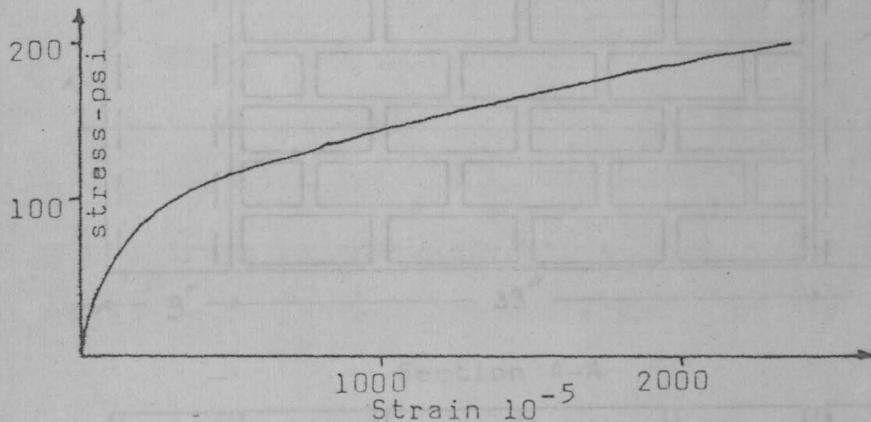


Fig.9.  $\gamma - \delta$  Curve of Mortar (in Shear Direction)

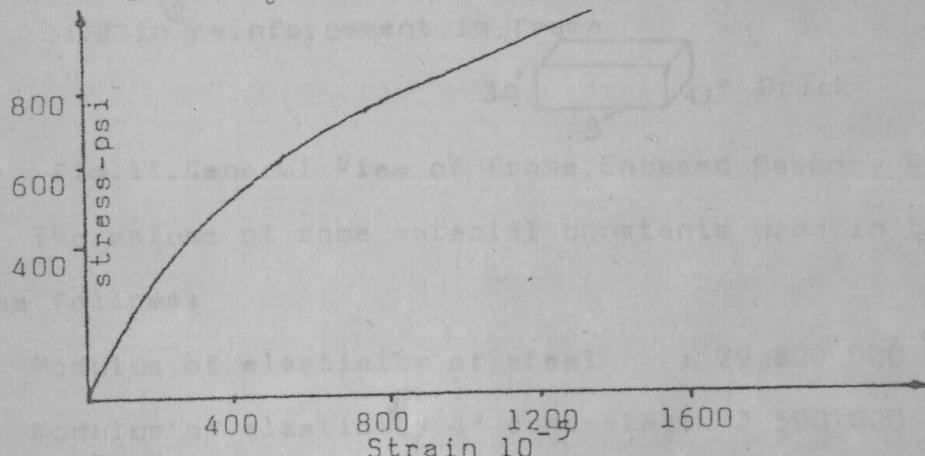


Fig.10.  $\sigma - \epsilon$  Curve of Mortar (in Normal Direction)

## 4. APPLICATIONS AND RESULTS

### 4.1 INTRODUCTION

The structure used in the applications is shown in Fig.11.

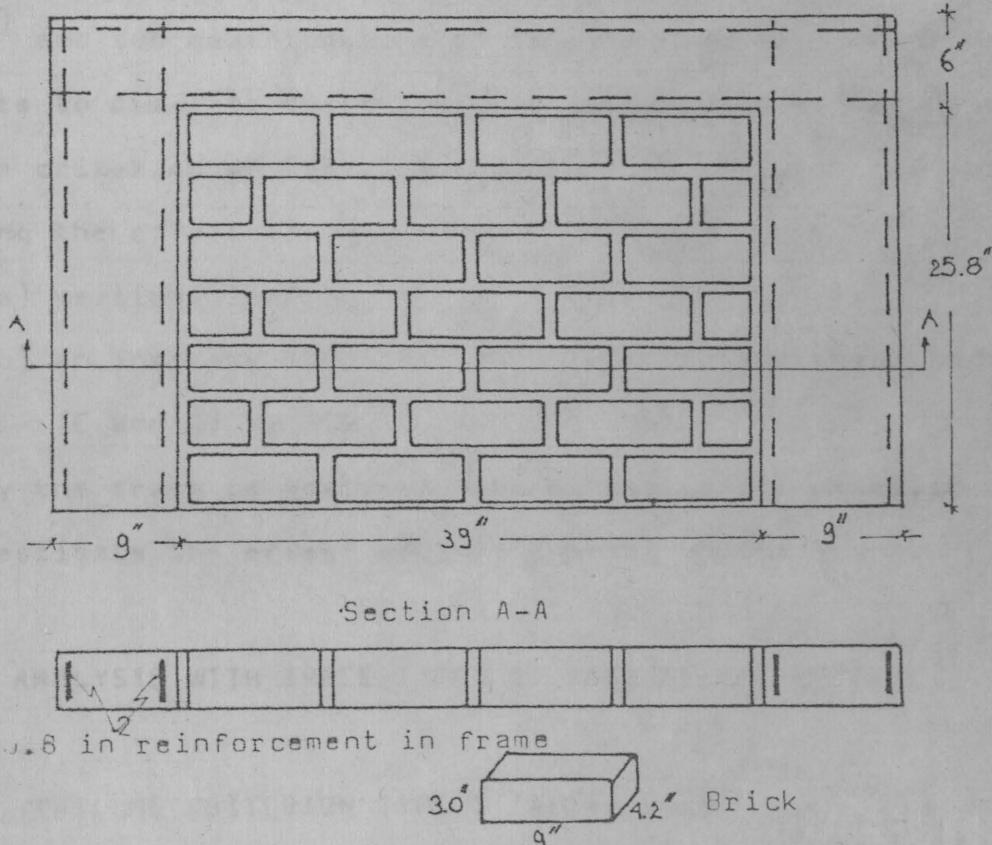


Fig.11.General View of Frame Encased Masonry Panel

The values of some material constants used in the analysis are as follows:

Modulus of elasticity of steel : 29 800 000 psi

Modulus of elasticity of concrete : 3 500 000 psi

Modulus of elasticity of mortar : 292 100 psi

Modulus of elasticity of brick	:	650 000 psi
Shear modulus of mortar	:	128 000 psi
Poisson ratio of concrete	:	0.20
Poisson ratio of brick	:	0.17
Allowable tensile stress of brick <sup>(8,9)</sup>	:	55 psi
Allowable tensile stress of concrete	:	650 psi
Poisson ratio of steel	:	0.17

The incremental finite element program is applied to the structure given in Fig.11. By doing this, failure criterion of Page<sup>(3)</sup> and two modifications of it were used for mortar joint elements to calculate their cracking pattern. Thereafter, using failure criterion of Page, the structure is analysed for investigating the effect of

- a) vertical loading
- b) an increase of mortar joint elastic and shear moduli (E and G) by 50%

Finally the frame is analysed without the infill panel, in order to investigate the effect of infill panel on the frame.

#### 4.2 ANALYSIS WITH THREE TYPES OF FAILURE CRITERIA

##### 4.2.1 FAILURE CRITERION TYPE I (After Page)

The criterion illustrated in Fig.6 is used and the structure is loaded horizontally upto 10000 lb in ten increments. Resulting cracking pattern is illustrated in Fig.12.

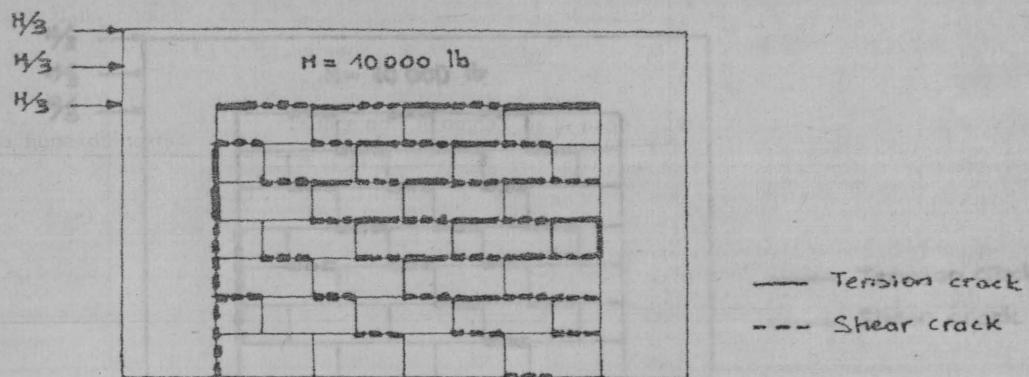


Fig.12.Crack Pattern According to Failure Criterion Type I.

#### 4.2.2 FAILURE CRITERION TYPE II

Failure criterion illustrated in Fig.13 is used.

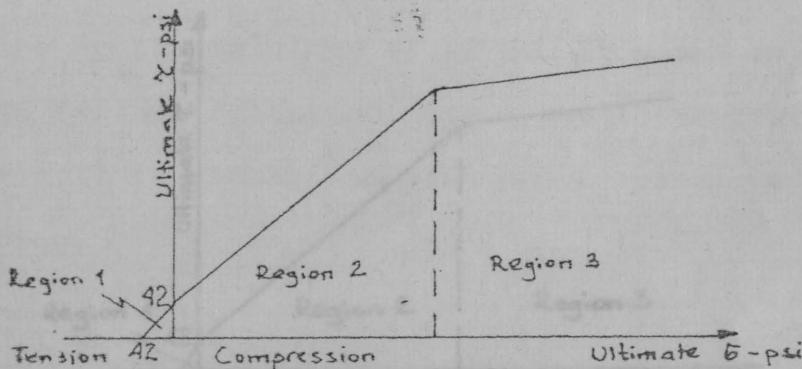


Fig.13.Failure Criterion Type II.

The relations adopted for this criterion are:

$$\text{Region 1: } \tau_u = -1.06 \sigma_{nu} - 42$$

$$\text{Region 2: } \tau_u = -0.83 \sigma_{nu} - 42$$

$$\text{Region 3: } \tau_u = -0.11 \sigma_{nu} - 277$$

When used in the analytical model this criterion yields the crack pattern illustrated in Fig.14. The structure is again loaded with 10 000 lb horizontally in ten increments.

loaded with 10 000 lb horizontally in ten increments.

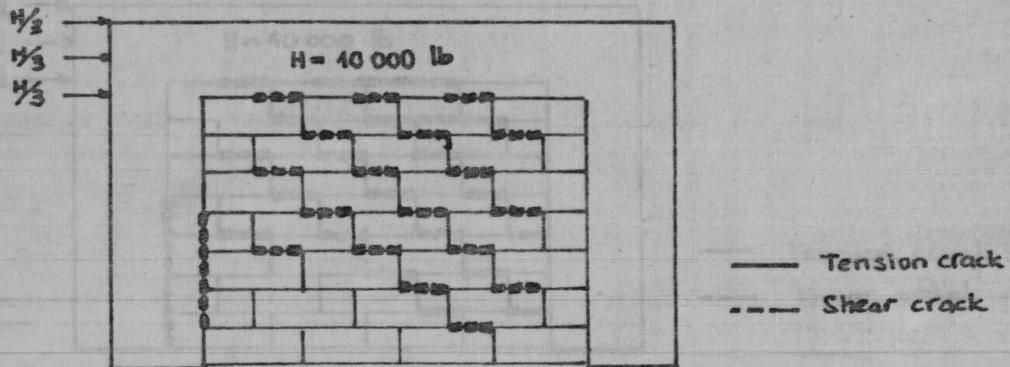


Fig.14. Crack Pattern According to Failure Criterion Type II.

#### 4.2.3 FAILURE CRITERION TYPE III

Failure criterion illustrated in Fig.15 is used.

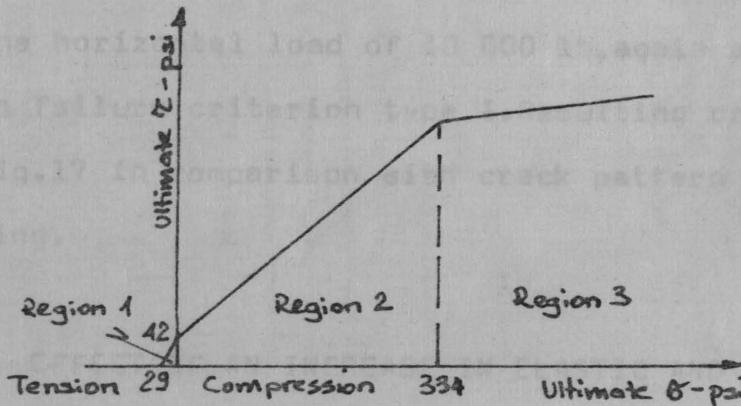


Fig.15. Failure Criterion Type III

The relations adopted for the model are:

$$\text{Region 1: } \tau_u = -1.44 \sigma_{nu} - 42$$

$$\text{Region 2: } \tau_u = -0.83 \sigma_{nu} - 42$$

$$\text{Region 3: } \tau_u = -0.11 \sigma_{nu} - 277$$

When used in the analytical model this criterion yields the crack pattern illustrated in Fig.16. The structure is again loaded with 10 000 lb horizontally in ten increments.

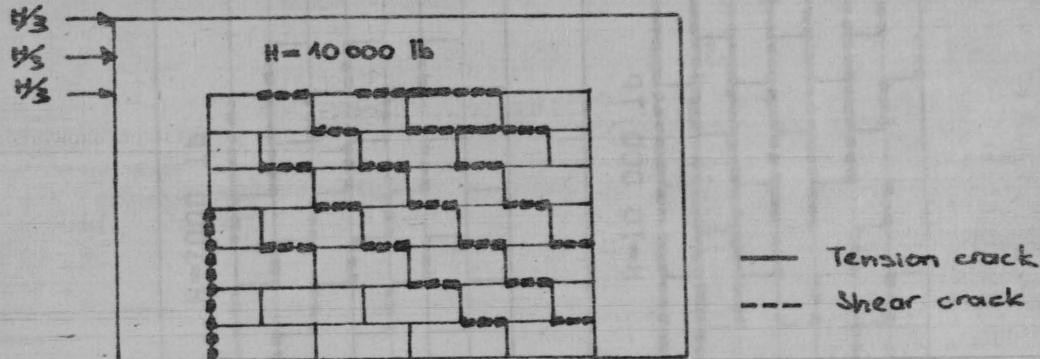


Fig.16.Crack Pattern According to Failure Criterion Type III.

#### 4.3 EFFECT OF VERTICAL LOADING

The structure is loaded vertically with 4550 lb in addition to the horizontal load of 10 000 lb, again at ten increments and using failure criterion type I. Resulting crack pattern is shown in Fig.17 in comparison with crack pattern of only horizontal loading.

#### 4.4 EFFECT OF AN INCREASE IN ELASTIC AND SHEAR MODULI OF MORTAR

In order to find out to which extent a variation in joint deformation characteristics ( $E$  and  $G$ ) effects the behaviour of the masonry infill panel, elastic and shear moduli of mortar joint elements are increased by 50%. Resulting crack pattern of the structure is illustrated in Fig.17 in comparison with crack pattern of the structure with original joint deformation characteristics. A horizontal load of 10 000 lb is applied.

