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

THESIS

Faik Kıvanç

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Civil Engineering Deparment
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Faik Kıvanç

BSCE

A Thesis

Presented to

the Faculty of the School of Engineering

Boğaziçi University

In Partial Fulfillment

of the Requirements for the Degree of

Master of Science

in

Civil Engineering

February 1982

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OF REINFORCED CONCRETE FRAMES
WITH MASONRY FILLER WALLS

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ACKNOWLEDGEMENT

I would like to express my sincere gratitudes to my thesis supervisor Dr. Hüsamettin Alper for his invaluable guidance and assistance during the course of my study.

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 çerçeve 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 şekilleri ve yük-sehim eğrileri 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

Π :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
ξ	:Dimensionless coordinate in x-direction
η	:Dimensionless coordinate in y-direction
σ_1	:Maximum principal stress
σ_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
θ	:Angle of principal direction
δ	: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 nonlinearities 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 (4) 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 (15) 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. Yokel and Fattal (16) 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 (17) 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. (7) 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[3]. 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[20]. Mayes et al.[8] 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 boundry 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.

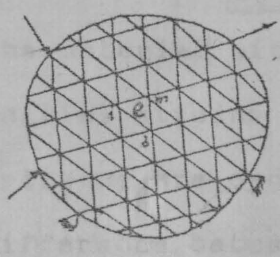


Fig. 1: Finite Element Discretization of a Plane Stress Region

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

$$\{u\} \; [N_i, N_j, \ldots.] \begin{Bmatrix} d_i \\ d_j \\ \vdots \end{Bmatrix} = [N]\{d\} \qquad (2.1)$$

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:

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

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

$$\{\varepsilon\} = [G]\{d\} \qquad (2.2)$$

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 (1) which define $[\Delta]$:

$$\{\varepsilon\} = \begin{Bmatrix} \partial u/\partial x \\ \partial v/\partial y \\ \partial u/\partial y + \partial v/\partial x \end{Bmatrix} = \underbrace{\begin{bmatrix} \partial/\partial x & 0 \\ 0 & \partial/\partial y \\ \partial/\partial y & \partial/\partial x \end{bmatrix}}_{[\Delta]} \begin{Bmatrix} u \\ v \end{Bmatrix}$$

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 $\{\varepsilon_0\}$, 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:

$$\{\sigma\}=[D](\{\varepsilon\}-\{\varepsilon_0\})\ \{\sigma_0\} \qquad (2.3)$$

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[1].

$$\{\sigma\}=[D]([G]\{d\}-\{\varepsilon_0\})+\{\sigma_0\}$$

$$[S]\{d\}-[D]\{\varepsilon_0\}+\{\sigma_0\} \qquad (2.4)$$

in which $[S]=[D][G]$ 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, Π, can be thought to be equal to the sum of the potential energies of individual elements:

$$\Pi=\sum \Pi_e$$

The potential energy functional Π_e of an element is:

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

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) Π_e is obtained as:

$$\Pi_e = \frac{1}{2}\int_V \{d\}^T[G]^T[D][G]\{d\}dV - \frac{1}{2}\int_V \{d\}^T[G]^T[D]\{\varepsilon_0\}dV +$$
$$+ \frac{1}{2}\int_V \{d\}^T[G]^T\{\sigma_0\}dV - \int_V \{d\}^T[N]^T\{p_B\}dV - \int_S \{d\}^T[N]^T\{p_s\}ds -$$
$$-\{d\}^T\{p_N\}$$

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,

$$\delta\Pi_e = 0 = \{\delta d\}^T\Big[\Big(\int_V [G]^T[D][G]dV\Big)\{d\} - \frac{1}{2}\Big(\int_V [G]^T dV\Big)[D]\{\varepsilon_0\} +$$
$$+ \frac{1}{2}\Big(\int_V [G]^T dV\Big)\{\sigma_0\} - \int_V [N]^T\{p_B\}dV - \int_S [N]^T\{p_s\}ds - \{p_N\}\Big]$$

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:

$$[k]\{d\} = \{p_N\} - (\{f\}_{\varepsilon_0} - \{f\}_{\sigma_0} - \{f\}_{p_B} + \{f\}_{p_s}) \qquad (2.5)$$

with

$$[k] = \int_V [G]^T[D][G]dV$$
$$\{f\}_{\varepsilon_0} = -\Big(\frac{1}{2}\int_V [G]^T dV\Big)[D]\{\varepsilon_0\}$$
$$\{f\}_{\sigma_0} = \Big(\frac{1}{2}\int_V [G]^T dV\Big)\{\sigma_0\}$$
$$\{f\}_{p_B} = -\int_V [N]^T\{p_B\}dV$$
$$\{f\}_{p_s} = -\int_V [N]^T\{p_s\}ds$$

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[10]. Thus equilibrium equations for the system takes the form:

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

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 nonhomogeniuty 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.[2] in their study of rock-joints, but it has been adopted to masonry also[3]. Element geometry in local coordinate system is shown below:

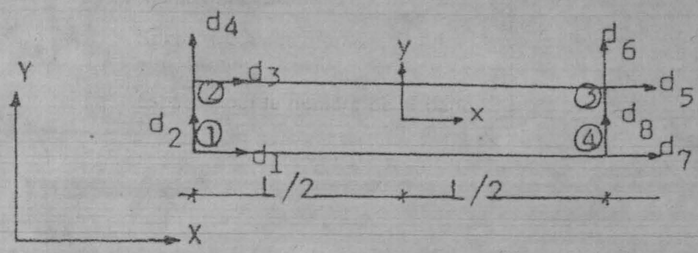


Fig.2. Joint Element (After Page[3])

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:

$$\{w\}=\begin{Bmatrix} w_s^{top}-w_s^{bottom} \\ w_n^{top}-w_n^{bottom} \end{Bmatrix} \qquad (2.7)$$

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:

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

For the shear direction, substituting $A=T\cdot L$ and $L'=t_m$ (where T represents wall thickness) into the formula $\delta=\frac{PL'}{AG}$:

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

solving for P

$$P=\delta\frac{GT}{t_m}L$$

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

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

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

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

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

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

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

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

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

Substituting into equation (2.7)

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

where

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

This equation can be written as

$$\{w\} = [G]\{d\} \tag{2.9}$$

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)

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

Carrying out the triple matrix multiplication yields

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

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:

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

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

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

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 $\theta=90^0$

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

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:

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

This process is illustrated in Fig.3.

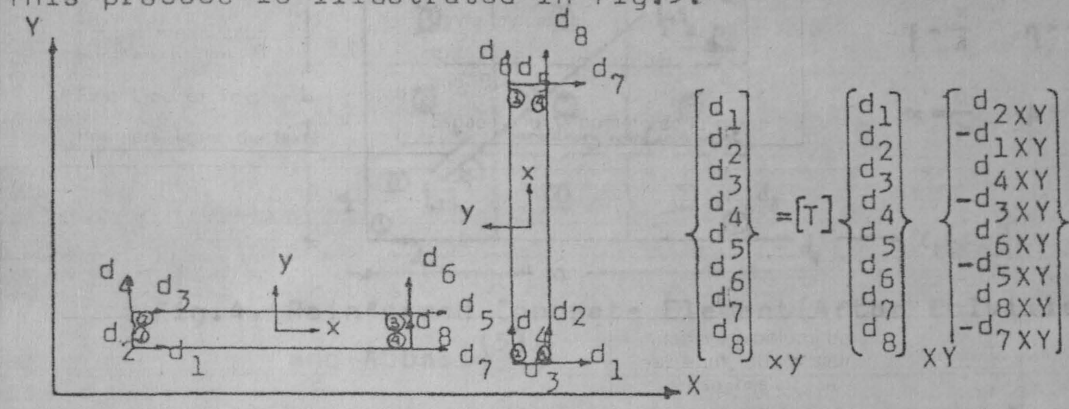


Fig.3. Coordinate Transformation For Joint Element

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:

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

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

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

2...2 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:

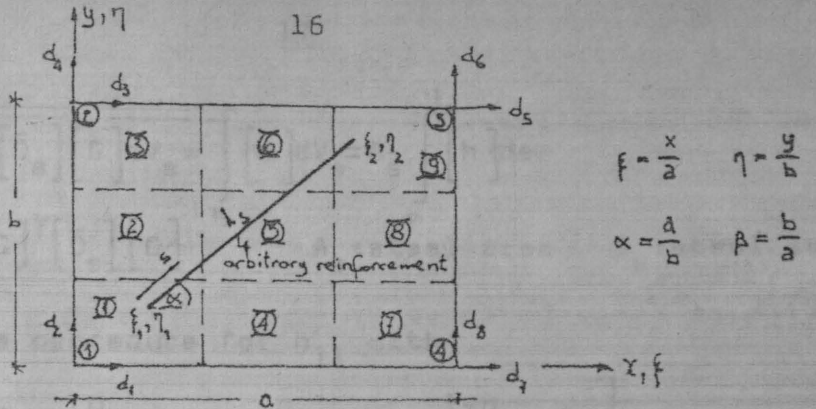


Fig.4. Reinforced Concrete Element(After Colville and Abbasi[5])

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

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

$$\left[k_c\right] = \int_V \left[G\right]^T \left[D_c\right]\left[G\right] dV = T \int_A \left[G\right]^T \left[D_c\right]\left[G\right] dA$$

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

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

where

$$\left[k_s'\right] = \int_{V_s} \left[G\right]^T \left[\bar{D}_s\right]\left[G\right] dV_s$$

with

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

m: number of reinforcing steel bars in the element

D_s : elasticity matrix of steel

D_c : elasticity matrix of concrete

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

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

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

Illustrating the procedure for h_{11} with

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

and using the notation from Fig.4

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

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

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

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

$$k_{11}=\int_V h_{11}dV=A_s\int_0^{l_s}h_{11}ds$$

$$=A_s\int_0^{l_s}(\bar{\eta}^2D_{11}-2\bar{\eta}sSD_{11}+s^2S^2D_{11}+\bar{\xi}^2D_{33}-2\bar{\xi}sCD_{33}+s^2C^2D_{33})ds=$$

$$=(\bar{\eta}^2D_{11}+\bar{\xi}^2D_{33})l_s+(-\bar{\eta}SD_{11}-\bar{\xi}CD_{33})l_s^2+(S^2D_{11}+C^2D_{33})\frac{l_s^3}{3}\,A_s=$$

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

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

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

and $V_s=A_sl_s$

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

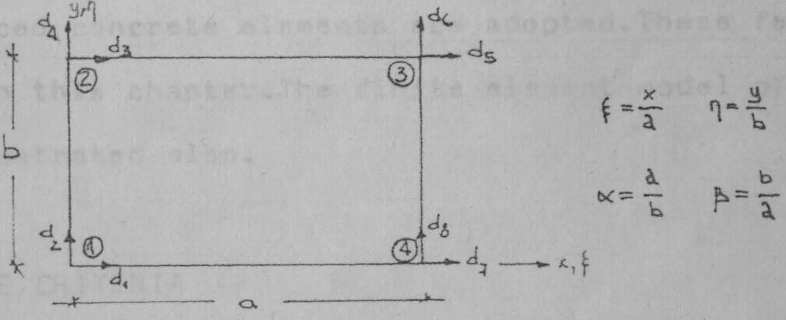


Fig.5. Brick Element

3. 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 Page[(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.Two linear best fit curves have bee 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.

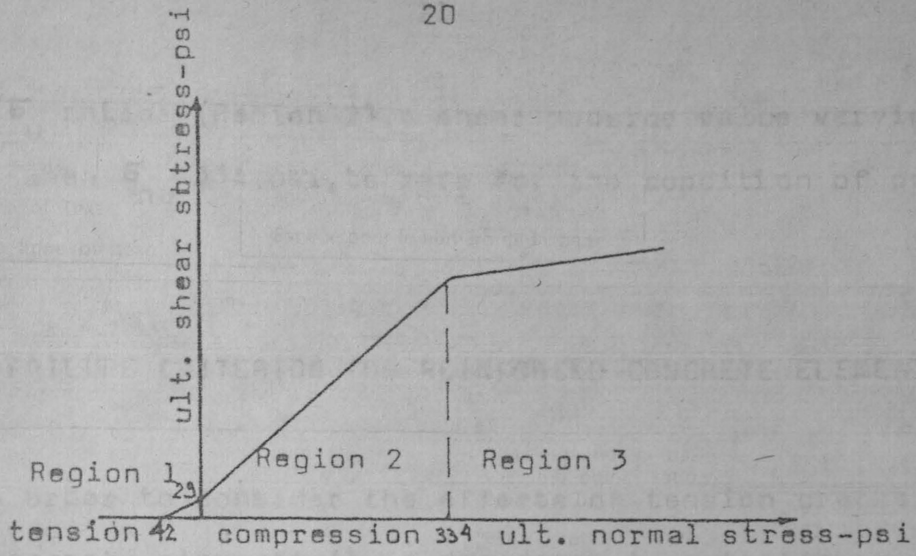


Fig.6. Failure Criterion for Joint Elements(After Page[3])

The relations adopted for the criterion are:

Region 1 : $\tau_u = -0.66\sigma_{nu} - 29$

Region 2 : $\tau_u = -0.87\sigma_{nu} - 29$

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

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 violate tensile bond failure is assumed to occur,and no residual capacit is assigned to that element (E=G=0).If failure occurs under a co bination of compressive and shear stress (Regions 2 and 3) a shear bond failure is simulated.The stiffness of the joint eleme 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 τ_u/σ_u ratios (Region 3),a constant residual value for shear modulus G of 3630 psi is allocated.For high τ_u/σ_u ratios (Region 2), a shear modulus value varying from 3630 psi when σ_{nu}=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 occurance of tension cracks. According to the criterion used by Zienkiewicz[6] 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, σ_{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:

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

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

θ, 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 θ,θ 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 σ_1 acts on the plane defined by θ,but if $\sigma' = \sigma_2$,then σ_1 acts on the plane defined by $\theta + 90^{\circ}$

After finding θ,σ_1 and σ_2 are compared with allowable tensile stress σ_{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 σ_1 and it is assumed that no stress is taken anymore in that direction.So vector $\{S_{ccp}\}$ has the form $\{S_{ccp}\} = \begin{Bmatrix} \sigma_1 \\ 0 \\ 0 \end{Bmatrix}$, $\{S_{ccp}\}$ representing the released stresses.

Case 3: Element subdivision cracks due to both σ_1 and σ_2 and it is assumed that no stresses are taken in both principal directions anymore.So vector $\{S_{ccp}\}$ takes the form $\{S_{ccp}\} = \begin{Bmatrix} \sigma_1 \\ \sigma_2 \\ 0 \end{Bmatrix}$.

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

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

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:

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

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

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

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:

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

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:

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

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:

$$\{pq_{cc}\} = \int_V [G]^T \{\sigma\}_i dV \qquad (3.2)$$

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:

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

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 (10), equation (3.1) can be written as the new equation of the system as follows:

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

$$\underbrace{-\{\Delta P\}_i-[K]_i\{d\}_i}_{I}+\underbrace{\{Pq_{cc}\}}_{II} \quad (3.4)$$

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 arrangment of equation (3.4), it takes the form

$$[K]_i(\{d\}_i+\{\Delta d\}_i)=\{\Delta P\}_i+\{Pq_{cc}\}$$

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

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

where δ is a sufficiently small number and $\|.\|$ 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.