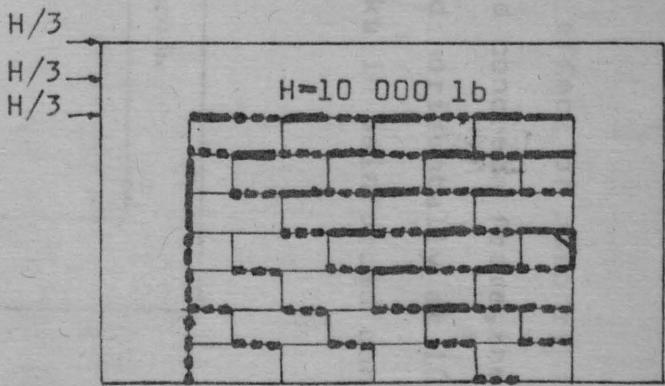
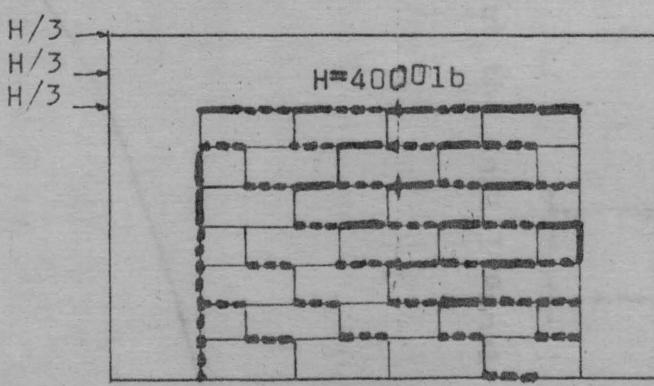
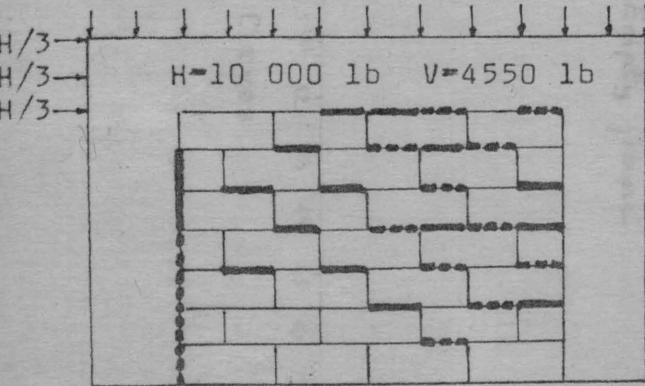
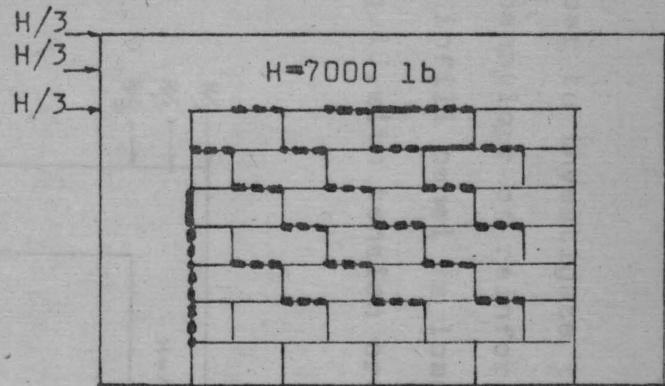
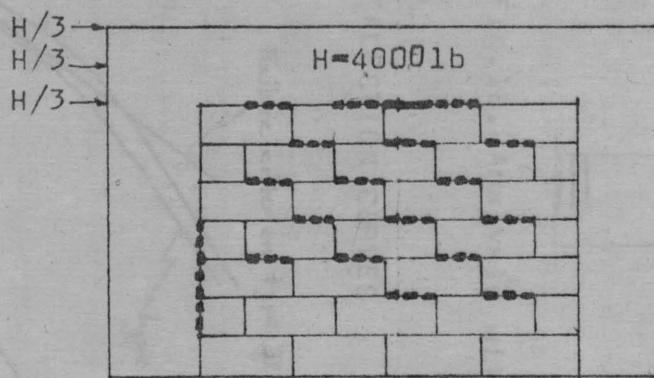
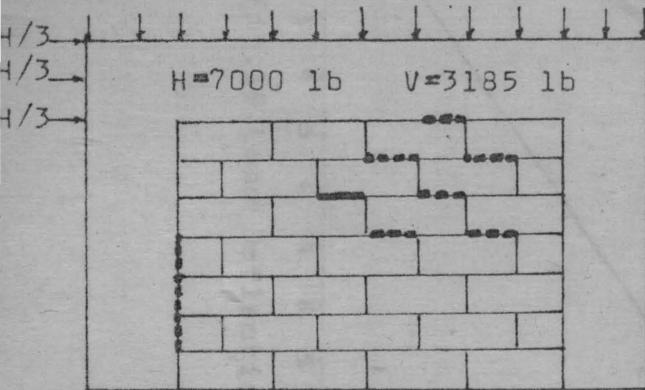
#### 4.5 ANALYSIS OF THE FRAME WITHOUT INFILL PANEL

Fig.17. Effect of Variation in Loading Condition and Material Properties of Mortar

Vertical Compression

Original Case

Increased E and G



In order to investigate the effect of masonry infill panel on the behaviour of reinforced concrete frame, the frame without the infill panel is loaded horizontally at 10 000 lb as shown in Fig.18, with tension cracks in reinforced concrete marked also.

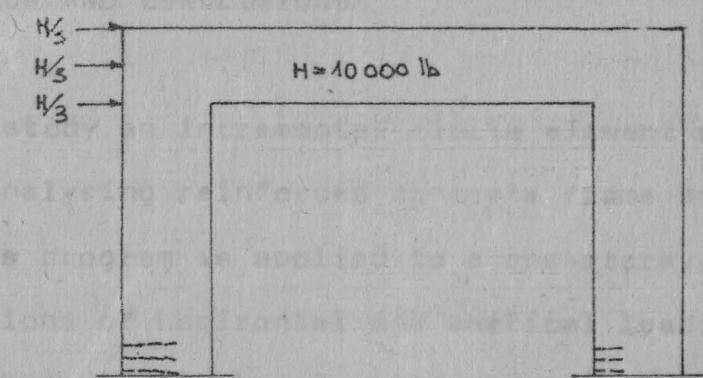


Fig.18. Analysis without the Infill Panel

#### 4.6 LOAD DEFLECTION CURVES

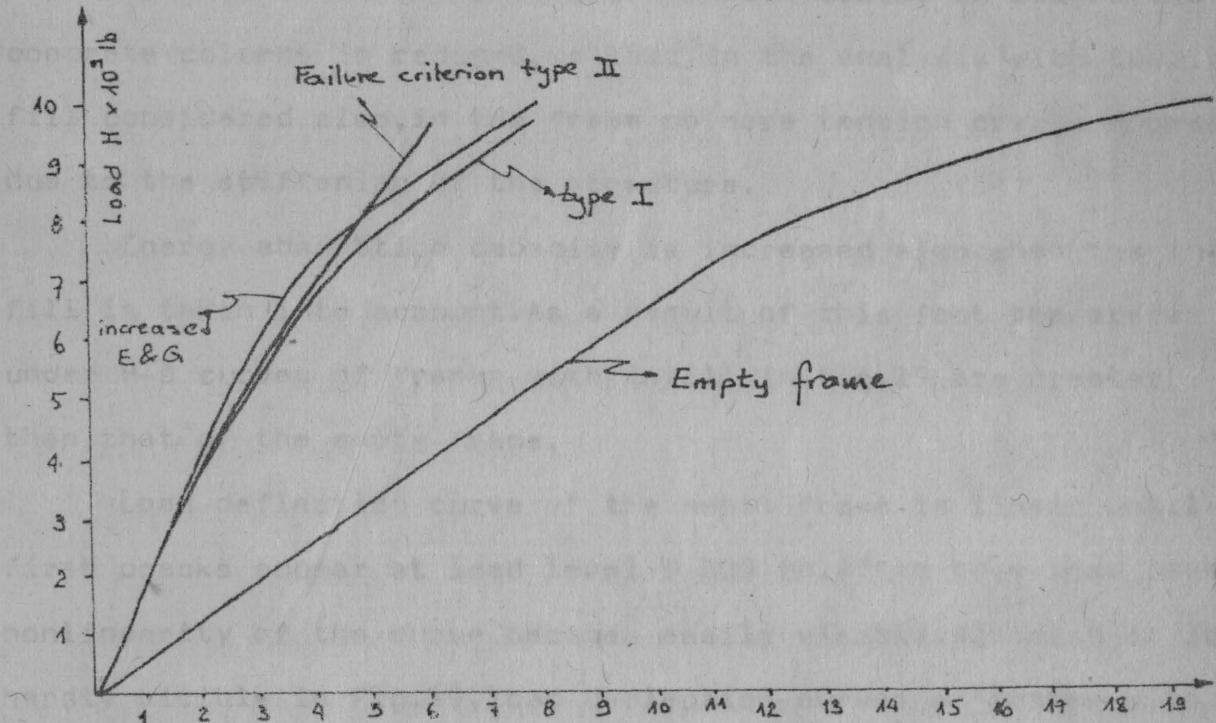


Fig.19. Load Deflection Curves

ising load on the system.

Presence of vertical loading increases the lateral stiffness of the structure. Furthermore, the cracking in mortar joints are reduced (Fig.17). This reduction in cracks is obviously due to vertical compression at horizontal joints sides.

## 5. DISCUSSION AND CONCLUSIONS

Cracking increases the lateral stiffness of the structure.

In this study an incremental finite element program was prepared for analysing reinforced concrete frame encased masonry panels and this program is applied to a one-storey, one-bay frame under combinations of horizontal and vertical loading.

Fig.19 shows that lateral stiffness of reinforced concrete frames is increased by a great amount, if the effect of the infill is incorporated into the calculations. Comparing Fig.17 and Fig.18 it can be seen that the extent of tension cracks in reinforced concrete columns is reduced, so that in the analysis with the infill considered also, in the frame no more tension cracks appear, due to the stiffening of the structure.

Energy absorption capacity is increased also, when the infill is taken into account. As a result of this fact the areas under  $H-\delta$  curves of frames with infill in Fig.19 are greater than that of the empty frame.

Load deflection curve of the empty frame is linear until first cracks appear at load level 8 000 lb. After this load level, nonlinearity of the curve becomes easily visible. Although it is hardly visible in Fig.19, load deflection curves of frames with infill are from the beginning nonlinear due to the nonlinear behaviour of mortar joints. This nonlinearity becomes easily visible as cracking in the mortar joints increase with the increa-

sing load on the system.

Presence of vertical loading increases the lateral stiffness of the structure furthermore, as the cracking in mortar joints are reduced (Fig.17). This reduction in cracks is obvious, because under vertical compression a horizontal joint element needs greater stress values in order to fail. This deletion of cracking increases the lateral stiffness of the structure.

As to be expected, an increase in material properties of mortar joints cause more joint elements to fail, in both tension and shear.

The computer program used demands a size of 97 K, for the problem in chapters 4.2, 4.3 and 4.4, which is a fairly large size and is a consequence of assigning three joints at each node of the infill panel. For the same problem the execution time is approximately 30 minutes, a long but inevitable execution time due to the many successive iterations. The size of the problem could be reduced considerably by making use of symmetry of the structure, if loading were symmetrical also. (e.g. only vertical loading). But in this case, where loading is antisymmetrical, such an application would be very complicated if not impossible.

The conclusions which can be derived are as follows:

1. Masonry infills increase overall stiffness of the structure considerably.
2. Energy absorbtion capacity is increased also.
3. Infill panels carry considerable portion of shear force.
4. Application of the computer program to a frame and infill with larger dimensions would not be practical and also not feasible.
5. Results lead to the conclusion that infills should be incor-

porated in the analysis and design of framed structures, if economy is desired. In case that the infill panel happens to fail in the analysis, this failed panel should be deleted and the analysis repeated.

-0-

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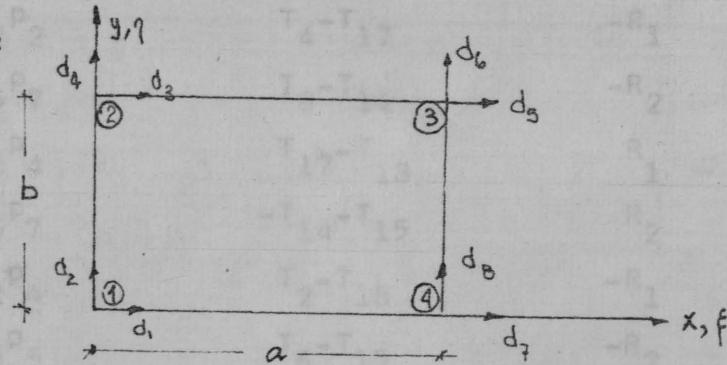
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## APPENDIX 2

## STIFFNESS COEFFICIENTS FROM REINFORCING BARS APPENDIX 1

## RECTANGULAR PLANE STRESS ELEMENT

Explicit forms of strain matrix  $[G]$  and stiffness matrix  $[k]$  for an eight parameter plane stress element in dimensionless coordinates:



$$[G] = \begin{bmatrix} -\frac{1-\eta}{a} & 0 & -\frac{\eta}{a} & 0 & \frac{\eta}{a} & 0 & \frac{1-\xi}{a} & 0 \\ 0 & -\frac{1-\xi}{b} & 0 & \frac{1-\xi}{b} & 0 & \frac{\xi}{b} & 0 & -\frac{\xi}{b} \\ -\frac{1-\xi}{b} & -\frac{1-\eta}{b} & \frac{1-\xi}{b} & -\frac{\eta}{a} & \frac{\xi}{b} & \frac{\eta}{a} & -\frac{\xi}{b} & \frac{1-\eta}{a} \end{bmatrix}$$

$4\beta + 2(1-\nu)\alpha$							
$\frac{3}{2}(1+\nu)$	$4\alpha + 2(1-\nu)\beta$						
$2\beta - 2(1-\nu)\alpha$	$-\frac{3}{2}(1-3\nu)$	$4\beta + 2(1-\nu)\alpha$					
$\frac{3}{2}(1-3\nu)$	$-4\alpha + (1-\nu)\beta$	$-\frac{3}{2}(1+\nu)$	$4\alpha + 2(1+\nu)\beta$				
$-2\beta - (1-\nu)\alpha$	$-\frac{3}{2}(1+\nu)$	$-4\beta + (1+\nu)\alpha$	$-\frac{3}{2}(1-3\nu)$	$4\beta + 2(1-\nu)\alpha$			
$-\frac{3}{2}(1+\nu)$	$-2\alpha - (1-\nu)\beta$	$\frac{3}{2}(1-3\nu)$	$2\alpha - 2(1-\nu)\beta$	$\frac{3}{2}(1+\nu)$	$4\alpha + 2(1-\nu)\beta$		
$-4\beta + (1+\nu)\alpha$	$\frac{3}{2}(1-3\nu)$	$-2\beta - (1-\nu)\alpha$	$\frac{3}{2}(1+\nu)$	$2\beta - 2(1-\nu)\alpha$	$-\frac{3}{2}(1-3\nu)$	$4\beta + 2(1-\nu)\alpha$	
$-\frac{3}{2}(1-3\nu)$	$2\alpha - 2(1-\nu)\beta$	$\frac{3}{2}(1+\nu)$	$-2\alpha - (1-\nu)\beta$	$\frac{3}{2}(1-3\nu)$	$-4\alpha + (1-\nu)\beta$	$-\frac{3}{2}(1+\nu)$	$4\alpha + 2(1-\nu)\beta$

## APPENDIX 2

STIFFNESS COEFFICIENT CONTRIBUTIONS FROM REINFORCING BARS  
OF R.C. ELEMENTS

	$\psi'$	$\psi''$	$\psi'''$
$k_{11}$	$C_1 P_1 - C_2 P_2$	$-T_2 - T_4$	$R_1$
$k_{12}$	$C_3 P_5$	$-T_6 - T_8$	$R_2$
$k_{13}$	$C_1 P_3 - C_2 P_2$	$T_4 - T_{17}$	$-R_1$
$k_{14}$	$-C_4 P_5 - C_5 P_7$	$T_8 - T_{14}$	$-R_2$
$k_{15}$	$-C_1 P_3 - C_2 P_4$	$T_{17} - T_{18}$	$R_1$
$k_{16}$	$-C_4 P_6 - C_5 P_7$	$-T_{14} - T_{15}$	$R_2$
$k_{17}$	$-C_1 P_1 - C_2 P_4$	$T_2 - T_{18}$	$-R_1$
$k_{18}$	$C_4 P_6 - C_5 P_5$	$T_6 - T_{15}$	$-R_2$
$k_{22}$	$C_6 P_2 - C_7 P_1$	$-T_{10} - T_{12}$	$R_3$
$k_{23}$	$C_4 P_7 - C_5 P_5$	$T_8 - T_{13}$	$-R_2$
$k_{24}$	$-C_6 P_2 - C_7 P_3$	$T_{10} - T_{20}$	$-R_3$
$k_{25}$	$-C_4 P_7 - C_5 P_6$	$T_{13} - T_{16}$	$R_2$
$k_{26}$	$-C_6 P_4 - C_7 P_3$	$-T_{19} - T_{20}$	$R_3$
$k_{27}$	$-C_4 P_5 - C_5 P_6$	$T_6 - T_{16}$	$-R_2$
$k_{28}$	$C_6 P_4 - C_7 P_1$	$T_{12} - T_{19}$	$-R_3$
$k_{33}$	$C_1 P_9 - C_2 P_2$	$T_1 - T_4$	$R_1$
$k_{34}$	$-C_3 P_7$	$T_5 - T_8$	$R_2$
$k_{35}$	$-C_1 P_9 - C_2 P_4$	$-T_1 - T_{18}$	$-R_1$
$k_{36}$	$-C_4 P_8 - C_5 P_7$	$-T_5 - T_{15}$	$-R_2$
$k_{37}$	$-C_1 P_3 - C_2 P_4$	$T_{17} - T_{18}$	$R_1$
$k_{38}$	$C_4 P_8 - C_5 P_5$	$T_{13} - T_{15}$	$R_2$
$k_{44}$	$C_6 P_2 - C_7 P_9$	$-T_{10} - T_{11}$	$R_3$
$k_{45}$	$C_4 P_7 - C_5 P_8$	$-T_5 - T_{16}$	$-R_2$
$k_{46}$	$C_6 P_4 - C_7 P_9$	$-T_{11} - T_{19}$	$-R_3$