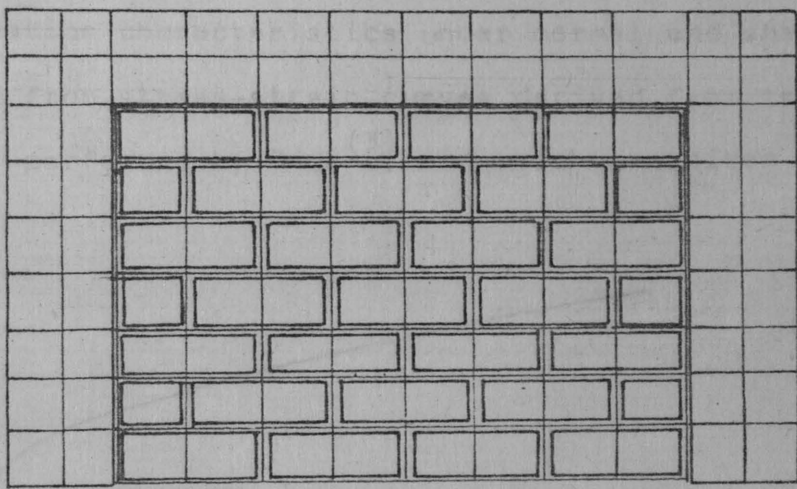


Fig.7. Finite Element Model of Frame-Masonry Panel

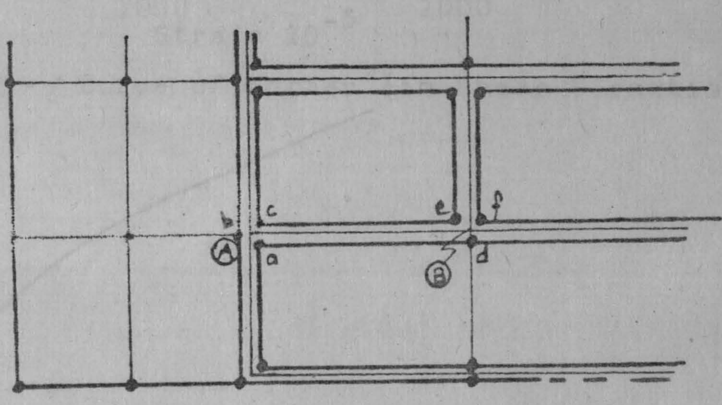


Fig.8. Close-up View of Lower Left Corner of Fig.7

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[3], and which are given in Fig.9 and Fig.10.

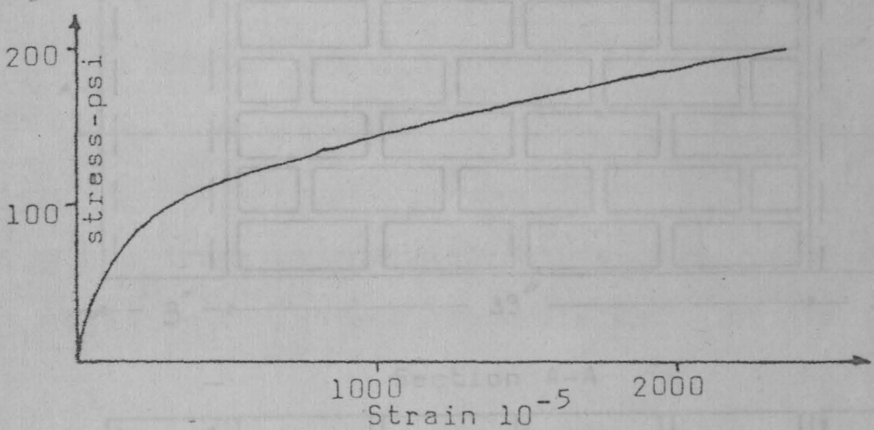


Fig.9. τ - γ Curve of Mortar (in Shear Direction)

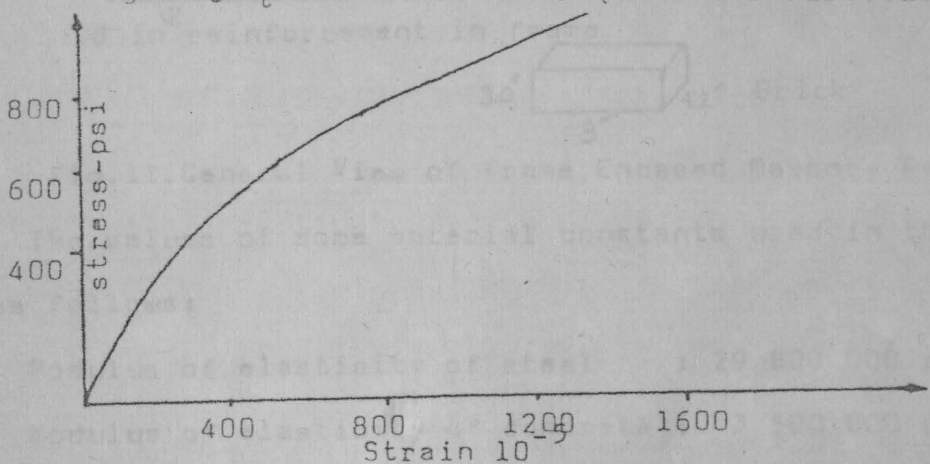


Fig.10. σ - ε Curve of Mortar (in Normal Direction)

4. APPLICATIONS AND RESULTS

4.1 INTRODUCTION

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

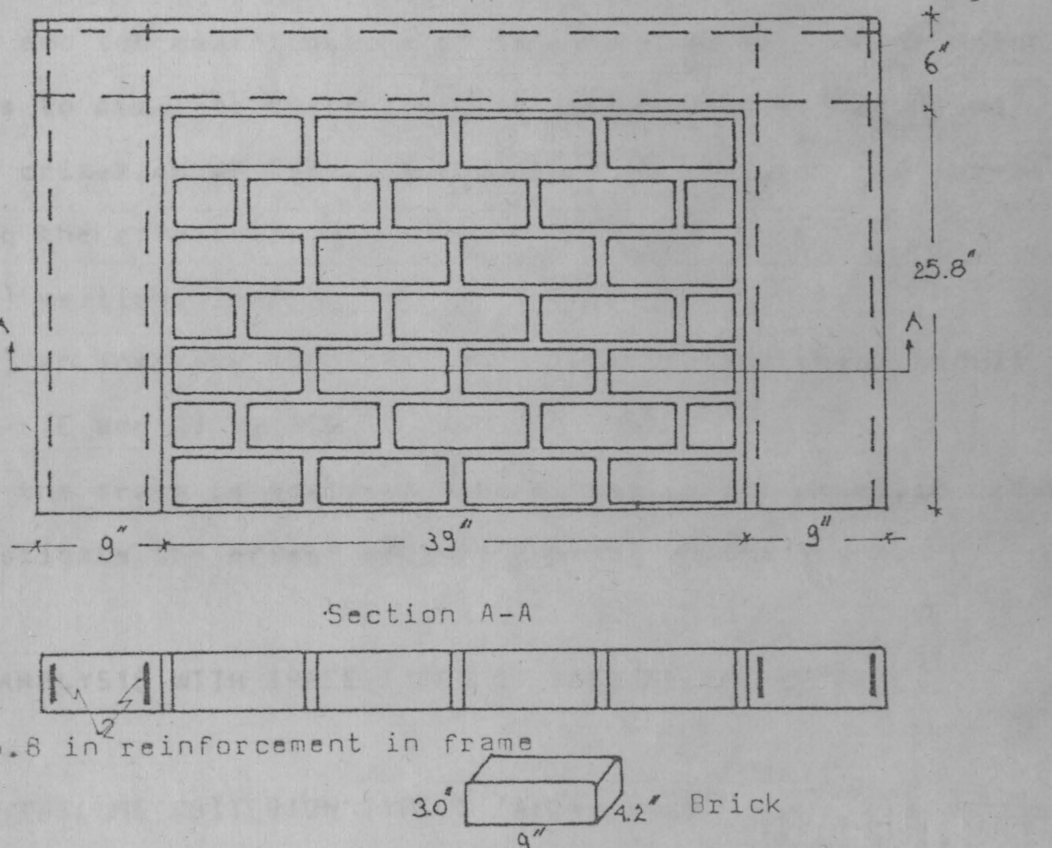


Fig.11.General View of Frame Encased Masonry Panel

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

Moudulus of elesticity 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 (8,9)	:	55 psi
Allowable tensile stress of concrete	:	650 psi
Poisson ratio of steel	:	0.17

The incremental finite element program isapplied to the structure given in Fig.11.By doing this,failure criterion of Page(3) 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.

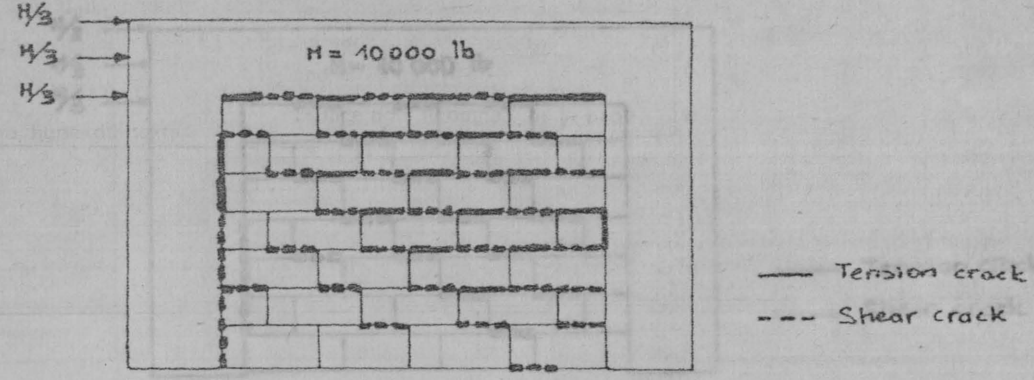


Fig.12.Crack Pattern According to Failure Criterion Type I.

4.2.2 FAILURE CRITERION TYPE II

Failure criterion illustrated in Fig.13 is used.

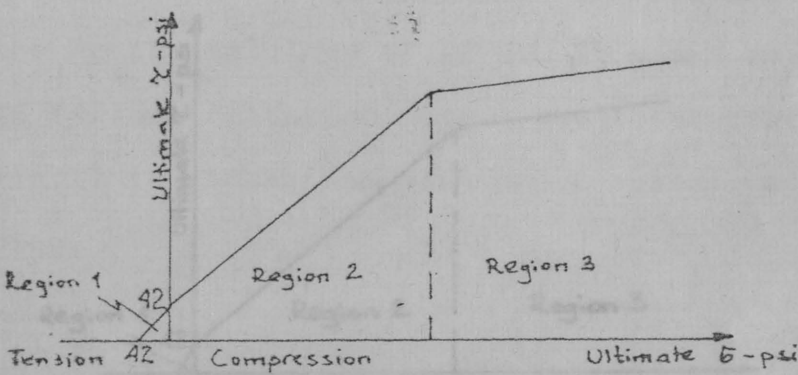


Fig.13.Failure Criterion Type II.

The relations adopted for this criterion are:

Region 1: $\tau_u = -1.0\sigma_{nu} - 42$

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

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.

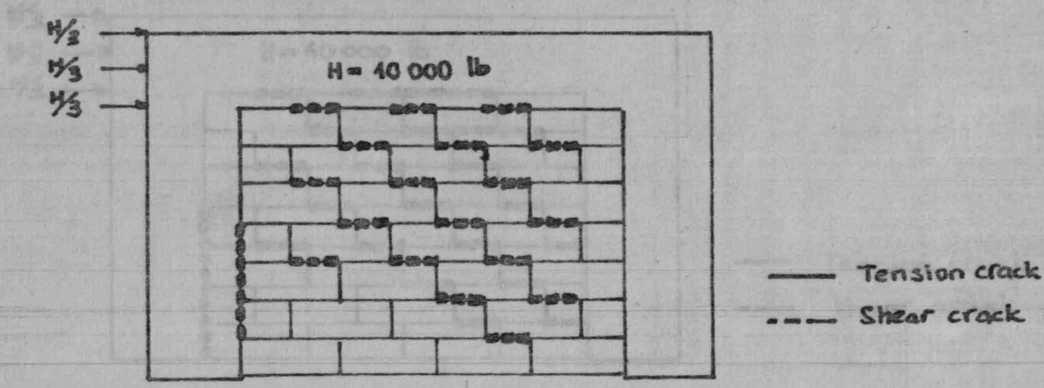


Fig.14. Crack Pattern According to Failure Criterion Type II.

4.2.3 FAILURE CRITERION TYPE III

Failure criterion illustrated in Fig.15 is used.

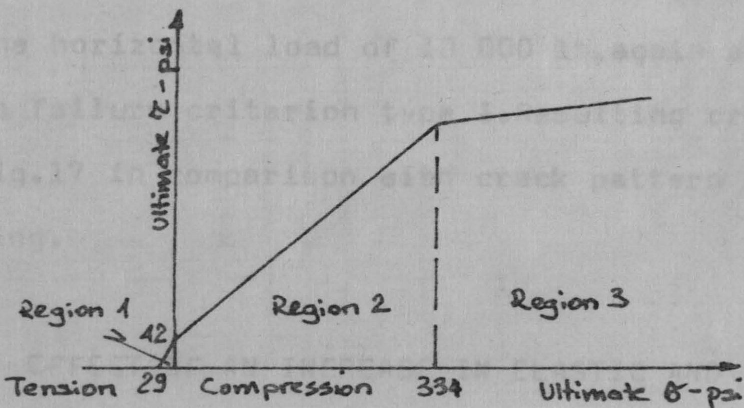


Fig.15. Failure Criterion Type III

The relations adopted for the model are:

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

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

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.

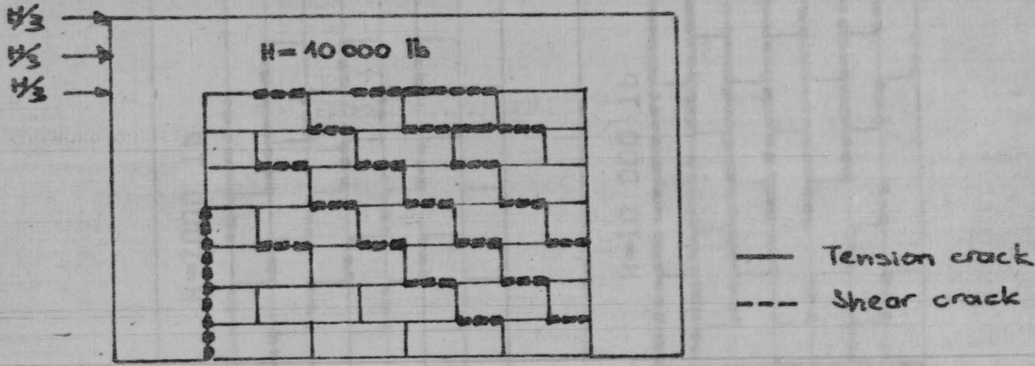


Fig.16.Crack Pattern According to Failure Criterion Type III.

4.3 EFFECT OF VERTICAL LOADING

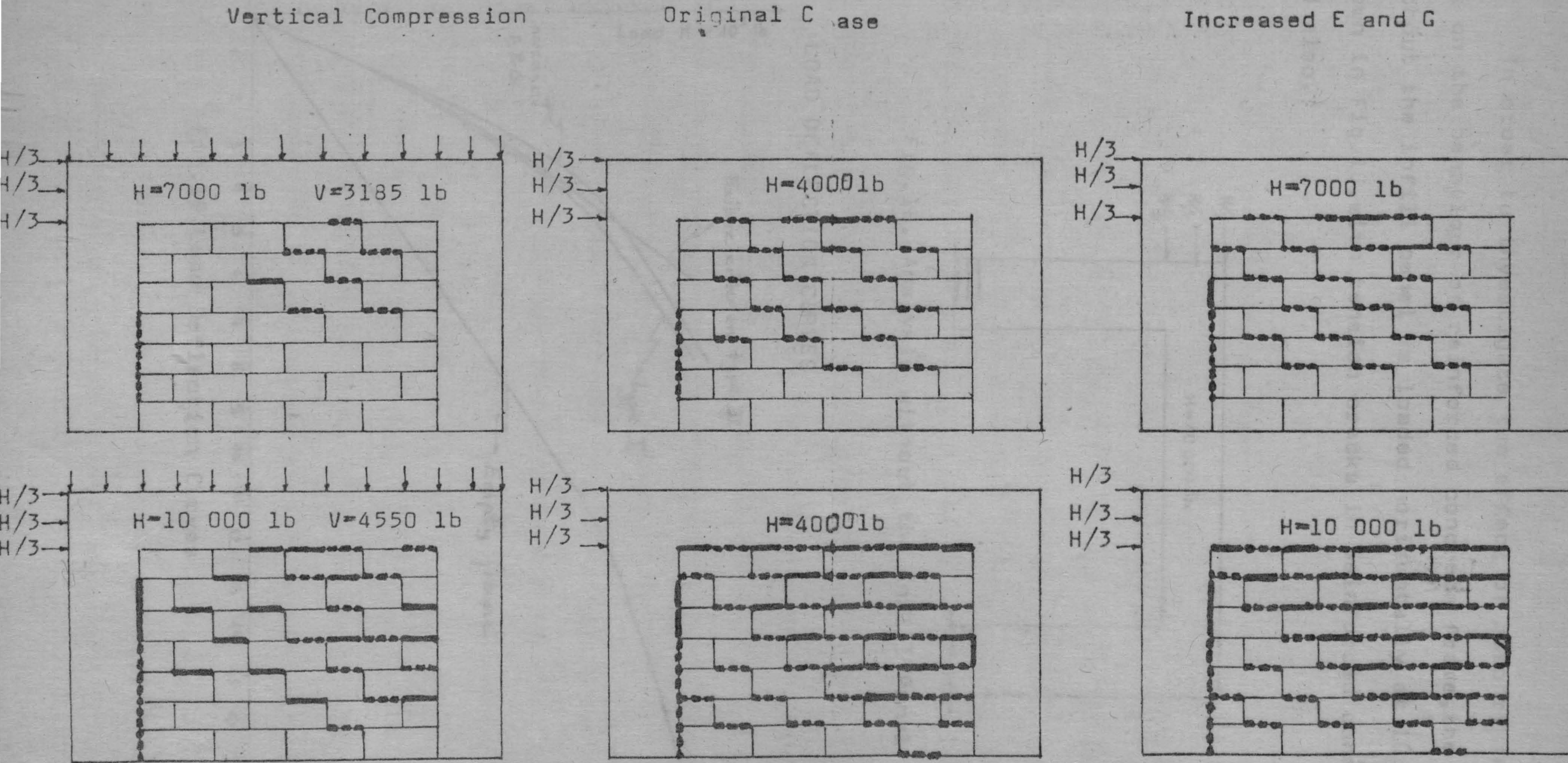
The structure is loaded vertically with 4550 lb in addition to the horizontal load of 10 000 lb,again at ten increments and using failure criterion type I.Resulting crack pattern is shown in Fig.17 in comparison with crack pattern of only horizontal loading.

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



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.

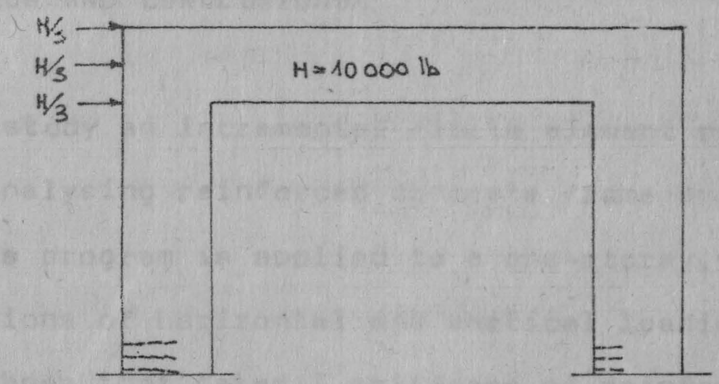


Fig.18. Analysis without the Infill Panel

4.6 LOAD DEFLECTION CURVES

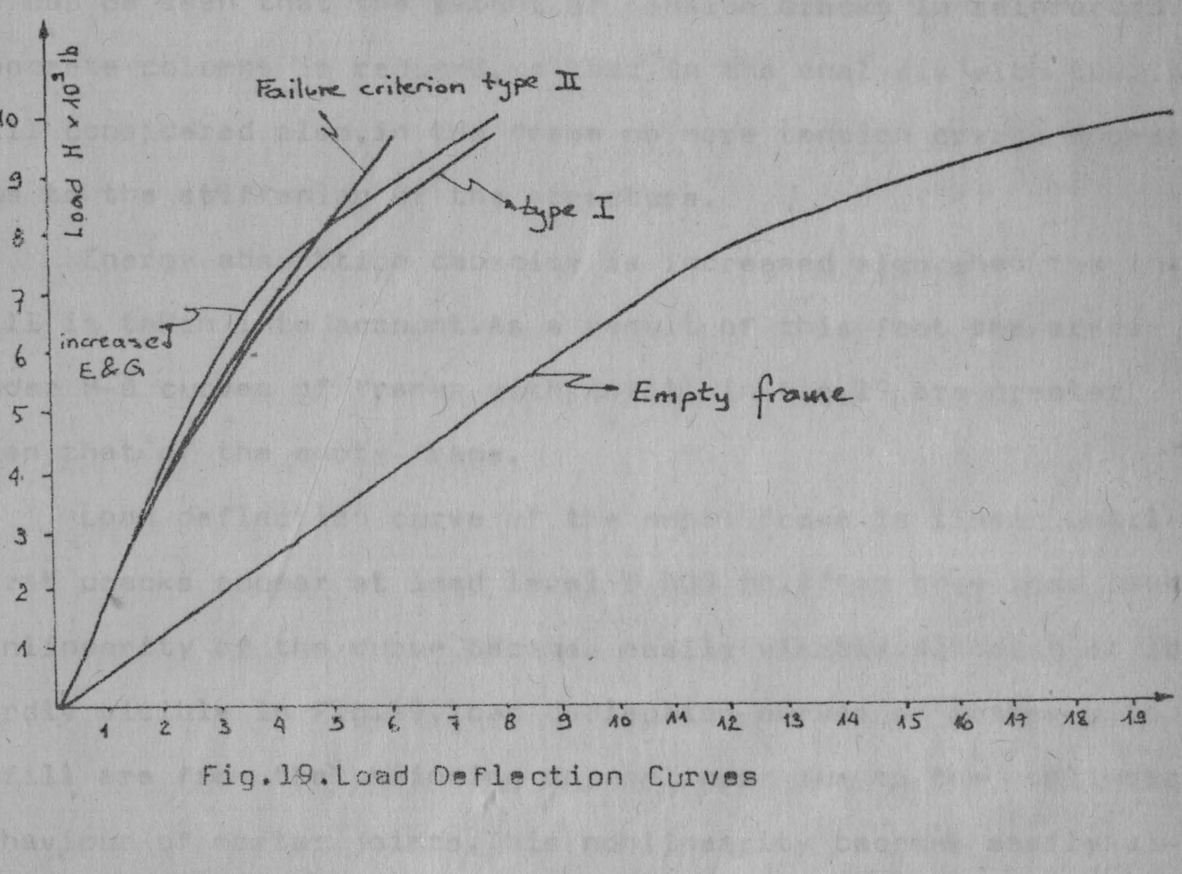


Fig.19. Load Deflection Curves

5. DISCUSSION AND CONCLUSIONS

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 absorbtion capacity is increased also,when the infill is taken into account.As a result of this fact the areas under H-δ 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 on nonlinear due to the nonlinear behaviour of mortar joints.This nonlinearity becomes easily visible as cracking in the mortar joints increase with the increasing 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.

-o-

REFERENCES

.Timoshenko S.P.,Goodier J.N.,Theory of Elasticity,3. ed. Newyork,McGraw Hill,1963.

.Goodman,R.E.,Taylor,R.L. and Brekke,T.L.,"A Model for the Mechanics of Jointed Rock",Journal of the Soil Mechanics and Foundations Division ,ASCE,Vol.94,May 1968,pp.637-659.

.Page,A.,"Finite Element Model for Masonry",Journal of the Structural Division,ASCE,Vol.104,August 1978,pp.1267-1285.

.Benjamin,J.R.,Williams,H.A.,"The Behaviour of the One Storey Brick Shear Walls",Journal of the Structural Division,ASCE Vol.84,July 1958,pp.1723-1-1723-30.

.Colville,J.,Abbasi,J.,"Plane Stress Reinforced Concrete Finite Elements",Journal of the Structural Division,ASCE Vol.100,May 1974, pp.1067-1083.

.Zienkiewicz,O.C.,Valiappan,S. and King,I.P.,"Stress Analysis of Rock as a 'No-tension' Material",Geotechnique ,London,England, Vol.18,1968,pp.56-66.

.Umemura,H.,Aogama,H.,Noguchi,H.,"Test of Plain Brick Walls Subjected to Lateral Force",Experimental Studies on Reinforced Concrete Members,Vol.3.,Faculty of Engineering,Department of Architecture, The University of Tokyo,pp.78-90.

.L.Mayes,Clough,R.,Omote Y. and Chen,S,"Expected Performance of Uniform Building Code Designed Masonry Buildings",Earthquake Resistant Masonry Construction,NBS Building Science Series 106,pp. 91-109.

.Hacim,Ergül,"Effect of Number of Bars and Amount of Prestress on the Lateral Load Behaviour of Post-tensioned Brick Walls", Master Thesis,Civil Engineering Department,Boğaziçi University, July 1980.

10. Tezcan, Semih, Çubuk Sistemlerin Elektronik Hesap Makineleri ile Çözümü, İTÜ Matbaası, İstanbul, 1971.

11. Dawson, R. and Ward M.A., "Dynamic Response of Framed Structures with Infill Walls", Proc. 5th World Conference on Earthquake Engineering, Rome, 2., 1507-1516 (1973)

12. Wen R.K. and Natarajan, P.S., "Inelastic Seismic Behaviour of Frame-Wall Systems", Proc. 5th World Conference on Earthquake Engineering, Rome, 1, 1343-1352 (1973).

13. Moss, P.J. and Carr, A.J., "Aspects of the Analysis of Frame-Panel Interaction", Bulln. New Zealand Society of Earthquake Engineering, 4, No. 1, 126-144 (March, 1971).

14. Mallick, D.V and Severn, R.J., "Dynamic Characteristics of Infilled Frames", Proc. Institution of Civil Engineers, 39, 261-287 (1968)

15. Stafford Smith, B. and Carter, C., "A Method of Analysis for Infilled Frames", Proc. Institution of Civil Engineers, 44, 31-48, (1969).

16. Yokel, F.Y., Fattal, S.G., "Failure Hypothesis for Masonry Shear Shear Walls", Journal of the Structural Division, ASCE, Vol. 102, March 1976, pp. 515-531.

17. R. Meli, "Behaviour of Masonry Walls under Lateral Loads", Proc. 5th World Conference on Earthquake Engineering, Rome, 1973, pp. 853-862.

18. Zienkiewicz, O.C., The Finite Element Method, McGraw Hill Inc, Maidenhead, England, 1977.

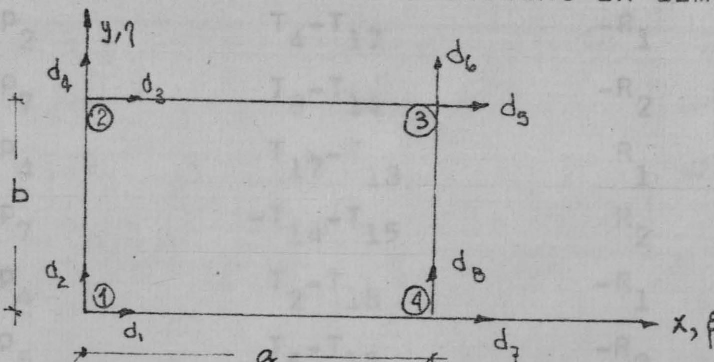
19. Newmark, N.M., Rosenblueth, E., Fundamentals of Earthquake Engineering, Prentice-Hall Inc., Englewood Cliffs, N.J., 1971.

20. Dowrick, D.J., Earthquake Resistant Design, John Wiley and Sons, England, 1978.

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:



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

$$[k] = \frac{ET}{12(1-\nu^2)} =$$

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

APPENDIX 2

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

	ψ'	ψ''	ψ'''
k_{11}	$C_1P_1-C_2P_2$	$-T_2-T_4$	R_1
k_{12}	C_3P_5	$-T_6-T_8$	R_2
k_{13}	$C_1P_3-C_2P_2$	T_4-T_{17}	$-R_1$
k_{14}	$-C_4P_5-C_5P_7$	T_8-T_{14}	$-R_2$
k_{15}	$-C_1P_3-C_2P_4$	$T_{17}-T_{18}$	R_1
k_{16}	$-C_4P_6-C_5P_7$	$-T_{14}-T_{15}$	R_2
k_{17}	$-C_1P_1-C_2P_4$	T_2-T_{18}	$-R_1$
k_{18}	$C_4P_6-C_5P_5$	T_6-T_{15}	$-R_2$
k_{22}	$C_6P_2-C_7P_1$	$-T_{10}-T_{12}$	R_3
k_{23}	$C_4P_7-C_5P_5$	T_8-T_{13}	$-R_2$
k_{24}	$-C_6P_2-C_7P_3$	$T_{10}-T_{20}$	$-R_3$
k_{25}	$-C_4P_7-C_5P_6$	$T_{13}-T_{16}$	R_2
k_{26}	$-C_6P_4-C_7P_3$	$-T_{19}-T_{20}$	R_3
k_{27}	$-C_4P_5-C_5P_6$	T_6-T_{16}	$-R_2$
k_{28}	$C_6P_4-C_7P_1$	$T_{12}-T_{19}$	$-R_3$
k_{33}	$C_1P_9-C_2P_2$	T_1-T_4	R_1
k_{34}	$-C_3P_7$	T_5-T_8	R_2
k_{35}	$-C_1P_9-C_2P_4$	$-T_1-T_{18}$	$-R_1$
k_{36}	$-C_4P_8-C_5P_7$	$-T_5-T_{15}$	$-R_2$
k_{37}	$-C_1P_3-C_2P_4$	$T_{17}-T_{18}$	R_1
k_{38}	$C_4P_8-C_5P_5$	$T_{13}-T_{15}$	R_2
k_{44}	$C_6P_2-C_7P_9$	$-T_{10}-T_{11}$	R_3
k_{45}	$C_4P_7-C_5P_8$	$-T_5-T_{16}$	$-R_2$
k_{46}	$C_6P_4-C_7P_9$	$-T_{11}-T_{19}$	$-R_3$

k_{47}	$C_4P_5-C_5P_8$	$-T_{14}-T_{16}$	R_2
k_{48}	$-C_6P_4-C_7P_3$	$-T_{19}-T_{20}$	R_3
k_{55}	$C_1P_9-C_2P_{10}$	T_1-T_3	R_1
k_{56}	C_3P_8	T_5-T_7	R_2
k_{57}	$C_1P_3-C_2P_{10}$	$-T_3-T_{17}$	$-R_1$
k_{58}	$-C_4P_8-C_5P_6$	$-T_7-T_{13}$	$-R_2$
k_{66}	$C_6P_{10}-C_7P_9$	T_9-T_{11}	R_3
k_{67}	$C_4P_6-C_5P_8$	$-T_7-T_{14}$	$-R_2$
k_{68}	$-C_6P_{10}-C_7P_3$	$-T_9-T_{20}$	$-R_3$
k_{77}	$C_1P_1-C_2P_{10}$	$-T_2-T_3$	R_1
k_{78}	$-C_3P_6$	$-T_6-T_7$	R_2
k_{88}	$C_6P_{10}-C_7P_1$	T_9-T_{12}	R_3