$k_{47}$	$C_4 P_5 - C_5 P_8$	$-T_{14} - T_{16}$	$R_2$
$k_{48}$	$-C_6 P_4 - C_7 P_3$	$-T_{19} - T_{20}$	$R_3$
$k_{55}$	$C_1 P_9 - C_2 P_{10}$	$T_1 - T_3$	$R_1$
$k_{56}$	$C_3 P_8$	$T_5 - T_7$	$R_2$
$k_{57}$	$C_1 P_3 - C_2 P_{10}$	$-T_3 - T_{17}$	$-R_1$
$k_{58}$	$-C_4 P_8 - C_5 P_6$	$-T_7 - T_{13}$	$-R_2$
$k_{66}$	$C_6 P_{10} - C_7 P_9$	$T_9 - T_{11}$	$R_3$
$k_{67}$	$C_4 P_6 - C_5 P_8$	$-T_7 - T_{14}$	$-R_2$
$k_{68}$	$-C_6 P_{10} - C_7 P_3$	$-T_9 - T_{20}$	$-R_3$
$k_{77}$	$C_1 P_1 - C_2 P_{10}$	$-T_2 - T_3$	$R_1$
$k_{78}$	$-C_3 P_6$	$-T_6 - T_7$	$R_2$
$k_{88}$	$C_6 P_{10} - C_7 P_1$	$T_9 - T_{12}$	$R_3$

$$C_1 = \frac{D_{11}}{\frac{a^2}{2}}$$

$$C_2 = \frac{D_{33}}{b^2}$$

$$C_3 = \frac{D_{12} - D_{33}}{ab}$$

$$C_4 = \frac{D_{12}}{ab}$$

$$C_5 = \frac{D_{33}}{ab}$$

$$C_6 = \frac{D_{22}}{b^2}$$

$$C_7 = \frac{D_{33}}{a^2}$$

$$C = \frac{F_2 - F_1}{L_s}$$

$$S = \frac{n_2 - n_1}{L_s}$$

$$V_s = A_s L_s$$

$$P_1 = \bar{\eta}^2$$

$$P_2 = \bar{\xi}^2$$

$$P_3 = \bar{\eta}\eta_1$$

$$P_4 = \bar{\xi}\xi_1$$

$$P_5 = \bar{\eta}\bar{\xi}$$

$$P_6 = \bar{\eta}\xi_1$$

$$P_7 = \eta_1\bar{\xi}$$

$$P_8 = \eta_1\xi_1$$

$$P_9 = \eta_1^2$$

$$P_{10} = \xi_1^2$$

$$R_1 = \frac{(C_1 S^2 - C_2 C^2) L_s^2}{3}$$

$$R_2 = \frac{C_3 S C L_s^2}{3}$$

$$R_3 = \frac{(C_6 C^2 - C_7 S^2) L_s^2}{3}$$

$$T_1 = C_1 \eta_1 S L_s$$

$$T_2 = C_1 \bar{\eta} S L_s$$

$$T_3 = C_2 \xi_1 C L_s$$

$$T_4 = C_2 \bar{\xi} C L_s$$

$$T_5 = \frac{C_3 \eta_1 C L_s}{2}$$

$$T_6 = \frac{C_3 \bar{\eta} C L_s}{2}$$

$$T_7 = \frac{C_3 \xi_1 S L_s}{2}$$

$$T_8 = \frac{C_3 \bar{\xi} S L_s}{2}$$

$$T_9 = C_6 \xi_1 C L_s$$

$$T_{10} = C_6 \bar{\xi} C L_s$$

$$T_{11} = C_7 \eta_1 S L_s$$

$$T_{12} = C_7 \bar{\eta} S L_s$$

$$T_{13} = \frac{C(C_4 \eta_1 - C_5 \bar{\eta}) L_s}{2}$$

$$T_{14} = \frac{C(C_4 \bar{\eta} - C_5 \eta_1) L_s}{2}$$

$$T_{15} = \frac{s(C_4 \xi_1 - C_5 \bar{\xi}) L_s}{2}$$

$$T_{16} = \frac{s(C_4 \bar{\xi} - C_5 \xi_1) L_s}{2}$$

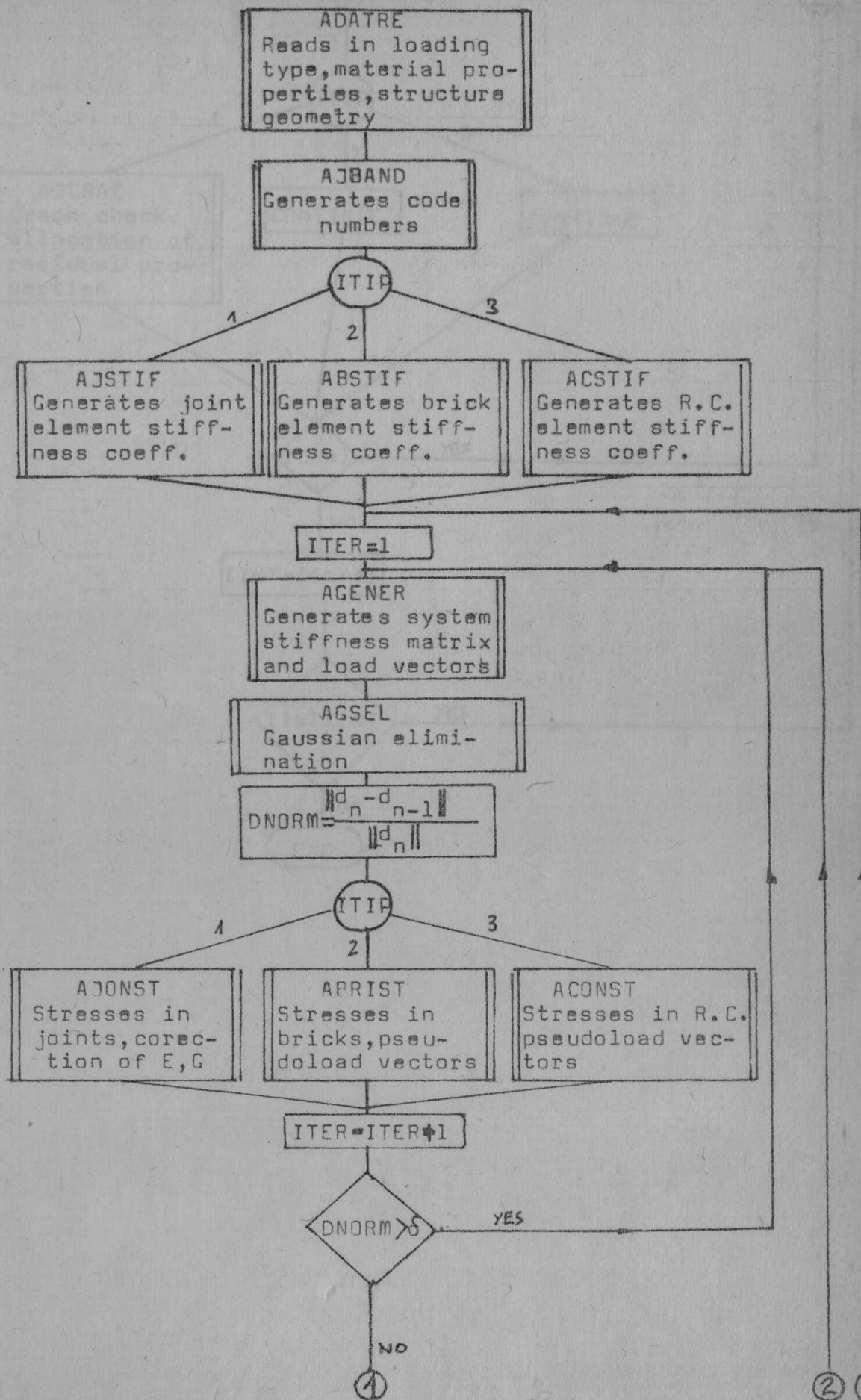
$$T_{17} = \frac{C_1 s(\eta_1 - \bar{\eta}) L_s}{2}$$

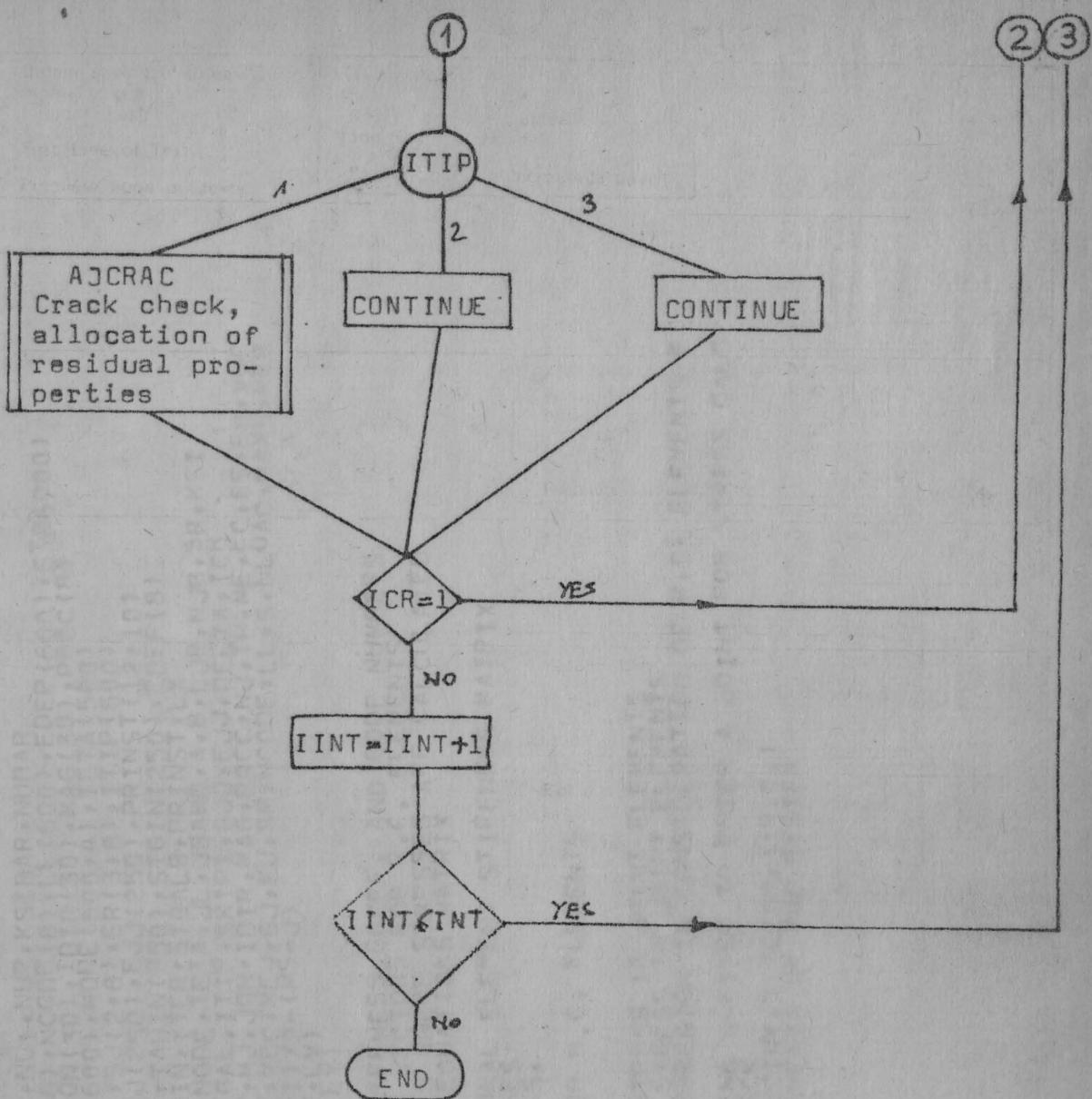
$$T_{18} = \frac{C_2 C(\bar{\xi} - \xi_1) L_s}{2}$$

$$T_{19} = \frac{C_6 C(\bar{\xi} - \xi_1) L_s}{2}$$

$$T_{20} = \frac{C_7 s(\eta_1 - \bar{\eta}) L_s}{2}$$

## APPENDIX 3 . FLOWCHART OF SOLUTION PROCEDURE





THEESIS\*AMEF(1).AMEF

```
1      REAL JL,KS,KN,KSI1,KSI2,NU1,NU2,KSIBAR,NUBAR
2      DIMENSION DEF(600),SM(36),NCODE(8),LL(600),EDEP(600),S(26000)
3      DIMENSION SKIP(110,9),JON(30),TDIR(30),MAG(30),PQCC(8)
4      DIMENSION JOX(600),JOY(600),NODE(500,4),TETA(500)
5      DIMENSION A(500),B(500),SJ(2,8),SR(3,8),ITIP(500)
6      DIMENSION SKIP1(250),GJJ(250),EJJ(250),PRINST(12,10)
7      DIMENSION STRESS(250,2),TAUIN(250),SIGIN(250),EDEF(8)
8      COMMON STRESS,TAUIN,SIGIN,ITER,SIGALB,PRINST,LX
9      COMMON VUS,VUB,JOX,JOY,NODE,TETA,JL,JBAND,A,B,LJB,NJB,SR,KSI
10     COMMON NU,X,SJ,KS,KN,SIGAL,ITIP,SKTP1,GJJ,EJJ,DELTA,TCR
11     COMMON MSS,TJ,EDEP,SKIP,MJ,JON,DIR,MAG,PQCC,NJ,TH,ME,EC,ES,EB,VUC
12     COMMON N,MS,INT,IINT,NC,MEC,MEJ,GJ,EJ,SM,NCODE,LL,S,NLOAD,MAXS,MEB
13     LOC(II,J)=I*MS-III*(II-1)/2-(MS-J)
14     DEFINE FILE 10(600,50,V,LY)
15     DEFINE FILE 29(110,8,V,LY)
```

C MAIN PROGRAM  
C FILE 10 STORES ELEMENT STIFFNESS COEFF. AND CODE NUMBERS  
C FILE 29 STORES PSEUDOLOAD VECTORS FOR R.C. ELEMENTS  
C X: POINT OF JOINT ELEMENTS WHERE STRESSES ARE CALCULATED  
C MAXS: CAPACITY OF SYSTEM EQUATIONS MATRIX  
C IINT: INCREMENT  
C MSS: SIZE OF ONE DIMENSIONAL ELEMENT STIFFNESS MATRIX  
C NLOAD: NO. OF LOADING CASES  
C JBAND: HALF BANDWIDTH OF 'S'  
C NC: NO. OF CORNERS  
C PQCC: PSEUDOLOADVECTOR FOR R.C. ELEMENTS  
C NCODE: CODE NUMBERS  
C N: NO. OF UNKNOWNS  
C TAUIN: CUMULATIVE SHEAR STRESS IN JOINT ELEMENTS  
C SIGIN: CUMULATIVE NORMAL STRESS IN JOINT ELEMENTS  
C SKIP: IF 0, ENTER THE SUBREGION IN CONSIDERATION OF R.C. ELEMENTS F  
C OR CRACK CHECK  
C ITIP1: VARIABLE DETERMINING WHETHER TO ENTER A JOINT FOR STRESS CALC  
C ULATIONS OR FOR CRACK CHECK  
C ITIP: TYPE OF ELEMENT(1:JOINT,2:BRICK,3:R.C.)  
C EDEP: CUMULATIVE DISPLACEMENTS OF THE SYSTEM

X=0  
MAXS=26000  
IINT=1  
MSS=36  
MSE=8  
NLOAD=1  
JBAND=0  
NC=4  
DO 40 I=1,8  
40 PQCC(I)=0.  
DO 50 LE=1,110

```

48      50 WRITE(29,LE)(PQCC(I),I=1,8)
49      CALL ADATRE
50      DO 8 I=1,MEJ
51      TAUIN(I)=0.
52      SIGIN(I)=0.
53      SKIP1(I)=0.
54      DO 10 I=1,MEC
55      DO 20 L=1,9
56      SKIP(I,L)=0. GO TO 1001
57      CONTINUE
58      DO 800 I=1,ME
59      LD=I
60      CALL AJBAND(I),
61      GO TO (801,802,803) ITIP(I)
62      CALL AJSTIF(I)
63      GO TO 800
64      CALL ABSTIF(I)
65      GO TO 800
66      CALL ACSTIF(I)
67      800 WRITE(10,LD)(SM(K),K=1,MSS),(NCODE(M),M=1,MS)
68      DO 23 J=1,N
69      23 EDEP(J)=0.
70      22 PRINT 600,I INT
71      600 FORMAT(/40X,10HINCREMENT ,I2/)
72      DO 25 J=1,N
73      25 DEF(J)=0.
74      27 ITER=1
75      28 DEPNOR=0.
76      DDNOR=0.
77      LX=0
78      PRINT 650,ITER CONVERGENCE ACHIEVED/
79      650 FORMAT(40X,10HITERASYON ,I2)
80      CALL AGENER
81      CALL AGSEL
82      DO 30 J=1,N
83      KELL(J+1)+J
84      DEP=S(K)
85      DD=DEP-DEF(J)
86      DEPNOR=DEPNOR+DEP*MSS,(NCODE(M),M=1,MS)
87      DDNOR=DDNOR+DD*DD
88      30 DEF(J)=DEP
89      DEPNOR=SQRT(DEPNOR)
90      DDNOR=SQRT(DDNOR)
91      DNORM=DDNOR/DEPNOR
92      PRINT 1,DNORM
93      1 FORMAT(40X,F6.4/)
94      DO 900 I=1,ME
95      GO TO (501,502,503) ITIP(I)