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

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

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

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

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

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

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

$$C=\frac{\xi_2-\xi_1}{L_s} \qquad S=\frac{\eta_2-\eta_1}{L_s}$$

$$V_s=A_sL_s$$

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

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

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

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

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

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

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

$$P_8=\eta_1\xi_1$$

$$P_9=\eta_1^2$$

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

$$R_1=\frac{(C_1S^2-C_2C^2)L_s^2}{3}$$

$$R_2=\frac{C_3SCL_s^2}{3}$$

$$R_3=\frac{(C_6C^2-C_7S^2)L_s^2}{3}$$

$$T_1 = C_1 \eta_1 S L_s$$

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

$$T_3 = C_2 \xi_1 C L_s$$

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

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

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

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

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

$$T_9 = C_6 \xi_1 C L_s$$

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

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

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

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

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

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

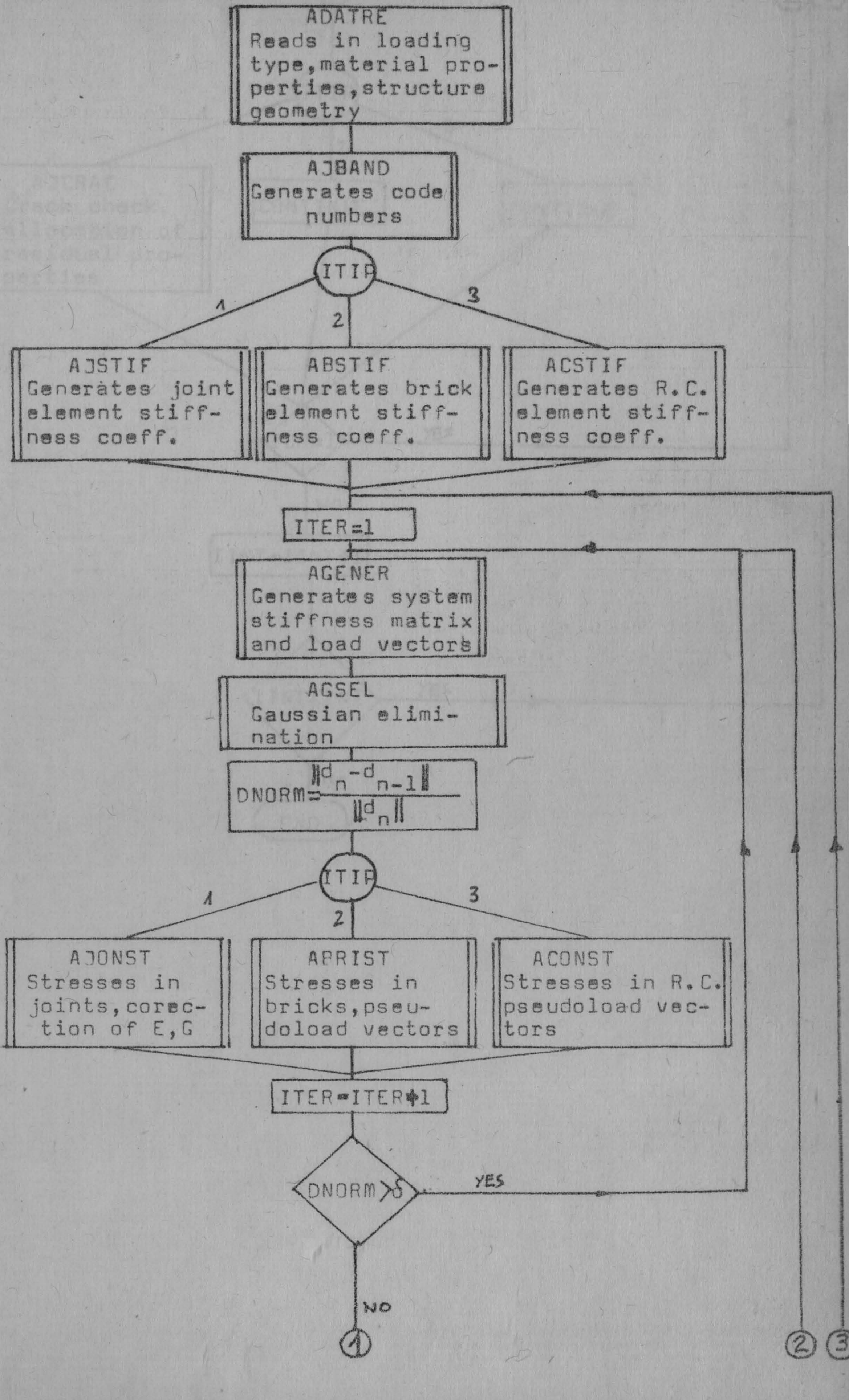
$$T_{16} = \frac{S(C_4 \bar{\xi} - C_5 \xi_1) L_s}{2}$$

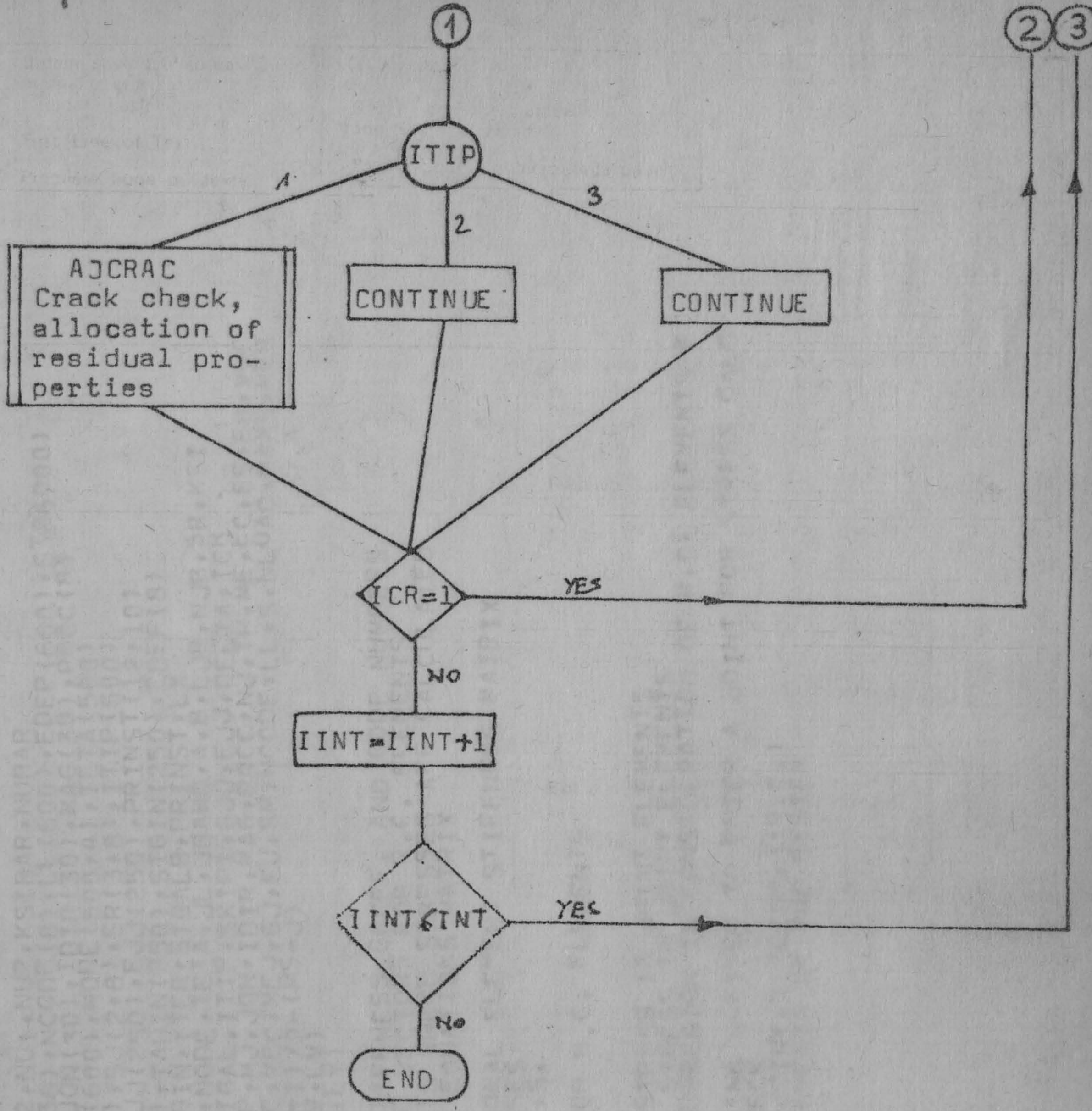
$$T_{17} = \frac{C_1 S(\eta_1 - \bar{\eta}) L_s}{2}$$

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

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

APPENDIX 3 .FLOWCHART OF SOLUTION PROCEDURE




1
2
3
ITIP
1
2
3
AJCRAC
Crack check,
allocation of
residual pro-
perties
CONTINUE
CONTINUE
ICR=1
YES
NO
IINT=IINT+1
IINT≤INT
YES
No
END

THESIS*AMEF(1).AMEF

```
      REAL JL,KS,KN,KSI1,KSI2,NU1,NU2,KSIBAR,NUBAR
      DIMENSION DEF(600),SM(36),NCODE(8),LL(600),EDEP(600),S(26000)
      DIMENSION SKIP(110,9),JON(30),IDIR(30),MAG(30),PQCC(8)
      DIMENSION JOX(600),JOY(600),NODE(500,4),TETA(500)
      DIMENSION A(500),B(500),SJ(2,8),SR(3,8),ITIP(500)
      DIMENSION SKIP1(250),GJJ(250),EJJ(250),PRINST(12,10)
      DIMENSION STRESS(250,2),TAUIN(250),SIGIN(250),EDEF(8)
      COMMON STRESS,TAUIN,SIGIN,ITER,SIGALB,PRINST,LX
      COMMON VUS,VUB,JOX,JOY,NODE,TETA,JL,JBAND,A,B,LJB,NJB,SR,KSI
      COMMON NU,X,SJ,KS,KN,SIGAL,ITIP,SKIP1,GJJ,EJJ,DELTA,ICR
      COMMON MSS,TJ,EDEP,SKIP,MJ,JON,IDIR,MAG,PQCC,NJ,TH,ME,EC,ES,EB,VUC
      COMMON N,MS,INT,IINT,NC,MEC,MEJ,GJ,EJ,SM,NCODE,LL,S,NLOAD,MAXS,MEB
      LOC(II,J)=II*MS-II*(II-1)/2-(MS-J)
      DEFINE FILE 10(600,50,V,LV)
      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: HALFBANDWIDTH 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     SKIP1: 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
      MS=8
      NLOAD=1
      JBAND=0
      NC=4
      DO 40 I=1,8
   40 PQCC(I)=0.
      DO 50 LE=1,110
```

```
   50 WRITE(29,LE)(PQCC(I),I=1,8)
      CALL ADATRE
      DO 8 I=1,MEJ
      TAUIN(I)=0.
      SIGIN(I)=0.
    8 SKIP1(I)=0.
      DO 10 I=1,MEC
      DO 20 L=1,9
   20 SKIP(I,L)=0.
   10 CONTINUE
      DO 800 I=1,ME
      LD=I
      CALL AJBAND(I)
      GO TO (801,802,803) ITIP(I)
  801 CALL AJSTIF(I)
      GO TO 800
  802 CALL ABSTIF(I)
      GO TO 800
  803 CALL ACSTIF(I)
  800 WRITE(10,LD)(SM(K),K=1,MSS),(NCODE(M),M=1,MS)
      DO 23 J=1,N
   23 EDEP(J)=0.
   22 PRINT 600,IINT
  600 FORMAT(/40X,10HINCREMENT ,I2/)
   24 DO 25 J=1,N
   25 DEF(J)=0.
   27 ITER=1
   28 DEPNOR=0.
      DDNOR=0.
      LX=0
      PRINT 650,ITER
  650 FORMAT(40X,10HITERASYON ,I2)
      CALL AGENER
      CALL AGSEL
      DO 30 J=1,N
      K=LL(J+1)+J
      DEP=S(K)
      DD=DEP-DEF(J)
      DEPNOR=DEPNOR+DEP*DEP
      DDNOR=DDNOR+DD*DD
   30 DEF(J)=DEP
      DEPNOR=SQRT(DEPNOR)
      DDNOR=SQRT(DDNOR)
      DNORM=DDNOR/DEPNOR
      PRINT 1,DNORM
    1 FORMAT(40X,F6.4/)
      DO 900 I=1,ME
      GO TO (501,502,503) ITIP(I)
```

```
  501 CALL ASJON(I)
      CALL AJONS'(I)
      GO TO 900
  502 CALL ABRIST(I)
      GO TO 900
  503 CALL ACONS'(I)
  900 CONTINUE
      ITER=ITER+1
      IF(ITER.EQ.15) GO TO 1001
      IF(DNORM.GT.DELTA) GO TO 28
      ICR=0
      DO 1000 I=1,ME
      GO TO (504,505,506) ITIP(I)
  504 CALL ASJON(I)
      CALL AJCRAC(I)
      GO TO 1000
  505 CONTINUE
      GO TO 1000
  506 CONTINUE
 1000 CONTINUE
      IF(ICR.EQ.1) GO TO 28
      DO 169 J=1,N
  169 EDEP(J)=EDEP(J)+DEF(J)
      IF(MEJ.EQ.0) GO TO 171
      DO 170 I=1,MEJ
      TAUIN(I)=TAUIN(I)+STRESS(I,1)
  170 SIGIN(I)=SIGIN(I)+STRESS(I,2)
  171 CONTINUE
      IINT=IINT+1
      PRINT 2
    2 FORMAT(36X,21HCONVERGENCE ACCHIEVED/)
      IF(MEC.EQ.0) GO TO 172
      PRINT 176
  176 FORMAT(17X,55HDISPLACEMENTS OF CONCRETE ELEMENTS AT THIS LOAD LEVE
     *L ://2X,7HELM. NO,6X,2HD1,11X,2HD2,11X,2HD3,11X,2HD4,11X,2HD5,11X,
     *2HD6,11X,2HD7,11X,2HD8/)
      LK=MEJ+MEB+1
      DO 177 LD=LK,ME
      READ(10,LD)(SM(K),K=1,MSS),(NCODE(M),M=1,MS)
      DO 9 J=1,8
      EDEF(J)=0.
      SAYN=1.
      IN=NCODE(J)
      IF(IN) 21,9,26
   21 IN=-IN
      SAYN=-1.
   26 EDEF(J)=EDEP(IN)*SAYN
    9 CONTINUE
```

```
  177 PRINT 11,LD,(EDEF(J),J=1,8)
   11 FORMAT(4X,I3,8(5X,F8.6))
      PRINT 4
    4 FORMAT(/17X,44HSTRESSES AT X-SECTION A-A AT THIS LOAD LEVEL/2X,8HE
     *LM. NO.,2X,5HSIGX2,5X,5HSIGY2,6X,4HTAU2,5X,5HSIGX5,5X,5HSIGY5,6X,4
     *HTAU5,5X,5HSIGX8,5X,5HSIGY8,6X,4HTAU8/)
      DO 29 I=1,12
   29 PRINT 5,(PRINST(I,J),J=1,10)
    5 FORMAT(2X,F5.1,9(4X,F6.1))
  172 IF(IINT.LE.INT) GO TO 22
      GO TO 1002
 1001 PRINT 13
   13 FORMAT(32X,25HCONVERGENCE NOT ACCHIEVED/)
 1002 STOP
      END
```

```
THESIS*AMEF(1).ADATRE
      SUBROUTINE ADATRE
      DIMENSION JOX(600),JOY(600),NODE(500,4),TETA(500)
      DIMENSION A(500),B(500),SR(3,8),EDEP(600),SKIP(110,9),JON(30)
      DIMENSION IDIR(30),MAG(30),PQCC(8),SM(36),NCODE(8)
      DIMENSION LL(600),S(26000),SJ(2,8),ITIP(500)
      DIMENSION SKIP1(250),GJJ(250),EJJ(250)
      DIMENSION STRESS(250,2),TAUIN(250),SIGIN(250),PRINST(12,10)
      COMMON STRESS,TAUIN,SIGIN,ITER,SIGALB,PRINST,LX
      COMMON VUS,VUB,JOX,JOY,NODE,TETA,JL,JBAND,A,B,LJB,NJB,SR,KSI
      COMMON NU,X,SJ,KS,KN,SIGAL,ITIP,SKIP1,GJJ,EJJ,DELTA,ICK
      COMMON MSS,TJ,EDEP,SKIP,MJ,JON,IDIR,MAG,PQCC,NJ,TH,ME,EC,ES,EB,VUC
      COMMON N,MS,INT,IINT,NC,MEC,MEJ,GJ,EJ,SM,NCODE,LL,S,NLOAD,MAXS,MEB
C   SUBROUTINE FOR READING IN DATA AND EVALUATING SYSTEM GEOMETRY
C   MJ: NO. OF NODAL LOADINGS
C   JON: JOINT NO.
C   IDIR: DIRECTION
C   MAG: MAGNITUDE
C   MEJ: NO. OF JOINT ELEMENTS
C   MEB: NO. OF BRICK ELEMENTS
C   MEC: NO. OF R.C. ELEMENTS
C   ME: TOTAL NO. OF ELEMENTS
C   NJ: NO. OF JOINTS
C   TH: WALL THICKNESS
C   TJ: JOINT THICKNESS
C   INT: NO. OF INCREMENTS
C   DELTA: A SUFF. SMALL NUMBER
C   SIGAL: ALL. TENSILE STRESS OF CONCRETE
C   EJ: ELASTIC MODULUS OF MORTAR JOINTS
C   GJ: SHEAR     "     "     "     "
C   EC: ELASTIC MODULUS OF CONCRETE
C   ES:  "        "      "    STEEL
C   EB*  "        "      "    BRICK
C   VUC: POISS. RATIO OF CONCRETE
C   VUB:  "       "   "    BRICK
C   VUS:  "       "   "    STEEL
C   JNN: JOINT NO. OF THE LASTLY RESTRICTED JOINT
C   VJN: JOINT NO.,S OF RESTRICTED JOINTS
C   DX: IF ,0, UNRESTRICTED IN X DIR.,OTHERWISE RESTRICTED
C   DY: IF ,0,       "         " Y  "        "          "
      PRINT 750
  750 FORMAT(1H1,49X,9HLOAD DATA//37X,8HJOINT NO,7X,9HDIRECTION,5X,9HMAG
     *NITUDE/)
      READ 710,MJ
  710 FORMAT(I2)
      DO 720 I=1,MJ
      READ 730,JON(I),IDIR(I),MAG(I)
  730 FORMAT(3I10)
```

```
  720 PRINT 740,JON(I),IDIR(I),MAG(I)
  740 FORMAT(37X,3I10,2X,2HLB)
      READ 92,MEC,MEJ,MEB,NJ,TH,TJ
      ME=MEC+MEJ+MEB
      PRINT 93,ME,MEC,MEJ,MEB,NJ,TH,TJ
      READ 90,INT,DELTA,SIGAL,SIGALB
   90 FORMAT(I3,3F10.4)
      PRINT 89,INT,DELTA,SIGAL,SIGALB
   89 FORMAT(/30X,17HNO. OF INTERVALS=,I3/30X,17HDELTA            =,F5.4/
     130X,17HSIGMA ALL. OF RC=,F5.1,2X,3HPSI/30X,20HSIGMA ALL. OF BRICK=
     2,F5.1,2X,3HPSI)
      READ 96,EJ,GJ,EC,ES,EB,VUC,VUS,VUB
      PRINT 95,EJ,GJ,EC,ES,EB,VUC,VUS,VUB
      PRINT 700
  700 FORMAT(1H1,1X,130(,*,)/50X,7HRESULTS/1X,130(,*,)//)
      DO 5 I=1,MEJ
      EJJ(I)=EJ
    5 GJJ(I)=GJ
      N=0
      IJ=1
      READ 91,JNN
   91 FORMAT(I5)
   70 READ 94,VJN,DX,DY
   94 FORMAT(3F10.0)
      JN=VJN
      IF(JN.EQ.IJ) GO TO 26
      JND=JN-1
      DO 10 J=IJ,JND
      N=N+1
      JOX(J)=N
      N=N+1
      JOY(J)=N
   10 CONTINUE
   26 JOX(JN)=0
      JOY(JN)=0
      IX=1
      IY=1
      IF(DX) 41,43,42
   41 DX=-DX
      IX=-1
   43 N=N+1
      JOX(JN)=N*IX
   42 IF(DY) 44,46,51
   44 DY=-DY
      IY=-1
   46 N=N+1
      JOY(JN)=N*IY
   51 IJ=JN+1
```

```
      IF(JN.NE.JNN) GO TO 70
      IF(JN.EQ.NJ) GO TO 100
      DO 189 J=IJ,NJ
      N=N+1
      JOX(J)=N
      N=N+1
      JOY(J)=N
  189 CONTINUE
  100 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
      NODE(I,2)=D2
      NODE(I,3)=D3
      NODE(I,4)=D4
   20 CONTINUE
   93 FORMAT(/30X,26HNO. OF ELEMENTS              =,I4/30X,26HNO. OF CONCRE
     1TE ELEMENTS=,I4/30X,26HNO. OF JOINT ELEMENTS     =,I4/30X,27HNO.  O
     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,34HPOISSON RATIO OF CONCRETE
     5     =,F5.3/30X,34HPOISSON RATIO OF STEEL                =,F5.3/30X,34HP
     6OISSON RATIO OF BRICK              =,F5.3)
   92 FORMAT(4I10,2F10.0)
      RETURN
      END
```