```

```

96
97      501 CALL ASJON(I)
98          CALL AJONS(I)
99          GO TO 900
100      502 CALL ABRIST(I)   !STRESSES AT X-SECTION A-A AT THIS LOAD LEVEL /2X,8HF
101          GO TO 900
102      503 CALL ACONST(I)
103          900 CONTINUE
104          ITER=ITER+1
105          IF(ITER.EQ.15) GO TO 1001
106          IF(DNORM.GT.DELTA) GO TO 28
107          ICR=0
108          DO 1000 I=1,ME
109          GO TO (504,505,506) ITIP(I) ! CONVERGENCE?
110      504 CALL ASJON(I)
111          CALL AJCRAC(I)
112          GO TO 1000
113      505 CONTINUE
114          GO TO 1000
115      506 CONTINUE
116      1000 CONTINUE
117          IF(ICR.EQ.1) GO TO 28
118          DO 169 J=1,N
119          EDEF(J)=EDEF(J)+DEF(J)
120          IF(MEJ.EQ.0) GO TO 171
121          DO 170 I=1,MEJ
122              TAUIN(I)=TAUIN(I)+STRESS(I,1)
123          SIGIN(I)=SIGIN(I)+STRESS(I,2)
124      171 CONTINUE
125          INT=INT+1
126          PRINT 2
127          2 FORMAT(36X,21HCONVERGENCE ACCHEVED/)
128          IF(MEC.EQ.0) GO TO 172
129          PRINT 176
130          176 FORMAT(17X,55HDISPLACEMENTS OF CONCRETE ELEMENTS AT THIS LOAD LEVE
131          *L ://2X,7HELM. NO,6X,2HD1,11X,2HD2,11X,2HD3,11X,2HD4,1IX,2HD5,11X,
132          *2HD6,11X,2HD7,11X,2HD8/)
133          LK=MEJ+MEB+1
134          DO 177 LD=LK,ME
135          READ(10,LD)(SM(K),K=1,MSS),(NCODE(M),M=1,MS)
136          DO 9 J=1,8
137          EDEF(J)=0.
138          SAYN=1.
139          IN=NCODE(J)
140          IF(IN) 21,9,26
141          21 IN=-IN
142          SAYN=-1.
143          26 EDEF(J)=EDEF(IN)*SAYN
9 CONTINUE

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144 177 PRINT 11,LB,(EDEF(J),J=1,8)
145 11 FORMAT(4X,I3,8(5X,F8.6))
146 11 PRINT 4
147 4 FORMAT(/17X,44HSTRESSES AT X-SECTION A-A AT THIS LOAD LEVEL/2X,8HE
148 *LM, NO.,2X,5HSIGX2,5X,5HSIGY2,6X,4HTAU2,5X,5HSIGX5,5X,5HSIGY5,6X,4
149 *HTAU5,5X,5HSIGX8,5X,5HSIGY8,6X,4HTAU8/1)
150 DO 29 I=1,12
151 29 PRINT 5,(PRINST(I,J),J=1,10)
152 5 FORMAT(2X,F5.1,9(4X,F6.1))
153 172 IF(IINT.LE.INT) GO TO 22
154 GO TO 1002
155 1001 PRINT 13
156 13 FORMAT(32X,25HCONVERGENCE NOT ACCIEVED/)
157 1002 STOP
158 END

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1. DIRECTIONS  
 2. NO. OF JOINT ELEMENTS  
 3. NO. OF BRICK ELEMENTS  
 4. NO. OF MORTAR ELEMENTS  
 5. TOTAL NO. OF ELEMENTS  
 6. NO. OF JOINTS  
 7. WALL THICKNESS  
 8. JOINT THICKNESS  
 9. NO. OF INCREMENTS  
 10. DEFLAT A SURF WALL NUMBER  
 11. TENSILE STRESS OF CONCRETE  
 12. ELASTIC MODULUS OF MORTAR JOINTS  
 13. SHEAR  
 14. ELASTIC MODULUS OF CONCRETE  
 15. ESR  
 16. ER  
 17. EB  
 18. VUC: POISS. RATIO OF CONCRETE  
 19. VUR: RATIO OF BRICK  
 20. VUSL: RATIO OF STEEL  
 21. JNT: JOINT NO. OF THE EASILY RESTRICTED JOINT  
 22. JNR: JOINT NO. OF RESTRICTED JOINTS  
 23. DX: IF = 0, UNRESTRICTED IN X DIR., OTHERWISE RESTRICTED  
 24. DY: IF = 0, UNRESTRICTED IN Y DIR., OTHERWISE RESTRICTED  
 25. DZ: IF = 0, UNRESTRICTED IN Z DIR., OTHERWISE RESTRICTED  
 26. PRINT 750

750 FORMAT(1HEX,99,9H,OCT DATA/377,8H,16TKT NO.,7X,9H,1PERCENT,5X,9HMAS

141 142 143 144 145 146 147 148 149 150 151 152 153 154 155 156 157 158

140 141 142 143 144 145 146 147 148 149 150 151 152 153 154 155 156 157 158

140 141 142 143 144 145 146 147 148 149 150 151 152 153 154 155 156 157 158

140 141 142 143 144 145 146 147 148 149 150 151 152 153 154 155 156 157 158

140 141 142 143 144 145 146 147 148 149 150 151 152 153 154 155 156 157 158

140 141 142 143 144 145 146 147 148 149 150 151 152 153 154 155 156 157 158

THEISIS\*AMEF(1).ADATRE

SUBROUTINE ADATRE  
1 DIMENSION JOX(600),JOY(600),NODE(500,4),TETA(500)  
2 DIMENSION A(500),B(500),SR(3,8),EDEP(600),SKIP(110,9),JON(30)  
3 DIMENSION IDIR(30),MAG(30),PQCC(8),SM(36),NCODE(8)  
4 DIMENSION LL(600),S(26000),SJ(2,8),ITIP(500)  
5 DIMENSION SKIP1(250),GJJ(250),EJJ(250)  
6 DIMENSION STRESS(250,2),TAUIN(250),SIGIN(250),PRINST(12,10)  
7 COMMON STRESS,TAUIN,SIGIN,ITER,SIGALB,PRINST,LX  
8 COMMON VUS,VUB,JOX,JOY,NODE,TETA,JL,JBAND,A,B,LJB,NJB,SR,KSI  
9 COMMON NU,X,SJ,KS,KN,SIGAL,ITIP,SKTP1,GJJ,EJJ,DELTA,ICR  
10 COMMON MSS,TJ,EDEP,SKIP,MJ,JON,DIR,MAG,PQCC,NJ,TH,ME,EC,ES,EB,VUC  
11 COMMON N,MS,INT,IINT,NC,MEC,MEJ,GJ,EJ,SM,NCODE,LL,S,NLOAD,MAXS,MEB  
12 COMMON MJ: NO. OF NUDAL LOADINGS  
13 C SUBROUTINE FOR READING IN DATA AND EVALUATING SYSTEM GEOMETRY  
14 C MJ: NO. OF NUDAL LOADINGS  
15 C JON: JOINT NO.  
16 C IDIR: DIRECTION  
17 C MAG: MAGNITUDE  
18 C MEJ: NO. OF JOINT ELEMENTS  
19 C MEB: NO. OF BRICK ELEMENTS  
20 C MEC: NO. OF K.C. ELEMENTS  
21 C ME: TOTAL NO. OF ELEMENTS  
22 C NJ: NO. OF JOINTS  
23 C TH: WALL THICKNESS  
24 C TJ: JOINT THICKNESS  
25 C INT: NO. OF INCREMENTS  
26 C DELTA: A SUFF. SMALL NUMBER  
27 C SIGAL: ALL. TENSILE STRESS OF CONCRETE  
28 C EJ: ELASTIC MODULUS OF MORTAR JOINTS  
29 C GJ: SHEAR      Ö      Ö      Ö  
30 C EC: ELASTIC MODULUS OF CONCRETE  
31 C ES:      Ö      Ö      Ö      STEEL  
32 C EB\*:      Ö      Ö      Ö      BRICK  
33 C VUC: POISS. RATIO OF CONCRETE  
34 C VUB:      Ö      Ö      Ö      BRICK  
35 C VUS:      Ö      Ö      Ö      STEEL  
36 C JNN: JOINT NO. OF THE LASTLY RESTRICTED JOINT  
37 C VJN: JOINT NO.,S OF RESTRICTED JOINTS  
38 C DX: IF ,0, UNRESTRICTED IN X DIR., OTHERWISE RESTRICTED  
39 C DY: IF ,0,      Ö      Ö Y      Ö      Ö  
40 C PRINT 750  
41 750 FORMAT(1H1,49X,9HLOAD DATA//37X,8HJOINT NO,7X,9HDIRECTION,5X,9HMAG  
42 \*NITUDE/)  
43 READ 710,MJ  
44 710 FORMAT(I2)  
45 DO 720 I=1,MJ  
46 READ 730,JON(I),IDIR(I),MAG(I)  
47 730 FORMAT(3I10)

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48 72D PRINT 740,JON(I),IDIR(I),MAG(I)
49 740 FORMAT(37X,3I10,2X,2HLB)
50 READ 92,MEC,MEJ,MEB,NJ,TH,TJ
51 MEC=MEC+MEJ+MEB
52 PRINT 93,ME,MEC,MEJ,MEB,NJ,TH,TJ
53 READ 90,INT,DELTA,SIGAL,SIGALB
54 90 FORMAT(I3,3F10.4)
55 PRINT 89,INT,DELTA,SIGAL,SIGALB
56 89 FORMAT(/30X,17HNO. OF INTERVALS=,I3/30X,17HDELTA      =,F5.4/
57 130X,17HSIGMA ALL. OF RC=,F5.1,2X,3HPSI/30X,20HSIGMA ALL. OF BRICK=
58 2,F5.1,2X,3HPSI)
59 READ 96,EJ,GJ,EC,ES,EB,VUC,VUS,VUB
60 PRINT 95,EJ,GJ,EC,ES,EB,VUC,VUS,VUB
61 PRINT 700
62 700 FORMAT(1H1,1X,130(**)/50X,7HRESULTS/1X,130(**)//)
63 DO 5 I=1,MEJ
64 EJJ(I)=EJ
65 GJJ(I)=GJ
66 N=0
67 IJ=1
68 READ 91,JNN
69 91 FORMAT(I5)
70 READ 94,VJN,DX,DY
71 94 FORMAT(3F10.0)
72 JN=VJN
73 IF(JN.EQ.IJ) GO TO 26
74 JND=JN-1
75 DO 10 J=IJ,JND
76 N=N+1
77 JOX(J)=N
78 N=N+1
79 JOY(J)=N
80 10 CONTINUE
81 26 JOX(JN)=0
82 JOY(JN)=0
83 IX=1
84 IX=1
85 IF(DX) 41,43,42
86 41 DX=-DX
87 IX=-1
88 43 N=N+1
89 JOX(JN)=N*IX
90 42 IF(DY) 44,46,51
91 44 DY=-DY
92 IX=-1
93 46 N=N+1
94 JOY(JN)=N*IY
95 51 IJ=JN+1

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86 *AMFF(1),AJPITE{JN:NE:NN) GO TO 70
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      DO 189 J=IJ,NJ
      N=N+1
      JOX(J)=N
      N=N+1
      JOY(J)=N
      CONTINUE
      PRINT 99,N
      99 FORMAT(//40X,16HNO. OF UNKNOWNs=,I4)
      DO 20 I=1,ME
      READ 97,VI,D1,D2,D3,D4,A(I),B(I),TETA(I),ITIP(I)
      97 FORMAT(8F5.0,I5)
      II=VI
      IF(II-I) 31,36,31
      31 PRINT 903,I,II
      903 FORMAT(///30X,7HELEMENT,I6,5X,15HIS OUT OF ORDER,I4/)
      CALL EXIT
      36 NODE(I,1)=D1
      37 NODE(I,2)=D2
      38 NODE(I,3)=D3
      39 NODE(I,4)=D4
      20 CONTINUE
      93 FORMAT(/30X,26HNO. OF ELEMENTS      =,I4/30X,26HNO. OF CONCRETE
      1TE ELEMENTS=,I4/30X,26HNO. OF JOINT ELEMENTS =,I4/30X,27HNO. 0
      2F BRICK ELEMENTS   =,I4/30X,26HNO. OF JOINTS    =,I4/30X,
      326HWALL THICKNESS =,F10.3,3X,2HIN/30X,26HJOINT THICKNESS
      4           =,F10.3,3X,2HIN//)
      96 FORMAT(8F10.0)
      95 FORMAT(/30X,34HMODULUS OF ELASTICITY OF MORTAR =,F10.1,2X,3HPSI/3
      10X,34HSHEAR MODULUS OF MORTAR      =,F10.1,2X,3HPSI/30X,34HMOD
      2ULUS OF ELASTICITY OF CONCRETE=,F10.1,2X,3HPSI/30X,34HMODULUS OF E
      3LASTICITY OF STEEL    =,F10.1,2X,3HPSI/30X,34HMODULUS OF ELASTICITY
      4 OF BRICK     =,F10.1,2X,3HPSI/30X,34HMODULUS OF ELASTICITY
      5           =,F5.3/30X,34HPOISSON RATIO OF CONCRETE =,F5.3/30X,34HP
      6OISSON RATIO OF BRICK =,F5.3)
      92 FORMAT(4I10,2F10.0)
      RETURN
      END

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THEISIS\*AMEF(1).AJBAND

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1      SUBROUTINE AJBAND(I)
2      DIMENSION NODE(500,4),JOX(600),JOY(600),NCODE(8),GJJ(250),EJJ(250)
3      DIMENSION TETA(500),A(500),B(500),SR(3,8),EDEP(600)
4      DIMENSION SKIP(110,9),JON(30),TDIR(30),MAG(30),PQCC(8)
5      DIMENSION LL(600),S(26000),SJ(2,8),SM(36),TTIP(500),SKIP1(250)
6      DIMENSION STRESS(250,2),TAUIN(250),SIGIN(250),PRINST(12,10)
7      COMMON STRESS,TAUIN,SIGIN,ITER,SIGNALB,PRINST,LX
8      COMMON VUS,VUB,JOX,JOY,NODE,TETA,JL,JBAND,A,B,LJB,NJB,SR,KSI
9      COMMON NU,X,SJ,KS,KN,SIGAL,ITIP,SKTP1,GJJ,EJJ,DELTA,ICK
10     COMMON MSS,TJ,EDEP,SKIP,MJ,JON,DIR,MAG,PQCC,NJ,TH,ME,EC,ES,EB,VUC
11     COMMON N,MS,INT,IINT,NC,MEC,MEJ,GJ,EJ,SM,NCODE,LL,S,NLOAD,MAXS,MEB
12     IM=0
13     DO 801 J=1,NC
14     IM=IM+1
15     IN=NODE(I,J)
16     NCODE(IM)=JOX(IN)
17     IM=IM+1
18     NCODE(IM)=JOY(IN) GO TO 101
19 801  CONTINUE
20     MSM=MS-1
21     DO 802 J=1,MSM
22     JP=J+1
23     IJ=NCODE(J)
24     IF(IJ) 12,802,13
25     102  IJ=-IJ
26     103  DO 803 K=JP,MS
27     IK=NCODE(K)
28     IF(IK) 14,803,15
29     14   IK=-IK
30     15   KF=ABS(IK-IJ)+1
31     IF(JBAND-KF) 61,803,803
32     61   JBAND=KF
33     803  CONTINUE
34     802  CONTINUE
35     RETURN
36     END
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THEISIS\*AMEF(1).AJSTIF