THESIS*AMEF(1).AJBAND

```
      SUBROUTINE AJBAND(I)
      DIMENSION NODE(500,4),JOX(600),JOY(600),NCODE(8),GJJ(250),EJJ(250)
      DIMENSION TETA(500),A(500),B(500),SR(3,8),EDEP(600)
      DIMENSION SKIP(110,9),JON(30),IDIR(30),MAG(30),PQCC(8)
      DIMENSION LL(600),S(26000),SJ(2,8),SM(36),ITIP(500),SKIP1(250)
      DIMENSION STRESS(250,2),TAUIN(250),SIGIN(250),PRINST(12,10)
      COMMON STRESS,TAUIN,SIGIN,ITER,SIGALB,PRINST,LX
      COMMON VUS,VUB,JOX,JOY,NODE,TETA,JL,JBAND,A,B,LJB,NJB,SR,KSI
      COMMON NU,X,SJ,KS,KN,SIGAL,ITIP,SKIP1,GJJ,EJJ,DELTA,ICR
      COMMON MSS,TJ,EDEP,SKIP,MJ,JON,IDIR,MAG,PQCC,NJ,TH,ME,EC,ES,EB,VUC
      COMMON N,MS,INT,IINT,NC,MEC,MEJ,GJ,EJ,SM,NCODE,LL,S,NLOAD,MAXS,MEB
      IM=0
      DO 801 J=1,NC
      IM=IM+1
      IN=NODE(I,J)
      NCODE(IM)=JOX(IN)
      IM=IM+1
      NCODE(IM)=JOY(IN)
  801 CONTINUE
      MSM=MS-1
      DO 802 J=1,MSM
      JP=J+1
      IJ=NCODE(J)
      IF(IJ) 12,802,13
   12 IJ=-IJ
   13 DO 803 K=JP,MS
      IK=NCODE(K)
      IF(IK) 14,803,15
   14 IK=-IK
   15 KF=ABS(IK-IJ)+1
      IF(JBAND-KF) 61,803,803
   61 JBAND=KF
  803 CONTINUE
  802 CONTINUE
      RETURN
      END
```

THESIS*AMEF(1).AJSTIF

```
      SUBROUTINE AJSTIF(I)
      REAL JL,KS,KN,K1,K2
      DIMENSION A(500),B(500),SM(36),TETA(500)
      DIMENSION JOX(600),JOY(600),NODE(500,4),SR(3,8),EDEP(600)
      DIMENSION SKIP(110,9),JON(30),IDIR(30),MAG(30)
      DIMENSION PQCC(8),NCODE(8),LL(600),S(26000)
      DIMENSION SJ(2,8),ITIP(500),SKIP1(250),GJJ(250),EJJ(250)
      DIMENSION STRESS(250,2),TAUIN(250),SIGIN(250),PRINST(12,10)
      COMMON STRESS,TAUIN,SIGIN,ITER,SIGALB,PRINST,LX
      COMMON VUS,VUB,JOX,JOY,NODE,TETA,JL,JBAND,A,B,LJB,NJB,SR,KSI
      COMMON NU,X,SJ,KS,KN,SIGAL,ITIP,SKIP1,GJJ,EJJ,DELTA,ICR
      COMMON MSS,TJ,EDEP,SKIP,MJ,JON,IDIR,MAG,PQCC,NJ,TH,ME,EC,ES,EB,VUC
      COMMON N,MS,INT,IINT,NC,MEC,MEJ,GJ,EJ,SM,NCODE,LL,S,NLOAD,MAXS,MEB
C STIFFNESS OF JOINT ELEMENTS
      JL=A(I)
      KS=GJJ(I)*TH/TJ
      KN=EJJ(I)*TH/TJ
      IF(TETA(I).NE.0.) GO TO 101
      K1=KS
      K2=KN
      GO TO 102
  101 K1=KN
      K2=KS
  102 SM(1)=2.*K1*JL/6.
      SM(2)=0.
      SM(3)=-2.*K1*JL/6.
      SM(4)=0.
      SM(5)=-K1*JL/6.
      SM(6)=0.
      SM(7)=K1*JL/6.
      SM(8)=0.
      SM(9)=2.*K2*JL/6.
      SM(10)=0.
      SM(11)=-2.*K2*JL/6.
      SM(12)=0.
      SM(13)=-K2*JL/6.
      SM(14)=0.
      SM(15)=K2*JL/6.
      SM(16)=2.*K1*JL/6.
      SM(17)=0.
      SM(18)=K1*JL/6.
      SM(19)=0.
      SM(20)=-K1*JL/6.
      SM(21)=0.
      SM(22)=2.*K2*JL/6.
      SM(23)=0.
      SM(24)=K2*JL/6.
```

```
      SM(25)=0.
      SM(26)=-K2*JL/6.
      SM(27)=2.*K1*JL/6.
      SM(28)=0.
      SM(29)=-2.*K1*JL/6.
      SM(30)=0.
      SM(31)=2.*K2*JL/6.
      SM(32)=0.
      SM(33)=-2.*K2*JL/6.
      SM(34)=2.*K1*JL/6.
      SM(35)=0.
      SM(36)=2.*K2*JL/6.
      RETURN
      END
```

THESIS*AMEF(1).ABSTIF

```
      SUBROUTINE ABSTIF(I)
      DIMENSION A(500),B(500),SM(36),TETA(500)
      DIMENSION JOX(600),JOY(600),NODE(500,4),SR(3,8),EDEP(600)
      DIMENSION SKIP(110,9),JON(30),IDIR(30),MAG(30)
      DIMENSION PQCC(8),NCODE(8),LL(600),S(26000)
      DIMENSION SJ(2,8),ITIP(500),SKIP1(250),GJJ(250),EJJ(250)
      DIMENSION STRESS(250,2),TAUIN(250),SIGIN(250),PRINST(12,10)
      COMMON STRESS,TAUIN,SIGIN,ITER,SIGALB,PRINST,LX
      COMMON VUS,VUB,JOX,JOY,NODE,TETA,JL,JBAND,A,B,LJB,NJB,SR,KSI
      COMMON NU,X,SJ,KS,KN,SIGAL,ITIP,SKIP1,GJJ,EJJ,DELTA,ICK
      COMMON MSS,TJ,EDEP,SKIP,MJ,JON,IDIR,MAG,PQCC,NJ,TH,ME,EC,ES,EB,VUC
      COMMON N,MS,INT,IINT,NC,MEC,MEJ,GJ,EJ,SM,NCODE,LL,S,NLOAD,MAXS,MEB
C STIFFNESS OF BRICK ELEMENTS
      COB=EB*TH/(12.*(1.-VUB**2))
      AB=A(I)
      BB=B(I)
      ALFA=AB/BB
      BETA=BB/AB
      SM(1)=(4.*BETA+2.*(1.-VUB)*ALFA)*COB
      SM(2)=3./2.*(1.+VUB)*COB
      SM(3)=(2.*BETA-2.*(1.-VUB)*ALFA)*COB
      SM(4)=3./2.*(1.-3.*VUB)*COB
      SM(5)=(-2.*BETA-(1.-VUB)*ALFA)*COB
      SM(6)=-3./2.*(1.+VUB)*COB
      SM(7)=(-4.*BETA+(1.-VUB)*ALFA)*COB
      SM(8)=-3./2.*(1.-3.*VUB)*COB
      SM(9)=(4.*ALFA+2.*(1.-VUB)*BETA)*COB
      SM(10)=-3./2.*(1.-3.*VUB)*COB
      SM(11)=(-4.*ALFA+(1.-VUB)*BETA)*COB
      SM(12)=-3./2.*(1.+VUB)*COB
      SM(13)=(-2.*ALFA-(1.-VUB)*BETA)*COB
      SM(14)=(3./2.*(1.-3.*VUB))*COB
      SM(15)=(2.*ALFA-2.*(1.-VUB)*BETA)*COB
      SM(16)=(4.*BETA+2.*(1.-VUB)*ALFA)*COB
      SM(17)=-3./2.*(1.+VUB)*COB
      SM(18)=(-4.*BETA+(1.-VUB)*ALFA)*COB
      SM(19)=3./2.*(1.-3.*VUB)*COB
      SM(20)=(-2.*BETA-(1.-VUB)*ALFA)*COB
      SM(21)=3./2.*(1.+VUB)*COB
      SM(22)=(4.*ALFA+2.*(1.-VUB)*BETA)*COB
      SM(23)=-3./2.*(1.-3.*VUB)*COB
      SM(24)=(2.*ALFA-2.*(1.-VUB)*BETA)*COB
      SM(25)=3./2.*(1.+VUB)*COB
      SM(26)=(-2.*ALFA-(1.-VUB)*BETA)*COB
      SM(27)=(4.*BETA+2.*(1.-VUB)*ALFA)*COB
      SM(28)=3./2.*(1.+VUB)*COB
      SM(29)=(2.*BETA-2.*(1.-VUB)*ALFA)*COB
```

```
      SM(30)=3./2.*(1.-3.*VUB)*COB
      SM(31)=(4.*ALFA+2.*(1.-VUB)*BETA)*COB
      SM(32)=-3./2.*(1.-3.*VUB)*COB
      SM(33)=(-4.*ALFA+(1.-VUB)*BETA)*COB
      SM(34)=(4.*BETA+2.*(1.-VUB)*ALFA)*COB
      SM(35)=-3./2.*(1.+VUB)*COB
      SM(36)=(4.*ALFA+2.*(1.-VUB)*BETA)*COB
      RETURN
      END
```

THESIS*AMEF(1).ACSTIF

```
      SUBROUTINE ACSTIF(I)
      REAL KSI1,KSI2,NU1,NU2,KSIBAR,NUBAR
      DIMENSION A(500),B(500),SM(36),TETA(500),KSI1(10)
      DIMENSION KSI2(10),NU2(10),SL(10),SA(10),SV(10),NU1(10),PQCC(8)
      DIMENSION JOX(600),JOY(600),NODE(500,4),SR(3,8),EDEP(600)
      DIMENSION SKIP(110,9),JON(30),IDIR(30),MAG(30)
      DIMENSION NCODE(8),LL(600),S(26000),SJ(2,8),ITIP(500),SKIP1(250)
      DIMENSION GJJ(250),EJJ(250)
      DIMENSION STRESS(250,2),TAUIN(250),SIGIN(250),PRINST(12,10)
      COMMON STRESS,TAUIN,SIGIN,ITER,SIGALB,PRINST,LX
      COMMON VUS,VUB,JOX,JOY,NODE,TETA,JL,JBAND,A,B,LJB,NJB,SR,KSI
      COMMON NU,X,SJ,KS,KN,SIGAL,ITIP,SKIP1,GJJ,EJJ,DELTA,ICR
      COMMON MSS,TJ,EDEP,SKIP,MJ,JON,IDIR,MAG,PQCC,NJ,TH,ME,EC,ES,EB,VUC
      COMMON N,MS,INT,IINT,NC,MEC,MEJ,GJ,EJ,SM,NCODE,LL,S,NLOAD,MAXS,MEB
C   STIFFNESS OF R.C. MEMBERS
      COC=EC*TH/(12.*(1.-VUC**2))
      DS11=ES/(1.-VUS**2)
      DS12=VUS*DS11
      DS22=DS11
      DS33=ES/(2.*(1.+VUS))
      DC11=EC/(1.-VUC**2)
      DC12=VUC*DC11
      DC22=DC11
      DC33=EC/(2.*(1.+VUC))
      READ 89,NR
   89 FORMAT(I5)
      IF(NR.EQ.0) GO TO 13
      READ 90,(KSI1(L),NU1(L),KSI2(L),NU2(L),SA(L),SL(L),L=1,NR)
   90 FORMAT(6F10.0)
C     READING IN REINFORCEMENT DATA
C     NR: NUMBER OF REINFORCING BARS
C     KSI1,KSI2,NU1,NU2 ARE COORDINATES OF STEEL BARS
C     SA:STEEL AREA
C     SL:STEEL LENGTH
C     SV:STEEL VOLUME
   13 C1=(DS11-DC11)/A(I)**2.
      C2=(DS33-DC33)/B(I)**2.
      C3=(DS12-DC12+DS33-DC33)/(A(I)*B(I))
      C4=(DS12-DC12)/(A(I)*B(I))
      C5=(DS33-DC33)/(A(I)*B(I))
      C6=(DS22-DC22)/B(I)**2.
      C7=(DS33-DC33)/A(I)**2.
      ALFA=A(I)/B(I)
      BETA=B(I)/A(I)
C     STIFFNESS COEFF. OF CONCRETE PART OF THE RC ELEMENTS
      SM(1)=(4.*BETA+2.*(1.-VUC)*ALFA)*COC
      SM(2)=3./2.*(1.+VUC)*COC
```

```
      SM(3)=(2.*BETA-2.*(1.-VUC)*ALFA)*COC
      SM(4)=3./2.*(1.-3.*VUC)*COC
      SM(5)=(-2.*BETA-(1.-VUC)*ALFA)*COC
      SM(6)=-3./2.*(1.+VUC)*COC
      SM(7)=(-4.*BETA+(1.-VUC)*ALFA)*COC
      SM(8)=-3./2.*(1.-3.*VUC)*COC
      SM(9)=(4.*ALFA+2.*(1.-VUC)*BETA)*COC
      SM(10)=-3./2.*(1.-3.*VUC)*COC
      SM(11)=(-4.*ALFA+(1.-VUC)*BETA)*COC
      SM(12)=-3./2.*(1.+VUC)*COC
      SM(13)=(-2.*ALFA-(1.-VUC)*BETA)*COC
      SM(14)=(3./2.*(1.-3.*VUC))*COC
      SM(15)=(2.*ALFA-2.*(1.-VUC)*BETA)*COC
      SM(16)=(4.*BETA+2.*(1.-VUC)*ALFA)*COC
      SM(17)=-3./2.*(1.+VUC)*COC
      SM(18)=(-4.*BETA+(1.-VUC)*ALFA)*COC
      SM(19)=3./2.*(1.-3.*VUC)*COC
      SM(20)=(-2.*BETA-(1.-VUC)*ALFA)*COC
      SM(21)=3./2.*(1.+VUC)*COC
      SM(22)=(4.*ALFA+2.*(1.-VUC)*BETA)*COC
      SM(23)=-3./2.*(1.-3.*VUC)*COC
      SM(24)=(2.*ALFA-2.*(1.-VUC)*BETA)*COC
      SM(25)=3./2.*(1.+VUC)*COC
      SM(26)=(-2.*ALFA-(1.-VUC)*BETA)*COC
      SM(27)=(4.*BETA+2.*(1.-VUC)*ALFA)*COC
      SM(28)=3./2.*(1.+VUC)*COC
      SM(29)=(2.*BETA-2.*(1.-VUC)*ALFA)*COC
      SM(30)=3./2.*(1.-3.*VUC)*COC
      SM(31)=(4.*ALFA+2.*(1.-VUC)*BETA)*COC
      SM(32)=-3./2.*(1.-3.*VUC)*COC
      SM(33)=(-4.*ALFA+(1.-VUC)*BETA)*COC
      SM(34)=(4.*BETA+2.*(1.-VUC)*ALFA)*COC
      SM(35)=-3./2.*(1.+VUC)*COC
      SM(36)=(4.*ALFA+2.*(1.-VUC)*BETA)*COC
C    STIFFNESS COEFF. CONTRIBUTIONS OF REINFORCING BARS
      DO 21 L=1,NR
      C=(KSI2(L)-KSI1(L))/SL(L)
      SS=(NU2(L)-NU1(L))/SL(L)
      KSIBAR=1.-KSI1(L)
      NUBAR=1.-NU1(L)
      SV(L)=SA(L)*SL(L)
      P1=NUBAR**2
      P2=KSIBAR**2
      P3=NUBAR*NU1(L)
      P4=KSIBAR*KSI1(L)
      P5=NUBAR*KSIBAR
      P6=NUBAR*KSI1(L)
      P7=NU1(L)*KSIBAR
```

```
      P8=NU1(L)*KSI1(L)
      P9=NU1(L)**2
      P10=KSI1(L)**2
      R1=(C1*SS**2+C2*C**2)*SL(L)**2/3.
      R2=C3*SS*C*SL(L)**2/3.
      R3=(C6*C**2+C7*SS**2)*SL(L)**2/3.
      T1=C1*NU1(L)*SS*SL(L)
      T2=C1*NUBAR*SS*SL(L)
      T3=C2*KSI1(L)*C*SL(L)
      T4=C2*KSIBAR*C*SL(L)
      T5=C3*NU1(L)*C*SL(L)/2.
      T6=C3*NUBAR*C*SL(L)/2.
      T7=C3*KSI1(L)*SS*SL(L)/2.
      T8=C3*KSIBAR*SS*SL(L)/2.
      T9=C6*KSI1(L)*C*SL(L)
      T10=C6*KSIBAR*C*SL(L)
      T11=C7*NU1(L)*SS*SL(L)
      T12=C7*NUBAR*SS*SL(L)
      T13=C*(C4*NU1(L)-C5*NUBAR)*SL(L)/2.
      T14=C*(C4*NUBAR-C5*NU1(L))*SL(L)/2.
      T15=SS*(C4*KSI1(L)-C5*KSIBAR)*SL(L)/2.
      T16=SS*(C4*KSIBAR-C5*KSI1(L))*SL(L)/2.
      T17=C1*SS*(NU1(L)-NUBAR)*SL(L)/2.
      T18=C2*C*(KSIBAR-KSI1(L))*SL(L)/2.
      T19=C6*C*(KSIBAR-KSI1(L))*SL(L)/2.
      T20=C7*SS*(NU1(L)-NUBAR)*SL(L)/2.
C STIFFNESS COEFF.OF CONCRETE AND STEEL PARTS ARE SUMMED UP
      SM(1)=SM(1)+SV(L)*(C1*P1+C2*P2-T2-T4+R1)
      SM(2)=SM(2)+SV(L)*(C3*P5-T6-T8+R2)
      SM(3)=SM(3)+SV(L)*(C1*P3-C2*P2+T4-T17-R1)
      SM(4)=SM(4)+SV(L)*(-C4*P5+C5*P7+T8+T14-R2)
      SM(5)=SM(5)+SV(L)*(-C1*P3-C2*P4+T17-T18+R1)
      SM(6)=SM(6)+SV(L)*(-C4*P6-C5*P7-T14+T15+R2)
      SM(7)=SM(7)+SV(L)*(-C1*P1+C2*P4+T2+T18-R1)
      SM(8)=SM(8)+SV(L)*(C4*P6-C5*P5+T6-T15-R2)
      SM(9)=SM(9)+SV(L)*(C6*P2+C7*P1-T10-T12+R3)
      SM(10)=SM(10)+SV(L)*(C4*P7-C5*P5+T8-T13-R2)
      SM(11)=SM(11)+SV(L)*(-C6*P2+C7*P3+T10-T20-R3)
      SM(12)=SM(12)+SV(L)*(-C4*P7-C5*P6+T13-T16+R2)
      SM(13)=SM(13)+SV(L)*(-C6*P4-C7*P3-T19+T20+R3)
      SM(14)=SM(14)+SV(L)*(-C4*P5+C5*P6+T6+T16-R2)
      SM(15)=SM(15)+SV(L)*(C6*P4-C7*P1+T12+T19-R3)
      SM(16)=SM(16)+SV(L)*(C1*P9+C2*P2+T1-T4+R1)
      SM(17)=SM(17)+SV(L)*(-C3*P7+T5-T8+R2)
      SM(18)=SM(18)+SV(L)*(-C1*P9+C2*P4-T1+T18-R1)
      SM(19)=SM(19)+SV(L)*(-C4*P8+C5*P7-T5-T15-R2)
      SM(20)=SM(20)+SV(L)*(-C1*P3-C2*P4+T17-T18+R1)
      SM(21)=SM(21)+SV(L)*(C4*P8+C5*P5+T13+T15+R2)
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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
ΔPRT,S AMEF.AGENER,.AGSEL,.ASJON,.AJONST,.AJCRAC,.ASREC .    .3/.1+ISK=ISK
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```
THESIS*AMEF(1).AGENER
      SUBROUTINE AGENER
      DIMENSION JOX(600),JOY(600),NCODE(8),SM(36),S(26000),ITIP(500)
      DIMENSION MAG(30),JON(30),IDIR(30),SKIP1(250)
      DIMENSION NODE(500,4),TETA(500),A(500),B(500),SR(3,8)
      DIMENSION EDEP(600),SKIP(110,9),LL(600),SJ(2,8)
      DIMENSION PQCC(8),GJJ(250),EJJ(250)
      DIMENSION STRESS(250,2),TAUIN(250),SIGIN(250),PRINST(12,10)
      COMMON STRESS,TAUIN,SIGIN,ITER,SIGALB,PRINST,LX
      COMMON VUS,VUB,JOX,JOY,NODE,TETA,JL,JBAND,A,B,LJB,NJB,SR,KSI
      COMMON NU,X,SJ,KS,KN,SIGAL,ITIP,SKIP1,GJJ,EJJ,DELTA,ICK
      COMMON MSS,TJ,EDEP,SKIP,MJ,JON,IDIR,MAG,PQCC,NJ,TH,ME,EC,ES,EB,VUC
      COMMON N,MS,INT,IINT,NC,MEC,MEJ,GJ,EJ,SM,NCODE,LL,S,NLOAD,MAXS,MEB
C GENERATION OF SYSTEMS EQUATIONS MATRIX (UNIDIMENSIONAL)
C WITH CODE NUMBER TECHNIQUE
      LOC(II,J)=II*MS-II*(II-1)/2-(MS-J)
      IUCGEN(I)=(I-N+JBAND-1)*(I-N+JBAND)/2
      NHEP=(N-JBAND)*JBAND+JBAND*(JBAND+1)/2+N*NLOAD
      IF(NHEP-MAXS) 70,70,71
   71 PRINT 72,NHEP,MAXS
   72 FORMAT(///36HPROBLEM SIZE TOO LARGE FOR S  MATRIX,2I5)
      PRINT 777
  777 FORMAT(1H1)
      CALL EXIT
   70 CONTINUE
      DO 62 I=1,NHEP
   62 S(I)=0.
      DO 9 NM=1,ME
      LD=NM
      READ (10,LD)(SM(K),K=1,MSS),(NCODE(M),M=1,MS)
      LJB=NLOAD+JBAND
      NJB=N-JBAND
      DO 8 L=1,MS
      SAYN=1.
      I=NCODE(L)
      IF(I) 20,8,22
   20 SAYN=-1.
      I=-I
   22 CONTINUE
      IX=(I-1)*LJB-I+1
      IUC=IUCGEN(I-1)
      DO 77 M=1,MS
      SAYN2=1.
      J=NCODE(M)
      IF(J) 30,77,32
   30 SAYN2=-1.
      J=-J
   32 IF(J-I) 77,78,78
```

```
   78 ID=L
      JD=M
      IF(L-M) 122,122,123
  123 ID=M
      JD=L
  122 LC=LOC(ID,JD)
      LO=IX+J
      IF(I-NJB-1) 79,79,80
   80 LO=LO-IUC
   79 S(LO)=S(LO)+SAYN*SAYN2*SM(LC)
   77 CONTINUE
    8 CONTINUE
    9 CONTINUE
      DO 700 IK=1,MJ
      WI=MAG(IK)/INT
      IJ=JON(IK)
      ID=IDIR(IK)
      GO TO (81,82) ID
   81 NUM=JOX(IJ)
      GO TO 83
   82 NUM=JOY(IJ)
   83 SAYN=1.
      I=NUM
      IF(I) 51,700,52
   51 SAYN=-1.
      NUM=-NUM
   52 I=NUM
      LO=(I-1)*LJB+JBAND+1
      IF(I-(NJB+1)) 791,791,801
  801 LO=LO-IUCGEN(I)
  791 S(LO)=S(LO)+SAYN*WI
  700 CONTINUE
      MC=MEJ+1
      DO 10 MN=MC,ME
      LD=MN
      LE=LD-MEJ
      READ (10,LD)(SM(K),K=1,MSS),(NCODE(M),M=1,MS)
      READ(29,LE)(PQCC(I),I=1,8)
      DO 45 NA=1,8
      NN=NCODE(NA)
      IF(NN) 46,45,48
   46 SAYN=-1
      NN=-NN
   48 LO=(NN-1)*LJB+JBAND+1
      IF(NN-(NJB+1)) 49,49,55
   55 LO=LO-IUCGEN(NN)
   49 S(LO)=S(LO)+PQCC(NA)
   45 CONTINUE
```

```
      IF(ITER.NE.1) GO TO 10
      DO 16 IM=1'8
   16 PQCC(IM)=0.
      WRITE(29,LE)(PQCC(I),I=1,8)
   10 CONTINUE
      RETURN
      END
```