```
1      SUBROUTINE AJSTIF(I)
2      REAL JL,KS,KN,K1,K2
3      DIMENSION A(500),B(500),SM(36),TETA(500)
4      DIMENSION JOX(600),JOY(600),NODE(500,4),SR(3,8),EDEP(600)
5      DIMENSION SKTP(110,9),JON(30),IDIR(30),MAG(30)
6      DIMENSION PQCC(8),NCODE(8),LL(600),S(26000)
7      DIMENSION SJ(2,8),ITIP(500),SKIP1(250),GJJ(250),EJJ(250)
8      DIMENSION STRESS(250,2),TAUIN(250),SIGIN(250),PRINST(12,10)
9      COMMON STRESS,TAUIN,SIGIN,ITER,SIGALB,PRINST,LX
10     COMMON VUS,VUB,JOX,JOY,NODE,TETA,JL,JBAND,A,B,LJB,NJB,SR,KSI
11     COMMON NU,X,SJ,KS,KN,SIGAL,ITIP,SKIP1,GJJ,EJJ,DELTA,TCR
12     COMMON MSS,TJ,EDEP,SKIP,MJ,JON,DIR,MAG,PQCC,NJ,TH,ME,EC,ES,EB,VUC
13     COMMON N,MS,INT,IINT,NC,MEC,MEJ,GJ,EJ,SM,NCODE,LL,S,NLOAD,MAXS,MEB
14
15 C STIFFNESS OF JOINT ELEMENTS
16     JL=A(I)
17     KS=GJJ(I)*TH/TJ
18     KN=EJJ(I)*TH/TJ
19     IF(TETA(I)*NE.0.) GO TO 101
20
21     K1=KS
22     K2=KN
23     GO TO 102
24 101   K1=KN
25     K2=KS
26 102   SM(1)=2.*K1*JL/6.
27     SM(2)=0.
28     SM(3)=-2.*K1*JL/6.
29     SM(4)=0.
30     SM(5)=-K1*JL/6.
31     SM(6)=0.
32     SM(7)=K1*JL/6.
33     SM(8)=0.
34     SM(9)=2.*K2*JL/6.
35     SM(10)=0.
36     SM(11)=-2.*K2*JL/6.
37     SM(12)=0.
38     SM(13)=-K2*JL/6.
39     SM(14)=0.
40     SM(15)=K2*JL/6.
41     SM(16)=2.*K1*JL/6.
42     SM(17)=0.
43     SM(18)=K1*JL/6.
44     SM(19)=0.
45     SM(20)=-K1*JL/6.
46     SM(21)=0.
47     SM(22)=2.*K2*JL/6.
        SM(23)=0.
        SM(24)=K2*JL/6.
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48 THE =MEF(1), ABS
49 SM(25)=0.
50 SM(26)=-K2*KL/6.
51 SM(27)=2.*K1*KL/6.
52 SM(28)=0.
53 SM(29)=-2.*K1*KL/6.
54 SM(30)=0.
55 SM(31)=2.*K2*KL/6.
56 SM(32)=0.
57 SM(33)=-2.*K2*KL/6.
58 SM(34)=2.*K1*KL/6.
59 SM(35)=0.
60 SM(36)=2.*K2*KL/6.
61 RETURN
62 C-----C
63 C-----C
64 C-----C
65 C-----C
66 C-----C
67 C-----C
68 C-----C
69 C-----C
70 C-----C
71 C-----C
72 C-----C
73 C-----C
74 C-----C
75 C-----C
76 C-----C
77 C-----C
78 C-----C
79 C-----C
80 C-----C
81 C-----C
82 C-----C
83 C-----C
84 C-----C
85 C-----C
86 C-----C
87 C-----C
88 C-----C
89 C-----C
90 C-----C
91 C-----C
92 C-----C
93 C-----C
94 C-----C
95 C-----C
96 C-----C
97 C-----C

```

THESES\*AMEF(1).ABSTIF  
 SURROUNTING\_ABSTIF(I)  
 1 DIMENSION A(500),B(500),SM(36),TETA(500)  
 2 DIMENSION JOX(600),JOY(600),NODE(500,4),SR(3,8),EDEP(600)  
 3 DIMENSION SKIP(110,9),JON(30),IDIR(30),MAG(30)  
 4 DIMENSION PQCC(8),NCODE(8),LL(600),S(26000)  
 5 DIMENSION SJ(2,8),ITIP(500),SKIP1(250),GJJ(250),EJJ(250)  
 6 DIMENSION STRESS(250,2),TAUIN(250),SIGIN(250),PRINST(12,10)  
 7 COMMON STRESS,TAUIN,SIGIN,ITER,SIGNALB,PRINST,LX  
 8 COMMON VUS,VUB,JOX,JOY,NODE,TETA,JL,JBAND,A,B,LJB,NJB,SR,KSI  
 9 COMMON NU,X,SJ,KS,KN,SIGAL,ITIP,SKIP1,GJJ,EJJ,DELTA,ICK  
 10 COMMON MSS,TJ,EDEP,SKIP,MJ,JON,DIR,MAG,PQCC,NJ,TH,ME,EC,ES,EB,VUC  
 11 COMMON N,MS,INT,IINT,NC,MEC,MEJ,GJ,EJ,SM,NCODE,LL,S,NLOAD,MAXS,MEB  
 12 C STIFFNESS OF BRICK ELEMENTS  
 13 COB=EB\*TH/(12.\*(1.-VUB\*\*2))  
 14 AB=A(I)  
 15 BB=B(I)  
 16 ALFA=AB/BB  
 17 BETA=BB/AB  
 18 SM(1)=(4.\*BETA+2.\*((1.-VUB)\*ALFA))\*COB  
 19 SM(2)=3./2.\*((1.+VUB)\*COB  
 20 SM(3)=(2.\*BETA-2.\*((1.-VUB)\*ALFA))\*COB  
 21 SM(4)=3./2.\*((1.-3.\*VUB)\*COB  
 22 SM(5)=(-2.\*BETA-(1.-VUB)\*ALFA))\*COB  
 23 SM(6)=-3./2.\*((1.+VUB)\*COB  
 24 SM(7)=(-4.\*BETA+(1.-VUB)\*ALFA))\*COB  
 25 SM(8)=-3./2.\*((1.-3.\*VUB)\*COB  
 26 SM(9)=(4.\*ALFA+2.\*((1.-VUB)\*BETA))\*COB  
 27 SM(10)=-3./2.\*((1.-3.\*VUB)\*COB  
 28 SM(11)=(-4.\*ALFA+(1.-VUB)\*BETA))\*COB  
 29 SM(12)=-3./2.\*((1.+VUB)\*COB  
 30 SM(13)=(-2.\*ALFA-(1.-VUB)\*BETA))\*COB  
 31 SM(14)=(3./2.\*((1.-3.\*VUB))\*COB  
 32 SM(15)=(2.\*ALFA-2.\*((1.-VUB)\*BETA))\*COB  
 33 SM(16)=(4.\*BETA+2.\*((1.-VUB)\*ALFA))\*COB  
 34 SM(17)=-3./2.\*((1.+VUB)\*COB  
 35 SM(18)=(-4.\*BETA+(1.-VUB)\*ALFA))\*COB  
 36 SM(19)=3./2.\*((1.-3.\*VUB)\*COB  
 37 SM(20)=(-2.\*BETA-(1.-VUB)\*ALFA))\*COB  
 38 SM(21)=3./2.\*((1.+VUB)\*COB  
 39 SM(22)=(4.\*ALFA+2.\*((1.-VUB)\*BETA))\*COB  
 40 SM(23)=-3./2.\*((1.-3.\*VUB)\*COB  
 41 SM(24)=(2.\*ALFA-2.\*((1.-VUB)\*BETA))\*COB  
 42 SM(25)=3./2.\*((1.+VUB)\*COB  
 43 SM(26)=(-2.\*ALFA-(1.-VUB)\*BETA))\*COB  
 44 SM(27)=(4.\*BETA+2.\*((1.-VUB)\*ALFA))\*COB  
 45 SM(28)=3./2.\*((1.+VUB)\*COB  
 46 SM(29)=(2.\*BETA-2.\*((1.-VUB)\*ALFA))\*COB  
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THE=48+AMER(1)+ACCS
48 SM(30)=3./2.* (1.-3.*VUB)*COB
49 SM(31)=(4.*ALFA+2.* (1.-VUB)*BETA)*COB
50 SM(32)=-3./2.* (1.-3.*VUR)*COB
51 SM(33)=(-4.*ALFA+(1.-VUR)*BETA)*COB (500),KST1(10)
52 SM(34)=(4.*BETA+2.* (1.-VUB)*ALFA)*COB (100),ESV(10),NU1(10),P00C1(10)
53 SM(35)=-3./2.* (1.+VUB)*COB (100),NU2(100),SRCP(100),EDCP(600)
54 SM(36)=(4.*ALFA+2.* (1.-VUB)*BETA)*COB (100),MAG(30)
55 RETURN (200),EJ(250),SKIP1(250)
56 END TENSION (200),EJ(250)
      ITENS=0H,SYSCO1(512),TATNE(256),SIGTN(256),PRINST(112+18)
      COMMON STRESS,TATNE,SIGTN,ITER,SYSCALB,PRINST,4
      COMMON VUS,VUB,VOD,LYR,NUDE,TETA,JL,JBA,ND,A,B,I,JP,NJB,SP,KST
      COMMON NU1,NU2,KST,K1,SYCAL,ITER,SK1P1,G1,I,EJ,DELTA,TETC
      COMMON M551,ITER,SK1P1,H,ITER,MAG,P00C1(10),TRIME,EC,ES,ER,VUE
      COMMON N,MS,KTNI,TINT,INC,PEC,MEJ,GJ,EJ,SM,NCODE,LL,S,NI,UAD,MAXS,NER
      C STIFFNESS OF RC MEMBERS
      COCEFC=TH/12.* (1.-VUC**2)
      DS11EE5=1.-VU5**2
      HS12EV05=DS11
      DS22EDS11
      DS73SES=ITER*(1.-VU5)
      DC11EEC=ITER-VUC**2
      DC12EVU5=DC11
      DC22EDC1
      DC32EEC1/2*(1.-VUC)
      READ 39, NR
      39 FORMAT(1I4)
      IF(NR,0,0) GO TO 13
      READ 30,3K7 TITL,NU1L,PRKST2(1:7),NU2(1:5),SA(1:7),SL(1:7),NR1
      GO FORMAT(4F11.0)
      C READING IN OF INFORMATION DATA
      C FOR NUMBER OF REINFORCING BARS
      C KST1+KST2+NU1+NU2 ARE COORDINATES OF STEEL BARS
      C SA STEEL AREA
      C SL STEEL LENGTH
      C SV STEEL VOLUME
      C C1=DS11*0.001192477*100
      C2=DS33=0.332/PI*1.572
      C3=DS12=0.1345734533/(ALFA*BETA)
      C4=DS12=0.1345734533/(ALFA*BETA)
      C5=DS33=0.1345734533/(ALFA*BETA)
      C6=DS22=0.1345734533
      C7=DS34=0.1345734533
      ALFA=ALFA(1)
      DISTA=DISTA(1)
      C STIFFNESS COEFF. OF CONCRETE PART OF THE RC ELEMENTS
      S1(1)=4.*ALFA,2.* (1.-VUC)*ALFA1*COB
      S1(2)=3./2.* (1.-VUC)*COB

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THEISIS\*AMEF(1).ACSTIF

1 SUBROUTINE ACSTIF(I) \*VUC\*ALFA)\*COC  
2 REAL KSI1,KSI2,NU1,NU2,KSIBAR,NUBAR  
3 DIMENSION A(500),B(500),SM(36),TETA(500),KSI1(10)  
4 DIMENSION KSI2(10),NU2(10),SL(10),SA(10),SV(10),NU1(10),PQCC(8)  
5 DIMENSION JOX(600),JOY(600),NODE(500,4),SR(3,8),EDEP(600)  
6 DIMENSION SKIP(110,9),JON(30),IDIR(30),MAG(30)  
7 DIMENSION NCODE(8),LL(600),S(26000),SJ(2,8),ITIP(500),SKIP1(250)  
8 DIMENSION GJJ(250),EJJ(250)  
9 DIMENSION STRESS(250,2),TAUIN(250),SIGIN(250),PRINST(12,10)

10 COMMON STRESS, TAUIN, SIGIN, ITER, SIGNALB, PRINST, LX  
11 COMMON VUS, VUB, JOX, JOY, NODE, TETA, JL, JBAND, A, B, LJB, NJB, SR, KSI  
12 COMMON NU, X, SJ, KS, KN, SIGNAL, ITIP, SKIP1, GJJ, EJJ, DELTA, ICR  
13 COMMON MSS, TJ, EDEP, SKIP, MJ, JON, IDIR, MAG, PQCC, NJ, TH, ME, EC, ES, EB, VUC  
14 COMMON N, MS, INT, TINT, NC, MEC, MEJ, GJ, EJ, SM, NCODE, LL, S, NLLOAD, MAXS, MEB

C STIFFNESS OF R.C. MEMBERS

15 COC=EC\*TH/(12.\*(1.-VUC\*\*2))

16 DS11=ES/(1.-VUS\*\*2)

17 DS12=VUS\*DS11

18 DS22=DS11

19 DS33=ES/(2.\*(1.+VUS))

20 DC11=EC/(1.-VUC\*\*2)

21 DC12=VUC\*DC11

22 DC22=DC11

23 DC33=EC/(2.\*(1.+VUC))

24 READ 89, NR

25 89 FORMAT(15)

26 IF(NR.EQ.0) GO TO 13

27 READ 90, (KSI1(L),NU1(L),KSI2(L),NU2(L),SA(L),SL(L),L=1, NR)

28 90 FORMAT(6F10.0)

29 READING IN REINFORCEMENT DATA

30 NR: NUMBER OF REINFORCING BARS

31 KSI1,KSI2,NU1,NU2 ARE COORDINATES OF STEEL BARS

32 SA:STEEL AREA

33 SL:STEEL LENGTH

34 SV:STEEL VOLUME

35 13 C1=(DS11-DC11)/A(I)\*\*2.

36 C2=(DS33-DC33)/B(I)\*\*2.

37 C3=(DS12-DC12+DS33-DC33)/(A(I)\*B(I))

38 C4=(DS12-DC12)/(A(I)\*B(I))

39 C5=(DS33-DC33)/(A(I)\*B(I))

40 C6=(DS22-DC22)/B(I)\*\*2.

41 C7=(DS33-DC33)/A(I)\*\*2.

42 ALFA=A(I)/B(I)

43 BETA=B(I)/A(I)

44 C STIFFNESS COEFF. OF CONCRETE PART OF THE RC ELEMENTS

45 SM(1)=(4.\*BETA+2.\*(1.-VUC)\*ALFA)\*COC

46 SM(2)=3./2.\*(1.+VUC)\*COC

47

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48      SM(3) = (2.*BETA - 2.* (1.-VUC)*ALFA)*COC
49      SM(4) = 3./2.* (1.-3.*VUC)*COC
50      SM(5) = (-2.*BETA - (1.-VUC)*ALFA)*COC
51      SM(6) = -3./2.* (1.+VUC)*COC
52      SM(7) = (-4.*BETA + (1.-VUC)*ALFA)*COC
53      SM(8) = -3./2.* (1.-3.*VUC)*COC
54      SM(9) = (4.*ALFA + 2.* (1.-VUC)*BETA)*COC
55      SM(10) = -3./2.* (1.-3.*VUC)*COC
56      SM(11) = (-4.*ALFA + (1.-VUC)*BETA)*COC
57      SM(12) = -3./2.* (1.+VUC)*COC
58      SM(13) = (-2.*ALFA - (1.-VUC)*BETA)*COC
59      SM(14) = (3./2.* (1.-3.*VUC))*COC
60      SM(15) = (2.*ALFA - 2.* (1.-VUC)*BETA)*COC
61      SM(16) = (4.*BETA + 2.* (1.-VUC)*ALFA)*COC
62      SM(17) = -3./2.* (1.+VUC)*COC
63      SM(18) = (-4.*BETA + (1.-VUC)*ALFA)*COC
64      SM(19) = 3./2.* (1.-3.*VUC)*COC
65      SM(20) = (-2.*BETA - (1.-VUC)*ALFA)*COC
66      SM(21) = 3./2.* (1.+VUC)*COC
67      SM(22) = (4.*ALFA + 2.* (1.-VUC)*BETA)*COC
68      SM(23) = -3./2.* (1.-3.*VUC)*COC
69      SM(24) = (2.*ALFA - 2.* (1.-VUC)*BETA)*COC
70      SM(25) = 3./2.* (1.+VUC)*COC
71      SM(26) = (-2.*ALFA - (1.-VUC)*BETA)*COC
72      SM(27) = (4.*BETA + 2.* (1.-VUC)*ALFA)*COC
73      SM(28) = 3./2.* (1.+VUC)*COC
74      SM(29) = (2.*BETA - 2.* (1.-VUC)*ALFA)*COC
75      SM(30) = 3./2.* (1.-3.*VUC)*COC
76      SM(31) = (4.*ALFA + 2.* (1.-VUC)*BETA)*COC
77      SM(32) = -3./2.* (1.-3.*VUC)*COC
78      SM(33) = (-4.*ALFA + (1.-VUC)*BETA)*COC
79      SM(34) = (4.*BETA + 2.* (1.-VUC)*ALFA)*COC
80      SM(35) = -3./2.* (1.+VUC)*COC
81      SM(36) = (4.*ALFA + 2.* (1.-VUC)*BETA)*COC

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C STIFFNESS COEFF. CONTRIBUTIONS ARE SUMMED UP

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82 C STIFFNESS COEFF. CONTRIBUTIONS OF REINFORCING BARS
83 DO 21 L=1,NR
84 C=(KSI2(L)-KSI1(L))/SL(L)
85 SS=(NU2(L)-NU1(L))/SL(L)
86 KSIBAR=1.-KSI1(L)
87 NUBAR=1.-NU1(L)
88 SV(L)=SA(L)*SL(L)
89 P1=NUBAR**2
90 P2=KSIBAR**2
91 P3=NUBAR*NU1(L)
92 P4=KSIBAR*KSI1(L)
93 P5=NUBAR*KSIBAR
94 P6=NUBAR*KSI1(L)
95 P7=NU1(L)*KSIBAR

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96
97 P8=NU1{L}*KSI1(L) -(C6*P2+C7*P0-T10+T11+R3)
98 P9=NU1{L}**2 -(C4*P7-C5*P8-T16+T16+R2)
99 P10=KSI1(L)**2 -(C1*SS**2+C2*C**2)*SL(L)**2/3.
100 R1=(C1*SS**2+C2*C**2)*SL(L)**2/3.
101 R2=C3*SS*C*SL(L)**2/3.
102 R3=(C6*C**2+C7*SS**2)*SL(L)**2/3.
103 T1=C1*NU1(L)*SS*SL(L)
104 T2=C1*NUBAR*SS*SL(L)
105 T3=C2*KSI1(L)*C*SL(L)
106 T4=C2*KSIBAR*C*SL(L)
107 T5=C3*NU1(L)*C*SL(L)/2.
108 T6=C3*NUBAR*C*SL(L)/2.
109 T7=C3*KSI1(L)*SS*SL(L)/2.
110 T8=C3*KSIBAR*SS*SL(L)/2.
111 T9=C6*KSI1(L)*C*SL(L)
112 T10=C6*KSIBAR*C*SL(L)
113 T11=C7*NU1(L)*SS*SL(L)
114 T12=C7*NUBAR*SS*SL(L)
115 T13=C*(C4*NU1(L)-C5*NUBAR)*SL(L)/2.
116 T14=C*(C4*NUBAR-C5*NU1(L))*SL(L)/2;
117 T15=SS*(C4*KSI1(L)-C5*KSIBAR)*SL(L)/2;
118 T16=SS*(C4*KSIBAR-C5*KSI1(L))*SL(L)/2;
119 T17=C1*SS*(NU1(L)-NUBAR)*SL(L)/2;
120 T18=C2*C*(KSIBAR-KSI1(L))*SL(L)/2;
121 T19=C6*C*(KSIBAR-KSI1(L))*SL(L)/2;
122 T20=C7*SS*(NU1(L)-NUBAR)*SL(L)/2.
123 C STIFFNESS COEFF. OF CONCRETE AND STEEL PARTS ARE SUMMED UP
124 SM(1)=SM(1)+SV(L)*(C1*P1+C2*P2-T2-T4+R1)
125 SM(2)=SM(2)+SV(L)*(C3*P5-T6-T8+R2)
126 SM(3)=SM(3)+SV(L)*(C1*P3-C2*P2+T4-T17-R1)
127 SM(4)=SM(4)+SV(L)*(-C4*P5+C5*P7+T8+T14-R2)
128 SM(5)=SM(5)+SV(L)*(-C1*P3-C2*P4+T17-T18+R1)
129 SM(6)=SM(6)+SV(L)*(-C4*P6-C5*P7-T14+T15+R2)
130 SM(7)=SM(7)+SV(L)*(-C1*P1+C2*P4+T2+T18-R1)
131 SM(8)=SM(8)+SV(L)*(C4*P6-C5*P5+T6-T15-R2)
132 SM(9)=SM(9)+SV(L)*(C6*P2+C7*P1-T10-T12+R3)
133 SM(10)=SM(10)+SV(L)*(C4*P7-C5*P5+T8-T13-R2)
134 SM(11)=SM(11)+SV(L)*(-C6*P2+C7*P3+T10-T20-R3)
135 SM(12)=SM(12)+SV(L)*(-C4*P7-C5*P6+T13-T16+R2)
136 SM(13)=SM(13)+SV(L)*(-C6*P4-C7*P3-T19+T20+R3)
137 SM(14)=SM(14)+SV(L)*(-C4*P5+C5*P6+T6+T16-R2)
138 SM(15)=SM(15)+SV(L)*(C6*P4-C7*P1+T12+T19-R3)
139 SM(16)=SM(16)+SV(L)*(C1*P9+C2*P2+T1-T4+R1)
140 SM(17)=SM(17)+SV(L)*(-C3*P7+T5-T8+R2)
141 SM(18)=SM(18)+SV(L)*(-C1*P9+C2*P4-T1+T18-R1)
142 SM(19)=SM(19)+SV(L)*(-C4*P8+C5*P7-T5-T15-R2)
143 SM(20)=SM(20)+SV(L)*(-C1*P3-C2*P4+T17-T18+R1)

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SM(22)=SM(22)+SV(L)*(C6*P2+C7*P9-T10+T11+R3)
SM(23)=SM(23)+SV(L)*(C4*P7-C5*P8-T5+T16-R2)
SM(24)=SM(24)+SV(L)*(C6*P4-C7*P9-T11+T19-R3)
SM(25)=SM(25)+SV(L)*(C4*P5+C5*P8-T14-T16+R2)
SM(26)=SM(26)+SV(L)*(-C6*P4-C7*P3-T19+T20+R3)
SM(27)=SM(27)+SV(L)*(C1*P9+C2*P10+T1+T3+R1)
SM(28)=SM(28)+SV(L)*(C3*P8+T5+T7+R2)
SM(29)=SM(29)+SV(L)*(C1*P3-C2*P10-T3-T17-R1)
SM(30)=SM(30)+SV(L)*(-C4*P8+C5*P6-T7-T13-R2)
SM(31)=SM(31)+SV(L)*(C6*P10+C7*P9+T9+T11+R3)
SM(32)=SM(32)+SV(L)*(C4*P6-C5*P8-T7+T14-R2)
SM(33)=SM(33)+SV(L)*(-C6*P10+C7*P3-T9-T20-R3)
SM(34)=SM(34)+SV(L)*(C1*P1+C2*P10-T2+T3+R1)
SM(35)=SM(35)+SV(L)*(-C3*P6-T6+T7+R2)
SM(36)=SM(36)+SV(L)*(C6*P10+C7*P1+T9-T12+R3)
```