THESIS*AMEF(1).AGSEL

```
      SUBROUTINE AGSEL
      DIMENSION LL(600),S(26000),Z(26000),ITIP(500),SKIP1(250),GJJ(250)
      DIMENSION JOX(600),JOY(600),NODE(500,4),TETA(500)
      DIMENSION A(500),B(500),SR(3,8),EDEP(600),SKIP(110,9),JON(30)
      DIMENSION IDIR(30),MAG(30),PQCC(8),SM(36),NCODE(8)
      DIMENSION SJ(2,8),EJJ(250)
      DIMENSION STRESS(250,2),TAUIN(250),SIGIN(250),PRINST(12,10)
      COMMON STRESS,TAUIN,SIGIN,ITER,SIGALB,PRINST,LX
      COMMON VUS,VUB,JOX,JOY,NODE,TETA,JL,JBAND,A,B,LJB,NJB,SR,KSI
      COMMON NU,X,SJ,KS,KN,SIGAL,ITIP,SKIP1,GJJ,EJJ,DELTA,ICR
      COMMON MSS,TJ,EDEP,SKIP,MJ,JON,IDIR,MAG,PQCC,NJ,TH,ME,EC,ES,EB,VUC
      COMMON N,MS,INT,IINT,NC,MEC,MEJ,GJ,EJ,SM,NCODE,LL,S,NLOAD,MAXS,MEB
      EQUIVALENCE (S,Z)
      JB=JBAND
      NE=N-1
      N1=N+1
      NL=N+NLOAD
      NM=NLOAD-1
      JBE=JB-1
      NF=N-JBE
      ND=JBE+NLOAD
      LL(1)=0
      J2=0
      JCOR=0
      DO 40 I=1,N
      J1=J2+1
      IF(I-NF) 41,41,42
   41 J2=J1+ND
      GO TO 43
   42 J2=J1+NL-I
   43 DO 1000 J=J1,J2
      JCOR=JCOR+1
 1000 Z(J)=Z(JCOR)
      J3=J2-NLOAD
      JA=J3+J1
      DO 44 K=J1,J3
      J=JA-K
      IF(Z(J)) 50,44,50
   44 CONTINUE
   50 LT=J-J1+1
      IF(LT+1-LL(I)) 51,52,52
   51 LT=LL(I)-1
   52 JT=J3-J1-LT+1
      IF(JT) 40,40,55
   55 JP=J3+1
      DO 56 J=JP,J2
      K=J-JT
```

```
   56 Z(K)=Z(J)
      J2=J2-JT
   40 LL(I+1)=LT
      NX=0
      DO 7 I=1,N
      NX=NX+LL(I+1)+NLOAD
    7 LL(I+1)=NX-I
      NX=LL(N)+N
      NY=N*NLOAD
      NZ=NX+NLOAD
      NT=NZ-NY
C  ELIMINASYON
      DO 10 K=1,NE
      NBK=LL(K)
      KK=NBK+K
      Q=1./Z(KK)
      Z(KK)=Q
      IB=K+1
      K2=LL(IB)+K
      IS=K2-NBK-NLOAD
      IF(IS-N) 12,11,11
   11 IE=N
      IS=NL
      GO TO 17
   12 IE=IS
      IF(IB-IE) 22,22,10
   22 K1=K2-NM
   17 J2=NBK+IS
      IN=IS-IE
      DO 13 I=IB,IE
      KI=NBK+I
      IF(Z(KI)) 14,13,14
   14 TA=Q*Z(KI)
      IH=LL(I)-NBK
      DO 15 KJ=K1,J2
      IJ=KJ+IH
      Z(IJ)=Z(IJ)-TA*Z(KJ)
   15 CONTINUE
      IF(IN) 18,18,13
   18 IH=LL(I+1)+I-K2
      DO 16 KJ=K1,K2
      IJ=KJ+IH
      Z(IJ)=Z(IJ)-TA*Z(KJ)
   16 CONTINUE
   13 CONTINUE
   10 CONTINUE
C  YERINE KOYMA
      KI=NX+1
```

```
      Q=1./Z(NX)
      DO 20 J=KI,NZ
   20 Z(J)=Q*Z(J)
      DO 38 IX=1,NE
      I=N-IX
      IB=I+1
      NBI=LL(I)
      II=NBI+I
      IC=LL(IB)+I
      Q=Z(II)
      IS=IC-NBI-NLOAD
      DO 36 K=1,NLOAD
      NK=NLOAD-K
      IK=IC-NK
      W=Z(IK)
      IF(IB-IS) 39,39,36
   39 DO 37 J=IB,IS
      IJ=NBI+J
      JK=LL(J+1)+J-NK
   37 W=W-Z(IJ)*Z(JK)
   36 Z(IK)=W*Q
   38 CONTINUE
      RETURN
      END
```

```
THESIS*AMEF(1).ASJON
      SUBROUTINE ASJON(I)
      REAL JL,KS,KN
      DIMENSION SJ(2,8),PQCC(8),SM(36),NCODE(8),LL(600),S(26000)
      DIMENSION JOX(600),JOY(600),NODE(500,4),TETA(500),A(500),B(500)
      DIMENSION SR(3,8),EDEP(600),SKIP(110,9),JON(30),IDIR(30),MAG(30)
      DIMENSION ITIP(500),SKIP1(250),GJJ(250),EJJ(250)
      DIMENSION STRESS(250,2),TAUIN(250),SIGIN(250),PRINST(12,10)
      COMMON STRESS,TAUIN,SIGIN,ITER,SIGALB,PRINST,LX
      COMMON VUS,VUB,JOX,JOY,NODE,TETA,JL,JBAND,A,B,LJB,NJB,SR,KSI
      COMMON NU,X,SJ,KS,KN,SIGAL,ITIP,SKIP1,GJJ,EJJ,DELTA,ICR
      COMMON MSS,TJ,EDEP,SKIP,MJ,JON,IDIR,MAG,PQCC,NJ,TH,ME,EC,ES,EB,VUC
      COMMON N,MS,INT,IINT,NC,MEC,MEJ,GJ,EJ,SM,NCODE,LL,S,NLOAD,MAXS,MEB
C     SUBROUTINE FOR DEVELOPING STRESS MATRIX OF JOINT ELEMENTS
      JL=A(I)
      AC=1.-2.*X/JL
      BC=1.+2.*X/JL
      KS=GJJ(I)*TH/TJ
      KN=EJJ(I)*TH/TJ
      SJ(1,1)=-AC/2.*KS
      SJ(1,2)=0.
      SJ(1,3)=AC/2.*KS
      SJ(1,4)=0.
      SJ(1,5)=BC/2.*KS
      SJ(1,6)=0.
      SJ(1,7)=-BC/2.*KS
      SJ(1,8)=0.
      SJ(2,1)=0.
      SJ(2,2)=-AC/2.*KN
      SJ(2,3)=0.
      SJ(2,4)=AC/2.*KN
      SJ(2,5)=0.
      SJ(2,6)=BC/2.*KN
      SJ(2,7)=0.
      SJ(2,8)=-BC/2.*KN
      RETURN
      END
```

```
THESIS*AMEF(1).AJONST
      SUBROUTINE AJONST(I)
      REAL JL,KS,KN
      DIMENSION SM(36),NCODE(8),EDEF(8),LL(600),SJ(2,8)
      DIMENSION STRESS(250,2),EDEP(600),S(26000),SEDEF(8),TETA(500)
      DIMENSION TAUIN(250),SIGIN(250),CEDEF(8),PRINST(12,10)
      DIMENSION JOX(600),JOY(600),NODE(500,4),A(500),B(500)
      DIMENSION SR(3,8),SKIP(110,9),JON(30),IDIR(30),MAG(30)
      DIMENSION PQCC(8),ITIP(500),SKIP1(250),GJJ(250),EJJ(250)
      COMMON STRESS,TAUIN,SIGIN,ITER,SIGALB,PRINST,LX
      COMMON VUS,VUB,JOX,JOY,NODE,TETA,JL,JBAND,A,B,LJB,NJB,SR,KSI
      COMMON NU,X,SJ,KS,KN,SIGAL,ITIP,SKIP1,GJJ,EJJ,DELTA,ICR
      COMMON MSS,TJ,EDEP,SKIP,MJ,JON,IDIR,MAG,PQCC,NJ,TH,ME,EC,ES,EB,VUC
      COMMON N,MS,INT,IINT,NC,MEC,MEJ,GJ,EJ,SM,NCODE,LL,S,NLOAD,MAXS,MEB
C     STRESS CALCULATION IN JOINT ELEMENTS AND REARRANGEMENT OF ELASTIC
C     AND SHEAR MODULI ACC. TO STRESS STATE PRESENT
      IF(SKIP1(I).EQ.1.) RETURN
      LD=I
      READ (10,LD)(SM(K),K=1,MSS),(NCODE(M),M=1,MS)
      DO 20 J=1,MS
      EDEF(J)=0.
      CEDEF(J)=0.
      SAYN=1.
      IN=NCODE(J)
      IF(IN) 21,20,22
   21 IN=-IN
      SAYN=-1.
   22 IX=LL(IN+1)+IN-(NLOAD-1)
      EDEF(J)=S(IX)*SAYN
      CEDEF(J)=EDEF(J)+EDEP(IN)*SAYN
   20 CONTINUE
      DO 24 J=1,8
      SEDEF(J)=EDEF(J)
   24 CONTINUE
C     TRANSFORMATION OF ELEMENT DISPLACEMENTS IF JOINT ELEMENT IS
C     IN VERTICAL POSITION
      IF(TETA(I).EQ.0.) GO TO 201
      EDEF(1)=SEDEF(2)
      EDEF(2)=-SEDEF(1)
      EDEF(3)=SEDEF(4)
      EDEF(4)=-SEDEF(3)
      EDEF(5)=SEDEF(6)
      EDEF(6)=-SEDEF(5)
      EDEF(7)=SEDEF(8)
      EDEF(8)=-SEDEF(7)
  201 CONTINUE
      DO 30 L=1,2
      SUM=0.
```

```
      DO 40 M=1,MS
   40 SUM=SUM+SJ(L,M)*EDEF(M)
   30 STRESS(I,L)=SUM/TH
      TAU=TAUIN(I)+STRESS(I,1)
      SIGMA=SIGIN(I)+STRESS(I,2)
      TAU=ABS(TAU)
      IF(SKIP1(I).EQ.0.) GO TO 35
C    ASSIGNING NEW MATERIAL PROPERTIES TO ALREADY CRACKED JOINTS
C    ACCORDING TO CALCULATED COMPRESSIVE STRESS
      IF(SIGMA.LT.(-334.)) GO TO 15
      GJJ(I)=-SIGMA*3630./334.
      GO TO 3000
   15 GJJ(I)=3630.
      GO TO 3000
   35 IF(SIGMA.LE.28..AND.SIGMA.GT.(-100.)) EJJ(I)=292100.-ABS(SIGMA)/10
     *0.*122000.
      IF(SIGMA.LE.(-100.).AND.SIGMA.GT.(-200.)) EJJ(I)=170100.-(ABS(SIGM
     *A)-100.)/100.*44100.
      IF(SIGMA.LE.(-200.).AND.SIGMA.GT.(-300.)) EJJ(I)=126000.-(ABS(SIGM
     *A)-200.)/100.*28200.
      IF(SIGMA.LE.(-300.).AND.SIGMA.GT.(-400.)) EJJ(I)=97800.-(ABS(SIGMA
     *)-300.)/100.*15300.
      IF(SIGMA.LE.(-400.).AND.SIGMA.GT.(-500.)) EJJ(I)=82500.-(ABS(SIGMA
     *)-400.)/100.*17400.
      IF(SIGMA.LE.(-500.).AND.SIGMA.GT.(-600.)) EJJ(I)=65100.-(ABS(SIGMA
     *)-500.)/100.*16800.
      IF(SIGMA.LE.(-600.).AND.SIGMA.GT.(-700.)) EJJ(I)=48300.-(ABS(SIGMA
     *)-600.)/100.*5400.
      IF(SIGMA.LE.(-700.).AND.SIGMA.GT.(-800.)) EJJ(I)=42900.-(ABS(SIGMA
     *)-700.)/100.*6600.
      IF(SIGMA.LE.(-800.)) EJJ(I)=36300.
      IF(TAU.LT.25.) GJJ(I)=128000.-TAU/25.*72500.
      IF(TAU.GE.25..AND.TAU.LT.50.) GJJ(I)=55500.-(TAU-25.)/25.*33300.
      IF(TAU.GE.50..AND.TAU.LT.75.) GJJ(I)=22200.-(TAU-50.)/25.*8900.
      IF(TAU.GE.75..AND.TAU.LT.100.) GJJ(I)=13300.-(TAU-75.)/25.*4600.
      IF(TAU.GE.100..AND.TAU.LT.125.) GJJ(I)=8700.-(TAU-100.)/25.*3500.
      IF(TAU.GE.125..AND.TAU.LT.150.) GJJ(I)=5200.-(TAU-125.)/25.*1570.
      IF(TAU.GE.125.) GJJ(I)=3630.
 3000 CALL AJSTIF(I)
      WRITE(10,LU)(SM(K),K=1,MSS)
      RETURN
      END
```

```
THESIS*AMEF(1).AJCRAC
      SUBROUTINE AJCRAC(I)
      REAL JL,KS,KN
      DIMENSION SM(36),NCODE(8),EDEF(8),LL(600),SJ(2,8)
      DIMENSION STRESS(250,2),EDEP(600),S(26000),SEDEF(8),TE!A(500)
      DIMENSION JOX(600),JOY(600),NODE(500,4),A(500),B(500)
      DIMENSION SR(3,8),SKIP(110,9),JON(30),IDIR(30),MAG(30)
      DIMENSION PQCC(8),ITIP(500),SKIP1(250),GJJ(250),EJJ(250)
      DIMENSION TAUIN(250),SIGIN(250),PRINST(12,10)
      COMMON STRESS,TAUIN,SIGIN,ITER,SIGALB,PRINST,LX
      COMMON VUS,VUB,JOX,JOY,NODE,TETA,JL,JBAND,A,B,LJB,NJB,SR,KSI
      COMMON NU,X,SJ,KS,KN,SIGAL,ITIP,SKIP1,GJJ,EJJ,DELTA,ICR
      COMMON MSS,TJ,EDEP,SKIP,MJ,JON,IDIR,MAG,PQCC,NJ,TH,ME,EC,ES,EB,VUC
      COMMON N,MS,INT,IINT,NC,MEC,MEJ,GJ,EJ,SM,NCODE,LL,S,NLOAD,MAXS,MEB
C   SUBROUTINE FOR CHECKING CRACS IN JOINT ELEMENTS AND ASSIGNING
C   RESIDUAL MATERIAL PROPERTIES IF NECESSARY
      IF(SKIP1(I).NE.0.) RETURN
      LD=I
      TAU=TAUIN(I)+STRESS(I,1)
      SIGMA=SIGIN(I)+STRESS(I,2)
      TAU=ABS(TAU)
C   IREG:REGION OF JOINT FAILURE,IF 1 TENSILE,IF 2 OR 3 SHEAR FAILURE
      IREG=0
      IF(SIGMA.LE.0.) GO TO 100
      IF(SIGMA.LE.28.) GO TO 50
      IREG=1
      GO TO 1000
   50 UTAU=-1.5*SIGMA+42.
      IF(TAU.GT.UTAU) IREG=1
      GO TO 1000
  100 IF(SIGMA.LT.(-334.)) GO TO 200
      UTAU=-0.75*SIGMA+42.
      IF(TAU.GT.UTAU) IREG=2
      GO TO 1000
  200 UTAU=-0.11*SIGMA+254.
      IF(TAU.GT.UTAU) IREG=3
 1000 IF(IREG.EQ.0) RETURN
      ICR=1
      TAU=TAUIN(I)+STRESS(I,1)
      GO TO (210'220,230) IREG
  210 GJJ(I)=0.
      EJJ(I)=0.
      SKIP1(I)=1.
      PRINT 80,I'IREG,TAU,SIGMA
   80 FORMAT(100(,*,)/50X,18H   CRACK IN JOINT ,I3/40X,24HTYPE OF FAILUR
     *E: TENSILE'6X,8HREGION: ,I1/40X,6H   TAU=,F10.2,6X,6HSIGMA=,F10.2/1
     *00(,*,))
      GO TO 3000
```

```
  220 GJJ(I)=-SIGMA*3630./334.
      SKIP1(I)=2.
      PRINT 90,I,IREG,TAU,SIGMA
   90 FORMAT(100(',*,')/50X,18H   CRACK IN JOINT ,I3/40X,24HTYPE OF FAILUR
     *E: SHEAR   ',6X,8HREGION: ,I1/40X,6H   TAU=,F10.2,6X,6HSIGMA=,F10.2/1
     *00(',*,'))
      GO TO 3000
  230 GJJ(I)=3630.
      SKIP1(I)=2.
      PRINT 90,I,IREG,TAU,SIGMA
      GO TO 3000
 3000 CALL AJSTIF(I)
      WRITE(10,LD)(SM(K),K=1,MSS)
      RETURN
      END
```

```
THESIS*AMEF(1).ASREC
      SUBROUTINE ASREC(I)
      REAL KSI,NU
      DIMENSION PQCC(8),SJ(2,8),ITIP(500)
      DIMENSION SKIP1(250),GJJ(250),EJJ(250)
      DIMENSION A(500),B(500),SR(3,8),SM(36),NCODE(8),LL(600),S(26000)
      DIMENSION JOX(600),JOY(600),NODE(500,4),TETA(500)
      DIMENSION EDEP(600),SKIP(110,9),JON(30),IDIR(30),MAG(30)
      DIMENSION STRESS(250,2),TAUIN(250),SIGIN(250),PRINST(12,10)
      COMMON STRESS,TAUIN,SIGIN,ITER,SIGALB,PRINST,LX
      COMMON VUS,VUB,JOX,JOY,NODE,TETA,JL,JBAND,A,B,LJB,NJB,SR,KSI
      COMMON NU,X,SJ,KS,KN,SIGAL,ITIP,SKIP1,GJJ,EJJ,DELTA,ICR
      COMMON MSS,TJ,EDEP,SKIP,MJ,JON,IDIR,MAG,PQCC,NJ,TH,ME,EC,ES,EB,VUC
      COMMON N,MS,INT,IINT,NC,MEC,MEJ,GJ,EJ,SM,NCODE,LL,S,NLOAD,MAXS,MEB
C     SUBROUTINE FOR CALCULATING STRESS MATRIX OF R.C. ELEMENTS
      C=EC/(1.-VUC*VUC)
      SR(1,1)=-C*(1.-NU)/A(I)
      SR(1,2)=-C*VUC*(1.-KSI)/B(I)
      SR(1,3)=-C*NU/A(I)
      SR(1,4)=C*VUC*(1.-KSI)/B(I)
      SR(1,5)=C*NU/A(I)
      SR(1,6)=C*VUC*KSI/B(I)
      SR(1,7)=C*(1.-NU)/A(I)
      SR(1,8)=-C*VUC*KSI/B(I)
      SR(2,1)=-C*VUC*(1.-NU)/A(I)
      SR(2,2)=-C*(1.-KSI)/B(I)
      SR(2,3)=-C*VUC*NU/A(I)
      SR(2,4)=C*(1.-KSI)/B(I)
      SR(2,5)=C*VUC*NU/A(I)
      SR(2,6)=C*KSI/B(I)
      SR(2,7)=C*VUC*(1.-NU)/A(I)
      SR(2,8)=-C*KSI/B(I)
      SR(3,1)=-C*(1.-VUC)*(1.-KSI)/(2.*B(I))
      SR(3,2)=-C*(1.-VUC)*(1.-NU)/(2.*A(I))
      SR(3,3)=C*(1.-VUC)*(1.-KSI)/(2.*B(I))
      SR(3,4)=-C*(1.-VUC)*NU/(2.*A(I))
      SR(3,5)=C*(1.-VUC)*KSI/(2.*B(I))
      SR(3,6)=C*(1.-VUC)*NU/(2.*A(I))
      SR(3,7)=-C*(1.-VUC)*KSI/(2.*B(I))
      SR(3,8)=C*(1.-VUC)*(1.-NU)/(2.*A(I))
      RETURN
      END
ΔPRT,S AMEF.ACONST,.ASBRIC,.ABRIST .                    EUNITNUC 0001
```