21 CONTINUE  
RETURN  
END

APRT,5 AMEF,AGENER,,AGSEL,,ASJON,,AJONST,,AJCRAC,,ASREC . . . . . 3/.1+ISK=ISK

THEISIS\*AMEF(1).AGENER

```
1      SUBROUTINE AGENER
2      DIMENSION JOX(600),JOY(600),NCODE(8),SM(36),S(26000),ITIP(500)
3      DIMENSION MAG(30),JON(30),IDIR(30),SKIP1(250)
4      DIMENSION NODE(500,4),TETA(500),A(500),B(500),SR(3,8)
5      DIMENSION EDEP(600),SKIP(110,9),LL(600),SJ(2,8)
6      DIMENSION PQCC(8),GJJ(250),EJJ(250)
7      DIMENSION STRESS(250,2),TAUIN(250),SIGIN(250),PRINST(12,10)
8      COMMON STRESS,TAUIN,SIGIN,ITER,SIGALB,PRINST,LX
9      COMMON VUS,VUB,JOX,JOY,NODE,TETA,JL,JBAND,A,B,LJB,NJB,SR,KSI
10     COMMON NU,X,SJ,KS,KN,SIGAL,ITIP,SKTP1,GJJ,EJJ,DELTA,TCR
11     COMMON MSS,TJ,EDEP,SKIP,MJ,JON,DIR,MAG,PQCC,NJ,TH,ME,EC,ES,EB,VUC
12     COMMON N,MS,INT,IINT,NC,MEC,MEJ,GJ,EJ,SM,NCODE,LL,S,NLOAD,MAXS,MEB
13
14 C GENERATION OF SYSTEMS EQUATIONS MATRIX (UNIDIMENSIONAL)
15 C WITH CODE NUMBER TECHNIQUE
16 LOC(II,J)=II*MS-II*(II-1)/2-(MS-J)
17 IUCGEN(I)=(I-N+JBAND-1)*(I-N+JBAND)/2
18 NHEP=(N-JBAND)*JBAND+JBAND*(JBAND+1)/2+N*NLOAD
19 IF(NHEP-MAXS) 70,70,71
20 PRINT 72,NHEP,MAXS
21 FORMAT(//36HPROBLEM SIZE TOO LARGE FOR S MATRIX,2I5)
22 PRINT 777
23 FORMAT(1H1)
24 CALL EXIT
25 70 CONTINUE
26 DO 62 I=1,NHEP
27 62 S(I)=0.
28 DO 9 NM=1,ME
29 LD=NM
30 READ (10,LD)(SM(K),K=1,MSS),(NCODE(M),M=1,MS)
31 LJB=NLOAD+JBAND
32 NJB=N-JBAND
33 DO 8 L=1,MS
34 SAYN=1.
35 I=ENCODE(L)
36 IF(I) 20,8,22
37 20 SAYN=-1.
38 I=-I
39 22 CONTINUE
40 IX=(I-1)*LJB-I+1
41 IUC=IUCGEN(I-1)
42 DO 77 M=1,MS
43 SAYN2=1.
44 J=ENCODE(M)
45 IF(J) 30,77,32
46 30 SAYN2=-1.
47 J=-J
48 32 IF(J-I) 77,78,78
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48 78 ID=L  
49 JD=M  
50 IF(L-M) 122,122,123  
51 123 ID=M  
52 JD=L  
53 122 LC=LOC(ID,JD)  
54 LO=IX+J  
55 IF(I-NJB-1) 79,79,80  
56 80 LO=LO-IUC  
57 79 S(LO)=S(LO)+SAYN\*SAYN2\*SM(LC)  
58 77 CONTINUE  
59 8 CONTINUE  
60 9 CONTINUE  
61 DO 700 IK=1,MJ  
62 WI=MAG(IK)/INT  
63 IJ=JON(IK)  
64 ID=IDIR(IK)  
65 GO TO (81,82) ID  
66 81 NUM=JOX(IJ)  
67 GO TO 83  
68 82 NUM=JOY(IJ)  
69 83 SAYN=1.  
70 I=NUM  
71 IF(I) 51,700,52  
72 51 SAYN=-1.  
73 NUM--NUM  
74 52 I=NUM  
75 LO=(I-1)\*LJB+JBAND+1  
76 IF(I-(NJB+1)) 791,791,801  
77 801 LO=LO-IUCGEN(I)  
78 791 S(LO)=S(LO)+SAYN\*WI  
79 700 CONTINUE  
80 MC=MEJ+1  
81 DO 10 MN=MC,ME  
82 LD=MN  
83 LE=LD-MEJ  
84 READ (10,LD)(SM(K),K=1,MSS),(NCODE(M),M=1,MS)  
85 READ(29,LE)(PQCC(I),I=1,8)  
86 DO 45 NA=1,8  
87 NN=NCODE(NA)  
88 IF(NN) 46,45,48  
89 46 SAYN=-1  
90 NN--NN  
91 48 LO=(NN-1)\*LJR+JBAND+1  
92 IF(NN-(NJB+1)) 49,49,55  
93 55 LO=LO-IUCGEN(NN)  
94 49 S(LO)=S(LO)+PQCC(NA)  
95 45 CONTINUE

THE 96 \* AMEF(1), A65  
97 IF (ITER.NE.1) GO TO 10  
98 DO 16 IM=1,8  
99 16 PQCC(IM)=0.  
100 WRITE(29,LE)(PQCC(I),I=1,8),NODE(500,4),TET(500)  
101 10 CONTINUE  
102 RETURN  
END  
NEDT=51(2,4)+EJ(250)  
DIMENSION STRESS(50,21),TAUIN(250),STGTH(250),PRINST(12,10)  
COMMON STRESS,TAUIN,STGTH,ITER,STGALR,PRINST,LX  
COMMON VEL,VUR,DX=DT,NOFF,TEA,A,DRAN,A,B,L,R,H,J,SPIK1  
COMMON NEX,SJ,KS,KN,STEAL,ITIP,SKIP,GSJ,FUD,DELT,A,TCH  
COMMON MSS,TJ,EDEP,SKIP,MJ,ONFIDIR,MAG,PQCC,NJ,TH,HE,FC,FS,FB,VIC  
COMMON L,NU,INT,ITNT,HC,NEC,SC,EJ,ED,SV,NCODE,D,ES,M,DAT,MAXS,VER  
EQUVALENCE (IS+2)  
JB=IRAND  
NC=6-1  
N1=N+1  
N2=N+2  
N3=N+3  
N4=N+4  
N5=N+5  
N6=N+6  
N7=N+7  
N8=N+8  
N9=N+9  
N10=N+10  
N11=N+11  
N12=N+12  
N13=N+13  
N14=N+14  
N15=N+15  
N16=N+16  
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N500=N+500

THEISIS\*AMEF(1).AGSEL

1 SUBROUTINE AGSEL  
2 DIMENSION LL(600),S(26000),Z(26000),ITIP(500),SKIP1(250),GJJ(250)  
3 DIMENSION JOX(600),JOY(600),NODE(500,4),TETA(500)  
4 DIMENSION A(500),B(500),SR(3,8),EDEP(600),SKIP(110,9),JON(30)  
5 DIMENSION IDIR(30),MAG(30),PQCC(8),SM(36),NCODE(8)  
6 DIMENSION SJ(2,8),EJJ(250)  
7 DIMENSION STRESS(250,2),TAUIN(250),SIGIN(250),PRINST(12,10)  
8 COMMON STRESS,TAUIN,SIGIN,ITER,SIGALB,PRINST,LX  
9 COMMON VUS,VUB,JOX,JOY,NODE,TETA,JL,JBAND,A,B,LJB,NJB,SR,KSI  
10 COMMON NU,X,SJ,KS,KN,SIGAL,ITIP,SKIP1,GJJ,EJJ,DELTA,ICK  
11 COMMON MSS,TJ,EDEP,SKIP,MJ,JON,DIR,MAG,PQCC,NJ,TH,ME,EC,ES,EB,VUC  
12 COMMON N,MS,INT,IINT,NC,MEC,MEJ,GJ,EJ,SM,NCODE,LL,S,NLOAD,MAXS,MEB  
13 EQUIVALENCE (S,Z)  
14 JB=JBAND  
15 NE=N-1  
16 N1=N+1  
17 NL=N+NLOAD  
18 NM=NLOAD-1  
19 JBE=JB-1  
20 NF=N-JBE  
21 ND=JBE+NLOAD  
22 LL(1)=0  
23 J2=0  
24 JCORE=0  
25 DO 40 I=1,N  
26 J1=J2+1  
27 IF(I-NF) 41,41,42  
28 41 J2=J1+ND  
29 GO TO 43  
30 42 J2=J1+NL-I  
31 43 DO 1000 J=J1,J2  
32 JCORE=JCOR+1  
33 1000 Z(J)=Z(JCOR)  
34 J3=J2-NLOAD  
35 JA=J3+J1  
36 DO 44 K=J1,J3  
37 J=JA-K  
38 IF(Z(J)) 50,44,50  
39 44 CONTINUE  
40 50 LT=J-J1+1  
41 IF(LT+1-LL(I)) 51,52,52  
42 51 LT=LL(I)-1  
43 52 JT=J3-J1-LT+1  
44 IF(JT) 40,40,55  
45 55 JP=J3+1  
46 DO 56 J=JP,J2  
47 K=J-JT

48 56 Z(K)=Z(J)  
49 J2=J2-JT  
50 40 LL(I+1)=LT  
51 NX=0  
52 DO 7 I=1,N  
53 NX=NX+LL(I+1)+NLOAD  
54 7 LL(I+1)=NX-I  
55 NX=LL(N)+N  
56 NY=N\*NLOAD  
57 NZ=NX+NLOAD  
58 NT=NZ-NY  
59 C ELIMINASYON  
60 DO 10 K=1,NE  
61 NBK=LL(K)  
62 KK=NBK+K  
63 Q=1./Z(KK)  
64 Z(KK)=Q  
65 IB=K+1  
66 K2=LL(IB)+K  
67 IS=K2-NBK-NLOAD  
68 IF(IS-N) 12,11,11  
69 11 IE=N  
70 IS=NL  
71 GO TO 17  
72 12 IE=IS  
73 IF(IE-IE) 22,22,10  
74 22 K1=K2-NM  
75 17 J2=NBK+IS  
76 IN=IS-IE  
77 DO 13 I=IB,IE  
78 KI=NBK+I  
79 IF(Z(KI)) 14,13,14  
80 14 TA=Q\*Z(KI)  
81 IH=LL(I)-NBK  
82 DO 15 KJ=K1,J2  
83 IJ=KJ+IH  
84 Z(IJ)=Z(IJ)-TA\*Z(KJ)  
85 15 CONTINUE  
86 IF(IN) 18,18,13  
87 18 IH=LL(I+1)+I-K2  
88 DO 16 KJ=K1,K2  
89 IJ=KJ+IH  
90 Z(IJ)=Z(IJ)-TA\*Z(KJ)  
91 16 CONTINUE  
92 13 CONTINUE  
93 10 CONTINUE  
94 C YERINE KOYMA  
95 KI=NX+1



THEISIS\*AMEF(1).ASJON

```
1      SUBROUTINE ASJON(I)
2      REAL JL,KS,KN
3      DIMENSION SJ(2,8),PQCC(8),SM(36),NCODE(8),LL(600),S(26000)
4      DIMENSION JOX(600),JOY(600),NODE(500,4),TETA(500),A(500),B(500)
5      DIMENSION SR(3,8),EDEP(600),SKIP(110,9),JON(30),IDIR(30),MAG(30)
6      DIMENSION ITIP(500),SKIP1(250),GJJ(250),EJJ(250)
7      DIMENSION STRESS(250,2),TAUIN(250),SIGIN(250),PRINST(12,10)
8      COMMON STRESS,TAUIN,SIGIN,ITER,SIGALB,PRINST,LX
9      COMMON VUS,VUB,JOX,JOY,NODE,TETA,JL,JBAND,A,B,LJB,NJB,SR,KSI
10     COMMON NU,X,SJ,KS,KN,SIGAL,ITIP,SKTP1,GJJ,EJJ,DELTA,TCR
11     COMMON MSS,TJ,EDEP,SKIP,MJ,JON,DIR,MAG,PQCC,NJ,TH,ME,EC,ES,EP,VUC
12     COMMON N,MS,TNT,IINT,NC,MEC,MEJ,GJ,EJ,SM,NCODE,LL,S,NLOAD,MAXS,MEB
13
14     C   SUBROUTINE FOR DEVELOPING STRESS MATRIX OF JOINT ELEMENTS
15     JL=A(I)
16     AC=1.-2.*X/JL
17     BC=1.+2.*X/JL
18     KS=GJJ(I)*TH/TJ
19     KN=EJJ(I)*TH/TJ
20     SJ(1,1)=-AC/2.*KS
21     SJ(1,2)=0.
22     SJ(1,3)=AC/2.*KS
23     SJ(1,4)=0.
24     SJ(1,5)=BC/2.*KS
25     SJ(1,6)=0.
26     SJ(1,7)=-BC/2.*KS
27     SJ(1,8)=0.
28     SJ(2,1)=0.
29     SJ(2,2)=-AC/2.*KN
30     SJ(2,3)=0.
31     SJ(2,4)=AC/2.*KN
32     SJ(2,5)=0.
33     SJ(2,6)=BC/2.*KN
34     SJ(2,7)=0.
35     SJ(2,8)=-BC/2.*KN
36     RETURN
END
```

201 CONTINUE

DO 202 L=1,2

SUM=L

THEISIS\*AMEF(1).AJONST

```
1      SUBROUTINE AJONST(I)
2      REAL JL,KS,KN
3      DIMENSION SM(36),NCODE(8),EDEF(8),LL(600),SJ(2,8)
4      DIMENSION STRESS(250,2),EDEP(600),S(26000),SEDEF(8),TETA(500)
5      DIMENSION TAUIN(250),SIGIN(250),CEDEF(8),PRINST(12,10)
6      DIMENSION JOX(600),JOY(600),NODE(500,4),A(500),B(500)
7      DIMENSION SR(3,8),SKIP(110,9),JON(30),IDIR(30),MAG(30)
8      DIMENSION PQCC(8),ITIP(500),SKTP1(250),GJJ(250),EJJ(250)
9      COMMON STRESS,TAUIN,SIGIN,ITER,SIGALB,PRINST,LX
10     COMMON VUS,VUB,JOX,JOY,NODE,TETA,JL,JBAND,A,B,LJB,NJB,SR,KSI
11     COMMON NU,X,SJ,KS,KN,SIGAL,ITIP,SKIP1,GJJ,EJJ,DELTA,ICK
12     COMMON MSS,TJ,EDEP,SKIP,MJ,JON,IDIR,MAG,PQCC,NJ,TH,ME,EC,ES,EB,VUC
13     COMMON N,MS,INT,IINT,NC,MEC,MEJ,GJ,EJ,SM,NCODE,LL,S,NLOAD,MAXS,MEB
14
15 C   STRESS CALCULATION IN JOINT ELEMENTS AND REARRANGEMENT OF ELASTIC/10
16 C   AND SHEAR MODULI ACC. TO STRESS STATE PRESENT
17 IF(SKIP1(I).EQ.1.) RETURN
18 LD=I
19 READ (10,LD)(SM(K),K=1,MSS),(NCODE(M),M=1,MS)
20 DO 20 J=1,MS
21 EDEF(J)=0.
22 CEDEF(J)=0.
23 SAYN=1.
24 IN=NCODE(J)
25 IF(IN) 21,20,22
26 IN=-IN
27 SAYN=-1.
28 IX=LL(IN+1)+IN-(NLOAD-1)
29 EDEF(J)=S(IX)*SAYN
30 CEDEF(J)=EDEF(J)+EDEP(IN)*SAYN
31 CONTINUE
32 DO 24 J=1,8
33 SEDEF(J)=EDEF(J)
34
35 C   TRANSFORMATION OF ELEMENT DISPLACEMENTS IF JOINT ELEMENT IS
36 C   IN VERTICAL POSITION
37 IF(TETA(I).EQ.0.) GO TO 201
38 EDEF(1)=SEDEF(2)
39 EDEF(2)=-SEDEF(1)
40 EDEF(3)=SEDEF(4)
41 EDEF(4)=-SEDEF(3)
42 EDEF(5)=SEDEF(6)
43 EDEF(6)=-SEDEF(5)
44 EDEF(7)=SEDEF(8)
45 EDEF(8)=-SEDEF(7)
46
47 201 CONTINUE
48 DO 30 L=1,2
49 SUM=0.
```

```

48 DO 40 M=1,MS
49 40 SUM=SUM+SJ(L,M)*EDEF(M)
50 30 STRESS(I,L)=SUM/TH
51 TAU=TAUIN(I)+STRESS(I,1)
52 SIGMA=SIGIN(I)+STRESS(I,2)
53 TAU=ABS(TAU)
54 IF(SKIP1(I).EQ.0.) GO TO 35
55 C ASSIGNING NEW MATERIAL PROPERTIES TO ALREADY CRACKED JOINTS
56 C ACCORDING TO CALCULATED COMPRESSIVE STRESS
57 IF(SIGMA.LT.(-334.)) GO TO 15
58 GJJ(I)=-SIGMA*3630./334.
59 GO TO 3000
60 15 GJJ(I)=3630.
61 GO TO 3000
62 35 IF(SIGMA.LE.28..AND.SIGMA.GT.(-100.)) EJJ(I)=292100.-ABS(SIGMA)/10
63 *0.*122000.
64 IF(SIGMA.LE.(-100.).AND.SIGMA.GT.(-200.)) EJJ(I)=170100.-ABS(SIGM
65 *A)-100.)/100.*44100.
66 IF(SIGMA.LE.(-200.).AND.SIGMA.GT.(-300.)) EJJ(I)=126000.-ABS(SIGM
67 *A)-200.)/100.*28200.
68 IF(SIGMA.LE.(-300.).AND.SIGMA.GT.(-400.)) EJJ(I)=97800.-ABS(SIGMA
69 *)-300.)/100.*15300.
70 IF(SIGMA.LE.(-400.).AND.SIGMA.GT.(-500.)) EJJ(I)=82500.-ABS(SIGMA
71 *)-400.)/100.*17400.
72 IF(SIGMA.LE.(-500.).AND.SIGMA.GT.(-600.)) EJJ(I)=65100.-ABS(SIGMA
73 *)-500.)/100.*16800.
74 IF(SIGMA.LE.(-600.).AND.SIGMA.GT.(-700.)) EJJ(I)=48300.-ABS(SIGMA
75 *)-600.)/100.*5400.
76 IF(SIGMA.LE.(-700.).AND.SIGMA.GT.(-800.)) EJJ(I)=42900.-ABS(SIGMA
77 *)-700.)/100.*6600.
78 100 IF(SIGMA.LE.(-800.)) EJJ(I)=36300.
79 IF(TAU.LT.25.) GJJ(I)=128000.-TAU/25.*72500.
80 IF(TAU.GE.25..AND.TAU.LT.50.) GJJ(I)=55500.-((TAU-25.)/25.*33300.
81 IF(TAU.GE.50..AND.TAU.LT.75.) GJJ(I)=22200.-((TAU-50.)/25.*8900.
82 IF(TAU.GE.75..AND.TAU.LT.100.) GJJ(I)=13300.-((TAU-75.)/25.*4600.
83 IF(TAU.GE.100..AND.TAU.LT.125.) GJJ(I)=8700.-((TAU-100.)/25.*3500.
84 IF(TAU.GE.125..AND.TAU.LT.150.) GJJ(I)=5200.-((TAU-125.)/25.*1570.
85 200 IF(TAU.GE.125.) GJJ(I)=3630.
86 3000 CALL AJSTIF(I)
87 WRITE(10,LV)(SM(K),K=1,MSS)
88 RETURN
89 END

```

THEESIS\*AMEF(1).AJCRAC  
 1 SUBROUTINE AJCRAC(I)  
 2 REAL JL,KS,KN  
 3 DIMENSION SM(36),NCODE(8),EDEF(8),LL(600),SJ(2,8)  
 4 DIMENSION STRESS(250,2),EDEP(600),S(26000),SEDEF(8),TETA(500)  
 5 DIMENSION JOX(600),JOY(600),NODE(500,4),A(500),B(500)  
 6 DIMENSION SR(3,8),SKIP(110,9),JON(30),IDIR(30),MAG(30)  
 7 DIMENSION PQCC(8),ITIP(500),SKTP1(250),GJJ(250),EJJ(250)  
 8 DIMENSION TAUIN(250),SIGIN(250),PRINST(12,10)  
 9 COMMON STRESS,TAUIN,SIGIN,ITER,SIGNAL,PRINST,LX  
 10 COMMON VUS,VUB,JOX,JOY,NODE,TETA,JL,JBAND,A,B,LJB,NJB,SR,KSI  
 11 COMMON NU,X,SJ,KS,KN,SIGNAL,ITIP,SKIP1,GJJ,EJJ,DELTA,ICR  
 12 COMMON MSS,TJ,EDEP,SKIP,MJ,JON,DIR,MAG,PQCC,NJ,TH,ME,EC,ES,EB,VUC  
 13 COMMON N,MS,INT,TINT,NC,MEC,MEJ,GJ,EJ,SM,NCODE,LL,S,NLOAD,MAXS,MEB  
 14 C SUBROUTINE FOR CHECKING CRACKS IN JOINT ELEMENTS AND ASSIGNING  
 15 C RESIDUAL MATERIAL PROPERTIES IF NECESSARY  
 16 IF(SKIP1(I).NE.0.) RETURN  
 17 LD=I  
 18 TAU=TAUIN(I)+STRESS(I,1)  
 19 SIGMA=SIGIN(I)+STRESS(I,2)  
 20 TAU=ARS(TAU)  
 21 C IREG:REGION OF JOINT FAILURE, IF 1 TENSILE, IF 2 OR 3 SHEAR FAILURE  
 22 IREG=0  
 23 IF(SIGMA.LE.0.) GO TO 100  
 24 IF(SIGMA.LE.28.) GO TO 50  
 25 IREG=1  
 26 GO TO 1000  
 27 50 UTAU=-1.5\*SIGMA+42.  
 28 IF(TAU.GT.UTAU) IREG=1  
 29 GO TO 1000  
 30 100 IF(SIGMA.LT.(-334.)) GO TO 200  
 31 UTAU=-0.75\*SIGMA+42.  
 32 IF(TAU.GT.UTAU) IREG=2  
 33 GO TO 1000  
 34 200 UTAU=-0.11\*SIGMA+254.  
 35 IF(TAU.GT.UTAU) IREG=3  
 36 1000 IF(IREG.EQ.0) RETURN  
 37 ICR=1  
 38 TAU=TAUIN(I)+STRESS(I,1)  
 39 GO TO (210,220,230) IREG  
 40 210 GJJ(I)=0.  
 41 EJJ(I)=0.  
 42 SKIP1(I)=1.  
 43 PRINT 80,I,IREG,TAU,SIGMA  
 44 80 FORMAT(100('\*\*')/50X,18H CRACK IN JOINT ,I3/40X,24H TYPE OF FAILUR  
 45 \*E: TENSILE'6X,8H REGION: ,I1/40X,6H TAU=,F10.2,6X,6HSIGMA=,F10.2/1  
 46 \*00('\*\*'))  
 47 GO TO 3000

48 220 GJJ(I)=SIGMA\*3630./334.  
49 SKIP1(I)=2.  
50 PRINT 90,I,IREG,TAU,SIGMA  
51 90 FORMAT(100(''),/50X,18H, CRACK IN JOINT ,I3/40X,24H TYPE OF FAILURE  
52 \*E: SHEAR '6X,8H REGION: ,I1/40X,6H TAU=F10.2,6X,6H SIGMA=F10.2/1  
53 \*00(''))  
54 GO TO 3000  
55 230 GJJ(I)=3630.  
56 SKIP1(I)=2.  
57 PRINT 90,I,IREG,TAU,SIGMA  
58 GO TO 3000  
59 3000 CALL AJSTIF(I)  
60 WRITE(10,LD)(SM(K),K=1,MSS)  
61 RETURN  
62 END

SUBROUTINE FOR CALCULATING STRESS MATRIX OF P.C. ELEMENTS

SR(1,1)=C1\*(1-NU1/ACT)  
SR(1,2)=C2\*(1-NU1/ACT)  
SR(1,3)=C3\*(1-NU1/ACT)  
SR(1,4)=C4\*(1-NU1/ACT)  
SR(1,5)=C5\*(1-NU1/ACT)  
SR(1,6)=C6\*(1-NU1/ACT)  
SR(1,7)=C7\*(1-NU1/ACT)  
SR(1,8)=C8\*(1-NU1/ACT)  
SR(1,9)=C9\*(1-NU1/ACT)  
SR(1,10)=C10\*(1-NU1/ACT)  
SR(1,11)=C11\*(1-NU1/ACT)  
SR(1,12)=C12\*(1-NU1/ACT)  
SR(1,13)=C13\*(1-NU1/ACT)  
SR(1,14)=C14\*(1-NU1/ACT)  
SR(1,15)=C15\*(1-NU1/ACT)  
SR(1,16)=C16\*(1-NU1/ACT)

PYTHON AMEP, ACONST, MCONST, ABEST,

CONTINUE 001

THESES\*AMEF(1).ASREC

```
1      SUBROUTINE ASREC(I)
2      REAL KSI,NU
3      DIMENSION PQCC(8),SJ(2,8),ITTP(500),L600,A(500),B(500)
4      DIMENSION SKIP1(250),GJJ(250),EJJ(250)
5      DIMENSION A(500),B(500),SR(3,8),SM(36),NCODE(8),LL(600),S(26000)
6      DIMENSION JOX(600),JOY(600),NODE(500,4),TETA(500)
7      DIMENSION EDEP(600),SKIP(110,9),JON(30),IDIR(30),MAG(30)
8      DIMENSION STRESS(250,2),TAUIN(250),SIGTN(250),PRINST(12,10)
9      COMMON STRESS,TAUIN,SIGIN,TTER,SIGALB,PRINST,LX
10     COMMON VUS,VUB,JOX,JOY,NODE,TETA,JL,JBAND,A,B,LJB,NJB,SR,KSI
11     COMMON NU,X,SJ,KS,KN,SIGAL,ITIP,SKIP1,GJJ,EJJ,DELTA,TCR
12     COMMON MSS,TJ,EDEP,SKIP,MJ,JON,DIR,MAG,PQCC,NJ,TH,ME,EC,ES,EB,VUC
13     COMMON N,MS,INT,TINT,NC,MEC,MEJ,GJ,EJ,SM,NCODE,LL,S,NLOAD,MAXS,MEB
14
15     C   SUBROUTINE FOR CALCULATING STRESS MATRIX OF R.C. ELEMENTS
16     C=EC/(1.-VUC*VUC)
17     SR(1,1)=-C*(1.-NU)/A(I)
18     SR(1,2)=-C*VUC*(1.-KSI)/B(I)
19     SR(1,3)=-C*NU/A(I)
20     SR(1,4)=C*VUC*(1.-KSI)/B(I)
21     SR(1,5)=C*NU/A(I)
22     SR(1,6)=C*VUC*KSI/B(I)
23     SR(1,7)=C*(1.-NU)/A(I)
24     SR(1,8)=-C*VUC*KSI/B(I)
25     SR(2,1)=-C*VUC*(1.-NU)/A(I)
26     SR(2,2)=-C*(1.-KSI)/B(I)
27     SR(2,3)=-C*VUC*NU/A(I)
28     SR(2,4)=C*(1.-KSI)/B(I)
29     SR(2,5)=C*VUC*NU/A(I)
30     SR(2,6)=C*KSI/B(I)
31     SR(2,7)=C*VUC*(1.-NU)/A(I)
32     SR(2,8)=-C*KSI/B(I)
33     SR(3,1)=-C*(1.-VUC)*(1.-KSI)/(2.*B(I))
34     SR(3,2)=-C*(1.-VUC)*(1.-NU)/(2.*A(I))
35     SR(3,3)=C*(1.-VUC)*(1.-KSI)/(2.*B(I))
36     SR(3,4)=C*(1.-VUC)*NU/(2.*A(I))
37     SR(3,5)=C*(1.-VUC)*KSI/(2.*B(I))
38     SR(3,6)=C*(1.-VUC)*NU/(2.*A(I))
39     SR(3,7)=C*(1.-VUC)*KSI/(2.*B(I))
40     SR(3,8)=C*(1.-VUC)*(1.-NU)/(2.*A(I))
41
42     RETURN
43
44     END
```

APRT,S AMEF,ACONST,ASBRIC,ABRIST .

EUNITNUC 0001

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THEISIS*AMEF(1).ACONST ACONST(I)
1   SUBROUTINE ACONST(I)
2   REAL KSI,NU
3   DIMENSION SM(36),NCODE(8),EDEF(8),LL(600),A(500),B(500)
4   DIMENSION SR(3,8),SC(3),SCP(3),SCC(3),S(26000),EDEP(600)
5   DIMENSION SKIP(110,9),PQCC(8)
6   DIMENSION JOX(600),JOY(600),NODE(500,4),TETA(500)
7   DIMENSION JON(30),IDIR(30),MAG(30),SJ(2,8),ITIP(500),SKIP1(250)
8   DIMENSION GJJ(250),EJJ(250)
9   DIMENSION STRESS(250,2),TAUIN(250),SIGIN(250),PRINST(12,10)
10  COMMON STRESS,TAUIN,SIGIN,ITER,SIGALB,PRINST,LX
11  COMMON VUS,VUB,JOX,JOY,NODE,TETA,JL,JBAND,A,B,LJB,NJB,SR,KSI
12  COMMON NU,X,SJ,KS,KN,SIGAL,ITIP,SKIP1,GJJ,EJJ,DELTA,TCR
13  COMMON MSS,TJ,EDEP,SKIP,MJ,JON,DIR,MAG,PQCC,NJ,TH,ME,EC,ES,EB,VUC
14  COMMON N,MS,INT,IINT,NC,MEC,MEJ,GJ,EJ,SM,NCODE,LL,S,NLOAD,MAXS,MEB
15  C SUBROUTINE FOR CALCULATING STRESSES IN R.C ELEMENTS,CHECKING
16  C FOR TENSION CRACKS AND ASSIGNING THE ACCORDING PSEUDOLOAD VECTOR
17  C SC:STRESS VECTOR
18  C SCP:PRINCIPAL STRESS VECTOR
19  C TETAP:PRINCIPLE ANGLE
20  C SCC:STRESSES IN CRACKED CONCRETE ELEMENT
21  C IR:NUMBER OF SUBREGION
22  C IUCGEN(NN)=(NN-N+JBAND-1)*(NN-N+JBAND)/2
23  LD=I
24  LE=L-1
25  IF(LD.EQ.161.OR.LD.EQ.170.OR.LD.EQ.201.OR.LD.EQ.208) GO TO 16
26  GO TO 15
27  16 LX=LX+1
28  DL=LD
29  PRINST(LX,1)=DL
30  15 IR=0
31  READ (10,LD)(SM(K),K=1,MSS),(NCODE(M),M=1,MS)
32  DO 20 J=1,MS
33  EDEF(J)=0.
34  SAYN=1.
35  IN=NCODE(J)
36  IF(IN.EQ.21,20,22)
37  21 IN=-IN
38  SAYN=-1.
39  22 IX=LL((IN+1)+IN-(NLOAD-1))
40  EDEF(J)=S(IX)*SAYN+EDEP(IN)*SAYN
41  20 CONTINUE
42  KSI=1./6.
43  DO 30 IJ=1,3
44  NU=1./6.
45  DO 40 IK=1,3
46  IR=IR+1
47  IF(SKIP(LE,IR).NE.0..AND.ITER.NE.1) GO TO 46

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```

48 CALL ASREC(I)
49 DO 50 L=1,3
50 SUM=0.
51 DO 60 M=1,MS
52 SUM=SUM+SR(L,M)*EDEF(M)
53 SC(L)=SUM
54 IF(LD.EQ.161.OR.LD.EQ.170.OR.LD.EQ.201.OR.LD.EQ.208) GO TO 17
55 GO TO 31
56 17 IF(IR.EQ.2.OR.IR.EQ.5.OR.IR.EQ.8) GO TO 18
57 GO TO 31
58 18 IR1=IR+2
59 J=0
60 DO 10 LI=IR,IR1
61 J=J+1
62 10 PRINT(LX,LI)=SC(J)
63 CONTINUE
64 SCP(1)=(SC(1)+SC(2))/2.+SQRT(((SC(1)-SC(2))/2.)**2+SC(3)**2)
65 SCP(2)=(SC(1)+SC(2))/2.-SQRT(((SC(1)-SC(2))/2.)**2+SC(3)**2)
66 SCP(3)=0.
67 TETAP=0.5*ATAN(2.*SC(3)/(SC(1)-SC(2)))
68 PI=ATAN(1.)*4.
69 TOL=0.00001
70 SIGX=(SC(1)+SC(2))/2.+(SC(1)-SC(2))/2.*COS(2.*TETAP)+SC(3)*SIN(2.*TETAP)
71 IF(ABS(SIGX-SCP(1)).LE.TOL) GO TO 25
72 TETAP=TETAP+PI/2.
73 25 IF(SCP(1).LE.SIGAL) GO TO 46
74 DTETAP=TETAP/PI*180.
75 IF(SCP(2).LE.SIGAL) GO TO 80
76 IF(SKIP(LE,IR).EQ.0.) PRINT 1,I,IR,DTETAP,SC(1),SC(2),SC(3)
77 GO TO 100
78 80 SCP(2)=0.
79 IF(SKIP(LE,IR).EQ.0.) PRINT 2,I,IR,DTETAP,SC(1),SC(2),SC(3)
80 1 FORMAT(25X,I3,9H,ELEMAN,T1,24H,BOLGE IKT YONDE CATLAK,5X,
81 113HCATLAK ACISI=F5.0,2X,7HSIGMAX=F6.1,2X,7HSIGMAY=F6.1,2X,4HTAU
82 2=F6.1)
83 2 FORMAT(25X,I3,9H,ELEMAN,T1,24H,BOLGE BIR YONDE CATLAK,5X,
84 113HCATLAK ACISI=F5.0,2X,7HSIGMAX=F6.1,2X,7HSIGMAY=F6.1,2X,4HTAU
85 2=F6.1)
86 100 SCC(1)=SCP(1)*COS(TETAP)*COS(TETAP)+SCP(2)*SIN(TETAP)*SIN(TETAP)
87 SCC(2)=SCP(1)*SIN(TETAP)*STN(TETAP)+SCP(2)*COS(TETAP)*COS(TETAP)
88 SCC(3)=SCP(1)*SIN(TETAP)*COS(TETAP)-SCP(2)*SIN(TETAP)*COS(TETAP)
89 READ(29,LE)(PQCC(KI),KI=1,8)
90 PQCC(1)=PQCC(1)+(-B(I)*(1.-NU)*SCC(1)-A(I)*(1.-KSI)*SCC(3))*TH/9.
91 PQCC(2)=PQCC(2)+(-A(I)*(1.-KSI)*SCC(2)-B(I)*(1.-NU)*SCC(3))*TH/9.
92 PQCC(3)=PQCC(3)+(-B(I)*NU*SCC(1)+A(I)*(1.-KSI)*SCC(3))*TH/9.
93 PQCC(4)=PQCC(4)+(A(I)*(1.-KSI)*SCC(2)-B(I)*KSI*SCC(3))*TH/9.
94 PQCC(5)=PQCC(5)+(B(I)*NU*SCC(1)+A(I)*KSI*SCC(3))*TH/9.

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96 PQCC(6)=PQCC(6)+(A(I)*KSI*SCC(2)+B(I)*NU*SCC(3))*TH/9.
97 PQCC(7)=PQCC(7)+(B(I)*(1.-NU)*SCC(1)-A(I)*KSI*SCC(3))*TH/9.
98 PQCC(8)=PQCC(8)+(-A(I)*KSI*SCC(2)+B(I)*(1.-NU)*SCC(3))*TH/9.
99 WRITE(29,LE)(PQCC(KI),KI=1,8)
100 SKIP(LE,IR)=1.
101 46 NU=NU+1./3.
102 40 CONTINUE
103 30 CONTINUE
104 RETURN
105 END