THESIS*AMEF(1).ACONST

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

```
      CALL ASREC(I)
      DO 50 L=1,3
      SUM=0.
      DO 60 M=1,MS
   60 SUM=SUM+SR(L,M)*EDEF(M)
   50 SC(L)=SUM
      IF(LD.EQ.161.OR.LD.EQ.170.OR.LD.EQ.201.OR.LD.EQ.208) GO TO 17
      GO TO 31
   17 IF(IR.EQ.2.OR.IR.EQ.5.OR.IR.EQ.8) GO TO 18
      GO TO 31
   18 IR1=IR+2
      J=0
      DO 10 LI=IR,IR1
      J=J+1
   10 PRINST(LX,LI)=SC(J)
   31 CONTINUE
      SCP(1)=(SC(1)+SC(2))/2.+SQRT(((SC(1)-SC(2))/2.)**2+SC(3)**2)
      SCP(2)=(SC(1)+SC(2))/2.-SQRT(((SC(1)-SC(2))/2.)**2+SC(3)**2)
      SCP(3)=0.
      TETAP=0.5*ATAN(2.*SC(3)/(SC(1)-SC(2)))
      PI=ATAN(1.)*4.
      TOL=0.00001
      SIGX=(SC(1)+SC(2))/2.+(SC(1)-SC(2))/2.*COS(2.*TETAP)+SC(3)*SIN(2.*
     1TETAP)
      IF(ABS(SIGX-SCP(1)).LE.TOL) GO TO 25
      TETAP=TETAP+PI/2.
   25 IF(SCP(1).LE.SIGAL) GO TO 46
      DTETAP=TETAP/PI*180.
      IF(SCP(2).LE.SIGAL) GO TO 80
      IF(SKIP(LE,IR).EQ.0.) PRINT 1,I,IR,DTETAP,SC(1),SC(2),SC(3)
      GO TO 100
   80 SCP(2)=0.
      IF(SKIP(LE,IR).EQ.0.) PRINT 2,I,IR,DTETAP,SC(1),SC(2),SC(3)
    1 FORMAT(25X,I3,9H. ELEMAN ,I1,24H. BOLGE IKI YONDE CATLAK,5X,
     113HCATLAK ACISI=,F5.0,2X,7HSIGMAX=,F6.1,2X,7HSIGMAY=,F6.1,2X,4HTAU
     2=,F6.1)
    2 FORMAT(25X,I3,9H. ELEMAN ,I1,24H. BOLGE BIR YONDE CATLAK,5X,
     113HCATLAK ACISI=,F5.0,2X,7HSIGMAX=,F6.1,2X,7HSIGMAY=,F6.1,2X,4HTAU
     2=,F6.1)
  100 SCC(1)=SCP(1)*COS(TETAP)*COS(TETAP)+SCP(2)*SIN(TETAP)*SIN(TETAP)
      SCC(2)=SCP(1)*SIN(TETAP)*SIN(TETAP)+SCP(2)*COS(TETAP)*COS(TETAP)
      SCC(3)=SCP(1)*SIN(TETAP)*COS(TETAP)-SCP(2)*SIN(TETAP)*COS(TETAP)
      READ(29,LE)(PQCC(KI),KI=1,8)
      PQCC(1)=PQCC(1)+(-B(I)*(1.-NU)*SCC(1)-A(I)*(1.-KSI)*SCC(3))*TH/9.
      PQCC(2)=PQCC(2)+(-A(I)*(1.-KSI)*SCC(2)-B(I)*(1.-NU)*SCC(3))*TH/9.
      PQCC(3)=PQCC(3)+(-B(I)*NU*SCC(1)+A(I)*(1.-KSI)*SCC(3))*TH/9.
      PQCC(4)=PQCC(4)+(A(I)*(1.-KSI)*SCC(2)-B(I)*KSI*SCC(3))*TH/9.
      PQCC(5)=PQCC(5)+(B(I)*NU*SCC(1)+A(I)*KSI*SCC(3))*TH/9.
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      PQCC(6)=PQCC(6)+(A(I)*KSI*SCC(2)+B(I)*NU*SCC(3))*TH/9.
      PQCC(7)=PQCC(7)+(B(I)*(1.-NU)*SCC(1)-A(I)*KSI*SCC(3))*TH/9.
      PQCC(8)=PQCC(8)+(-A(I)*KSI*SCC(2)+B(I)*(1.-NU)*SCC(3))*TH/9.
      WRITE(29,LE)(PQCC(KI),KI=1,8)
      SKIP(LE,IR)=1.
   46 NU=NU+1./3.
   40 CONTINUE
      KSI=KSI+1./3.
   30 CONTINUE
      RETURN
      END
```

```
THESIS*AMEF(1).ASBRIC
      SUBROUTINE ASBRIC(I)
      REAL KSI,NU
      DIMENSION PQCC(8),SJ(2,8),ITIP(500)
      DIMENSION SKIP1(250),GJJ(250),EJJ(250)
      DIMENSION A(500),B(500),SR(3,8),SM(36),NCODE(8),LL(600),S(26000)
      DIMENSION JOX(600),JOY(600),NODE(500,4),TETA(500)
      DIMENSION EDEP(600),SKIP(110,9),JON(30),IDIR(30),MAG(30)
      DIMENSION STRESS(250,2),TAUIN(250),SIGIN(250),PRINST(12,10)
      COMMON STRESS,TAUIN,SIGIN,ITER,SIGALB,PRINST,LX
      COMMON VUS,VUB,JOX,JOY,NODE,TETA,JL,JBAND,A,B,LJB,NJB,SR,KSI
      COMMON NU,X,SJ,KS,KN,SIGAL,ITIP,SKIP1,GJJ,EJJ,DELTA,ICR
      COMMON MSS,TJ,EDEP,SKIP,MJ,JON,IDIR,MAG,PQCC,NJ,TH,ME,EC,ES,EB,VUC
      COMMON N,MS,INT,IINT,NC,MEC,MEJ,GJ,EJ,SM,NCODE,LL,S,NLOAD,MAXS,MEB
C     SUBROUTINE FOR CALCULATING STRESS MATRIX OF BRICK ELEMENTS
      C=EB/(1.-VUB*VUB)
      SR(1,1)=-C*(1.-NU)/A(I)
      SR(1,2)=-C*VUB*(1.-KSI)/B(I)
      SR(1,3)=-C*NU/A(I)
      SR(1,4)=C*VUB*(1.-KSI)/B(I)
      SR(1,5)=C*NU/A(I)
      SR(1,6)=C*VUB*KSI/B(I)
      SR(1,7)=C*(1.-NU)/A(I)
      SR(1,8)=-C*VUB*KSI/B(I)
      SR(2,1)=-C*VUB*(1.-NU)/A(I)
      SR(2,2)=-C*(1.-KSI)/B(I)
      SR(2,3)=-C*VUB*NU/A(I)
      SR(2,4)=C*(1.-KSI)/B(I)
      SR(2,5)=C*VUB*NU/A(I)
      SR(2,6)=C*KSI/B(I)
      SR(2,7)=C*VUB*(1.-NU)/A(I)
      SR(2,8)=-C*KSI/B(I)
      SR(3,1)=-C*(1.-VUB)*(1.-KSI)/(2.*B(I))
      SR(3,2)=-C*(1.-VUB)*(1.-NU)/(2.*A(I))
      SR(3,3)=C*(1.-VUB)*(1.-KSI)/(2.*B(I))
      SR(3,4)=-C*(1.-VUB)*NU/(2.*A(I))
      SR(3,5)=C*(1.-VUB)*KSI/(2.*B(I))
      SR(3,6)=C*(1.-VUB)*NU/(2.*A(I))
      SR(3,7)=-C*(1.-VUB)*KSI/(2.*B(I))
      SR(3,8)=C*(1.-VUB)*(1.-NU)/(2.*A(I))
      RETURN
      END
```

THESIS*AMEF(1).ABRIST

```
      SUBROUTINE ABRIST(I)
      REAL KSI,NU
      DIMENSION SM(36),NCODE(8),EDEF(8),LL(600),A(500),B(500)
      DIMENSION SR(3,8),SC(3),SCP(3),SCC(3),S(26000),EDEP(600)
      DIMENSION SKIP(110,9),PQCC(8)
      DIMENSION JOX(600),JOY(600),NODE(500,4),TETA(500)
      DIMENSION JON(30),IDIR(30),MAG(30),SJ(2,8),ITIP(500),SKIP1(250)
      DIMENSION GJJ(250),EJJ(250)
      DIMENSION STRESS(250,2),TAUIN(250),SIGIN(250),PRINST(1Z,10)
      COMMON STRESS,TAUIN,SIGIN,ITER,SIGALB,PRINST,LX
      COMMON VUS,VUB,JOX,JOY,NODE,TETA,JL,JBAND,A,B,LJB,NJB,SR,KSI
      COMMON NU,X,SJ,KS,KN,SIGAL,ITIP,SKIP1,GJJ,EJJ,DELTA,ICR
      COMMON MSS,TJ,EDEP,SKIP,MJ,JON,IDIR,MAG,PQCC,NJ,TH,ME,EC,ES,EB,VUC
      COMMON N,MS,INT,IINT,NC,MEC,MEJ,GJ,EJ,SM,NCODE,LL,S,NLOAD,MAXS,MEB
C     SUBROUTINE FOR CALCULATING STRESSES IN BRICK ELEMENTS,CHECKING FOR
C     TENSION CRACKS AND ASSIGNING PSEUDOLOAD VECTOR IF NECESSARY
      IUCGEN(NN)=(NN-N+JBAND-1)*(NN-N+JBAND)/2
      LD=I
      LE=LD-MEJ
      IF(LD.EQ.105.OR.LD.EQ.112.OR.LD.EQ.119.OR.LD.EQ.126.OR.LD.EQ.133.O
     *R.LD.EQ.140.OR.LD.EQ.147.OR.LD.EQ.154) GO TO 27
      GO TO 15
   27 LX=LX+1
      DL=LD
      PRINST(LX,1)=DL
   15 IR=0
      READ (10,LD)(SM(K),K=1,MSS),(NCODE(M),M=1,MS)
      DO 20 J=1,MS
      EDEF(J)=0.
      SAYN=1.
      IN=NCODE(J)
      IF(IN) 21,20,22
   21 IN=-IN
      SAYN=-1.
   22 IX=LL(IN+1)+IN-(NLOAD-1)
      EDEF(J)=S(IX)*SAYN+EDEP(IN)*SAYN
   20 CONTINUE
      KSI=1./6.
      DO 30 IJ=1,3
      NU=1./6.
      DO 40 IK=1,3
      IR=IR+1
      IF(SKIP(LE,IR).NE.0..AND.ITER.NE.1) GO TO 46
      CALL ASBRIC(I)
      DO 50 L=1,3
      SUM=0.
      DO 60 M=1,MS
```

```
   60 SUM=SUM+SR(L,M)*EDEF(M)
   50 SC(L)=SUM
      IF(LD.EQ.105.OR.LD.EQ.112.OR.LD.EQ.119.OR.LD.EQ.126.OR.LD.EQ.133.O
     *R.LD.EQ.140.OR.LD.EQ.147.OR.LD.EQ.154) GO TO 28
      GO TO 31
   28 IF(IR.EQ.2.OR.IR.EQ.5.OR.IR.EQ.8) GO TO 29
      GO TO 31
   29 IR1=IR+2
      J=0
      DO 10 LI=IR,IR1
      J=J+1
   10 PRINST(LX,LI)=SC(J)
   31 CONTINUE
      SCP(1)=(SC(1)+SC(2))/2.+SQRT(((SC(1)-SC(2))/2.)**2+SC(3)**2)
      SCP(2)=(SC(1)+SC(2))/2.-SQRT(((SC(1)-SC(2))/2.)**2+SC(3)**2)
      SCP(3)=0.
      TETAP=0.5*ATAN(2.*SC(3)/(SC(1)-SC(2)))
      PI=ATAN(1.)*4.
      TOL=0.00001
      SIGX=(SC(1)+SC(2))/2.+(SC(1)-SC(2))/2.*COS(2.*TETAP)+SC(3)*SIN(2.*
     1TETAP)
      IF(ABS(SIGX-SCP(1)).LE.TOL) GO TO 25
      TETAP=TETAP+PI/2.
   25 IF(SCP(1).LE.SIGALB) GO TO 46
      DTETAP=TETAP/PI*180.
      IF(SCP(2).LE.SIGALB) GO TO 80
      IF(SKIP(LE,IR).EQ.0.)PRINT 1,I,IR,DTETAP,SC(1),SC(2),SC(3)
      GO TO 100
   80 SCP(2)=0.
      IF(SKIP(LE,IR).EQ.0.) PRINT 2,I,IR,DTETAP,SC(1),SC(2),SC(3)
    1 FORMAT(25X,I3,9H. TUGLA  ,I1,24H. BOLGE IKI YONDE CATLAK,5X,
     113HCATLAK ACISI=,F5.0,2X,7HSIGMAX=,F6.1,2X,7HSIGMAY=,F6.1,2X,4HTAU
     2=,F6.1)
    2 FORMAT(25X,I3,9H. TUGLA  ,I1,24H. BOLGE BIR YONDE CATLAK,5X,
     113HCATLAK ACISI=,F5.0,2X,7HSIGMAX=,F6.1,2X,7HSIGMAY=,F6.1,2X,4HTAU
     2=,F6.1)
  100 SCC(1)=SCP(1)*COS(TETAP)*COS(TETAP)+SCP(2)*SIN(TETAP)*SIN(TETAP)
      SCC(2)=SCP(1)*SIN(TETAP)*SIN(TETAP)+SCP(2)*COS(TETAP)*COS(TETAP)
      SCC(3)=SCP(1)*SIN(TETAP)*COS(TETAP)-SCP(2)*SIN(TETAP)*COS(TETAP)
      READ(29,LE)(PQCC(KI),KI=1,8)
      PQCC(1)=PQCC(1)+(-B(I)*(1.-NU)*SCC(1)-A(I)*(1.-KSI)*SCC(3))*TH/9.
      PQCC(2)=PQCC(2)+(-A(I)*(1.-KSI)*SCC(2)-B(I)*(1.-NU)*SCC(3))*TH/9.
      PQCC(3)=PQCC(3)+(-B(I)*NU*SCC(1)+A(I)*(1.-KSI)*SCC(3))*TH/9.
      PQCC(4)=PQCC(4)+(A(I)*(1.-KSI)*SCC(2)-B(I)*KSI*SCC(3))*TH/9.
      PQCC(5)=PQCC(5)+(B(I)*NU*SCC(1)+A(I)*KSI*SCC(3))*TH/9.
      PQCC(6)=PQCC(6)+(A(I)*KSI*SCC(2)+B(I)*NU*SCC(3))*TH/9.
      PQCC(7)=PQCC(7)+(B(I)*(1.-NU)*SCC(1)-A(I)*KSI*SCC(3))*TH/9.
      PQCC(8)=PQCC(8)+(-A(I)*KSI*SCC(2)+B(I)*(1.-NU)*SCC(3))*TH/9.
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      WRITE(29,LE)(PQCC(KI),KI=1,8)
      SKIP(LE,IR)=1.
   46 NU=NU+1./3.
   40 CONTINUE
      KSI=KSI+1./3.
   30 CONTINUE
      RETURN
      END
ΔPRT,S AMEF.AMEF,.ADATRE,.AJBAND,.AJSTIF,.ABSTIF,.ACSTIF .        0001 OT UG
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