```

C - SUBROUTINE FOR CALCULATING STRESS MATRIX OF BRICK ELEMENT

```

SR(1,1)=C*(1-V(1)/A(1))
SR(1,2)=C*(1-V(1)/B(1))
SR(1,3)=C*(1-V(1)/D(1))
SR(1,4)=C*(1-V(1)/E(1))
SR(2,1)=C*(1-V(2)/A(1))
SR(2,2)=C*(1-V(2)/B(1))
SR(2,3)=C*(1-V(2)/D(1))
SR(2,4)=C*(1-V(2)/E(1))
SR(3,1)=C*(1-V(3)/A(1))
SR(3,2)=C*(1-V(3)/B(1))
SR(3,3)=C*(1-V(3)/D(1))
SR(3,4)=C*(1-V(3)/E(1))
SR(4,1)=C*(1-V(4)/A(1))
SR(4,2)=C*(1-V(4)/B(1))
SR(4,3)=C*(1-V(4)/D(1))
SR(4,4)=C*(1-V(4)/E(1))
END

```

THEESIS\*AMEF(1).ASBRIC

1           SUBROUTINE ASBRIC(I)  
2           REAL KSI,NU  
3           DIMENSION HQCC(8),SJ(2,8),ITIP(500)  
4           DIMENSION SKTP1(250),GJJ(250),EJJ(250)  
5           DIMENSION A(500),B(500),SR(3,8),SM(36),NCODE(8),LL(600),S(26000)  
6           DIMENSION JOX(600),JOY(600),NODE(500,4),TETA(500)  
7           DIMENSION EDEP(600),SKIP(110,9),JON(30),IDIR(30),MAG(30)  
8           DIMENSION STRESS(250,2),TAUIN(250),STGIN(250),PRINST(12,10)  
9           COMMON STRESS,TAUIN,SIGIN,ITER,SIGALB,PRINST,LX  
10          COMMON VUS,VUB,JOX,JOY,NODE,TETA,JL,JBAND,A,B,LJB,NJB,SR,KSI  
11          COMMON NU,X,SJ,KS,KN,SIGAL,ITIP,SKTP1,GJJ,EJJ,DELTA,ICR  
12          COMMON MSS,TJ,EDEP,SKIP,MJ,JON,DIR,MAG,PQCC,NJ,TH,ME,EC,ES,EB,VUC  
13          COMMON N,MS,INT,IINT,NC,MEC,MEJ,GJ,EJ,SM,NCODE,LL,S,NLOAD,MAXS,MEB  
14          C     SUBROUTINE FOR CALCULATING STRESS MATRIX OF BRICK ELEMENTS  
15          C=EB/(1.-VUB\*VUB)  
16          SR(1,1)=-C\*(1.-NU)/A(I)  
17          SR(1,2)=-C\*VUB\*(1.-KSI)/B(I)  
18          SR(1,3)=-C\*NU/A(I)  
19          SR(1,4)=C\*VUR\*(1.-KSI)/B(I)  
20          SR(1,5)=C\*NU/A(I)  
21          SR(1,6)=C\*VUB\*KSI/B(I)  
22          SR(1,7)=C\*(1.-NU)/A(I)  
23          SR(1,8)=-C\*VUB\*KSI/B(I)  
24          SR(2,1)=-C\*VUB\*(1.-NU)/A(I)  
25          SR(2,2)=-C\*(1.-KSI)/B(I)  
26          SR(2,3)=-C\*VUB\*NU/A(I)  
27          SR(2,4)=C\*(1.-KSI)/B(I)  
28          SR(2,5)=C\*VUB\*NU/A(I)  
29          SR(2,6)=C\*KSI/B(I)  
30          SR(2,7)=C\*VUB\*(1.-NU)/A(I)  
31          SR(2,8)=-C\*KSI/B(I)  
32          SR(3,1)=-C\*(1.-VUB)\*(1.-KSI)/(2.\*B(I))  
33          SR(3,2)=-C\*(1.-VUB)\*(1.-NU)/(2.\*A(I))  
34          SR(3,3)=C\*(1.-VUR)\*(1.-KSI)/(2.\*B(I))  
35          SR(3,4)=-C\*(1.-VUB)\*NU/(2.\*A(I))  
36          SR(3,5)=C\*(1.-VUB)\*KSI/(2.\*B(I))  
37          SR(3,6)=C\*(1.-VUB)\*NU/(2.\*A(I))  
38          SR(3,7)=-C\*(1.-VUB)\*KSI/(2.\*B(I))  
39          SR(3,8)=C\*(1.-VUB)\*(1.-NU)/(2.\*A(I))  
40          RETURN  
41          END

THESES\*AMEF(1).ABRIST

1 SUBROUTINE ABRIST(I)  
2 REAL KSI,NU  
3 DIMENSION SM(36),NCODE(8),EDEF(8),LL(600),A(500),B(500)  
4 DIMENSION SR(3,8),SC(3),SCP(3),SCC(3),S(26000),EDEP(600)  
5 DIMENSION SKIP(110,9),PQCC(8)  
6 DIMENSION JOX(600),JOY(600),NODE(500,4),TETA(500)  
7 DIMENSION JON(30),IDIR(30),MAG(30),SJ(2,8),ITIP(500),SKIP1(250)  
8 DIMENSION GJJ(250),EJJ(250)  
9 DIMENSION STRESS(250,2),TAUIN(250),SIGIN(250),PRINST(12,10)  
10 COMMON STRESS,TAUIN,SIGIN,TTER,SIGNALB,PRINST,LX  
11 COMMON VUS,VUB,JOX,JOY,NODE,TETA,JL,JBAND,A,B,LJB,NJB,SR,KSI  
12 COMMON NU,X,SJ,KS,KN,SIGAL,ITIP,SKTP1,GJJ,FJJ,DELTA,TCR  
13 COMMON MSS,TJ,EDEP,SKIP,MJ,JON,DIR,MAG,PQCC,NJ,TH,ME,EC,ES,EB,VUC  
14 COMMON N,MS,INT,IINT,NC,MEC,MEJ,GJ,EJ,SM,NCODE,LL,S,NLOAD,MAXS,MEB  
15 C SUBROUTINE FOR CALCULATING STRESSES IN BRICK ELEMENTS, CHECKING FOR  
16 C TENSION CRACKS AND ASSIGNING PSEUDOLOAD VECTOR IF NECESSARY  
17 IUCGEN(NN)=(NN-N+JBAND-1)\*(NN-N+JBAND)/2  
18 LD=I  
19 LE=LD-MEJ  
20 IF(LD.EQ.105.OR.LD.EQ.112.OR.LD.EQ.119.OR.LD.EQ.126.OR.LD.EQ.133.0  
21 \*R.LD.EQ.140.OR.LD.EQ.147.OR.LD.EQ.154) GO TO 27  
22 GO TO 15  
23 27 LX=LX+1  
24 DL=LD  
25 PRINST(LX,1)=DL  
26 15 IR=0  
27 READ (10,LU)(SM(K),K=1,MSS),(NCODE(M),M=1,MS)  
28 DO 20 J=1,MS  
29 EDEF(J)=0.  
30 SAYN=1.  
31 IN=NCODE(J)  
32 IF(IN) 21,20,22  
33 21 IN=-IN  
34 SAYN=-1.  
35 22 IX=LL(IN+1)+IN-(NLOAD-1)  
36 EDEF(J)=S(IX)\*SAYN+EDEP(IN)\*SAYN  
37 20 CONTINUE  
38 KSI=1./6.  
39 DO 30 IJ=1,3  
40 NU=1./6.  
41 DO 40 IK=1,3  
42 IR=IR+1  
43 IF(SKIP(LE,IR).NE.0..AND.ITER.NE.1) GO TO 46  
44 CALL ASBRIC(I)  
45 DO 50 L=1,3  
46 SUM=0.  
47 DO 60 M=1,MS

```

48      60 SUM=SUM+SR(L,M)*EDEF(M)
49      50 SC(L)=SUM
50      45 IF(LD.EQ.105.OR.LD.EQ.112.OR.LD.EQ.119.OR.LD.EQ.126.OR.LD.EQ.133.O
51      40 *R.LD.EQ.140.OR.LD.EQ.147.OR.LD.EQ.154) GO TO 28
52      40 GO TO 31
53      28 IF(IR.EQ.2.OR.IR.EQ.5.OR.IR.EQ.8) GO TO 29
54      28 GO TO 31
55      29 IR1=IR+2
56      29 J=0
57      29 DO 10 LI=IR,IR1
58      29 J=J+1
59      10 PRINST(LX,LI)=SC(J)
60      31 CONTINUE
61      31 SCP(1)=(SC(1)+SC(2))/2.+SQRT(((SC(1)-SC(2))/2.)**2+SC(3)**2)
62      31 SCP(2)=(SC(1)+SC(2))/2.-SQRT(((SC(1)-SC(2))/2.)**2+SC(3)**2)
63      31 SCP(3)=0.
64      31 TETAP=0.5*ATAN(2.*SC(3)/(SC(1)-SC(2)))
65      31 PI=ATAN(1.)*4.
66      31 TOL=0.00001
67      31 SIGX=(SC(1)+SC(2))/2.+(SC(1)-SC(2))/2.*COS(2.*TETAP)+SC(3)*SIN(2.*TETAP)
68      31 IF(ABS(SIGX-SCP(1)).LE.TOL) GO TO 25
69      31 TETAP=TETAP+PI/2.
70      25 IF(SCP(1).LE.SIGALB) GO TO 46
71      25 DTETAP=TETAP/PI*180.
72      25 IF(SCP(2).LE.SIGALB) GO TO 80
73      25 IF(SKIP(LE,IR).EQ.0.)PRINT 1,I,IR,DTETAP,SC(1),SC(2),SC(3)
74      25 GO TO 100
75      80 SCP(2)=0.
76      80 IF(SKIP(LE,IR).EQ.0.) PRINT 2,T,IR,DTETAP,SC(1),SC(2),SC(3)
77      1 FORMAT(25X,I3,9H,TUGLA,I1,24H,BOLGE IKI YONDE CATLAK,5X,
78      113HCATLAK ACISI=,F5.0,2X,7HSIGMAX=,F6.1,2X,7HSIGMAY=,F6.1,2X,4HTAU
79      2E,F6.1)
80      2 FORMAT(25X,I3,9H,TUGLA,I1,24H,BOLGE BIR YONDE CATLAK,5X,
81      113HCATLAK ACISI=,F5.0,2X,7HSIGMAX=,F6.1,2X,7HSIGMAY=,F6.1,2X,4HTAU
82      2E,F6.1)
83      100 SCC(1)=SCP(1)*COS(TETAP)*COS(TETAP)+SCP(2)*SIN(TETAP)*SIN(TETAP)
84      100 SCC(2)=SCP(1)*SIN(TETAP)*SIN(TETAP)+SCP(2)*COS(TETAP)*COS(TETAP)
85      100 SCC(3)=SCP(1)*SIN(TETAP)*COS(TETAP)-SCP(2)*SIN(TETAP)*COS(TETAP)
86      100 READ(29,LE)(PQCC(KI),KI=1,8)
87      100 PQCC(1)=PQCC(1)+(-B(I)*(1.-NU)*SCC(1)-A(I)*(1.-KSI)*SCC(3))*TH/9.
88      100 PQCC(2)=PQCC(2)+(-A(I)*(1.-KSI)*SCC(2)-B(I)*(1.-NU)*SCC(3))*TH/9.
89      100 PQCC(3)=PQCC(3)+(-B(I)*NU*SCC(1)+A(I)*(1.-KSI)*SCC(3))*TH/9.
90      100 PQCC(4)=PQCC(4)+(A(I)*(1.-KSI)*SCC(2)-B(I)*KSI*SCC(3))*TH/9.
91      100 PQCC(5)=PQCC(5)+(B(I)*NU*SCC(1)+A(I)*KSI*SCC(3))*TH/9.
92      100 PQCC(6)=PQCC(6)+(A(I)*KSI*SCC(2)+B(I)*NU*SCC(3))*TH/9.
93      100 PQCC(7)=PQCC(7)+(B(I)*(1.-NU)*SCC(1)-A(I)*KSI*SCC(3))*TH/9.
94      100 PQCC(8)=PQCC(8)+(-A(I)*KSI*SCC(2)+B(I)*(1.-NU)*SCC(3))*TH/9.

```

96  
97        WRITE(29,LE)(PQCC(KI),KI=1,8)  
98        SKIP(LE,IR)=1.  
99        46 NU=NU+1./3.  
100        40 CONTINUE  
101        30 KSI=KSI+1./3.  
102        30 CONTINUE  
103        RETURN  
          END

APRT,S AMEF,AMEF,,ADATRE,,AJHAND,,AJSTIF,,ABSTIF,,ACSTIF .      0001 OT UG