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ANALYTICAL MODELLING OF PRESTRESSED CONCRETE BOX GIRDERS SUBJECTED TO COMBINED LOADING

by

© GRAHAM TAYLOR

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The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies and Research, for acceptance, a thesis entitled ANALYTICAL MODELLING OF PRESTRESSED CONCRETE BOX GIRDERS SUBJECTED TO COMBINED LOADING submitted by GRAHAM TAYLOR in partial fulfilment of the requirements for the degree of Doctor of Philosophy in Civil Engineering.

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ABSTRACT

An analytical computer model has been developed for the analysis of reinforced or prestressed concrete multi-celled box girders acted upon by loading combinations comprised of torque, bending moment, and shear. Any cross-sectional geometry defined by linear segments can be accommodated, and generality of loading or boundary conditions is ensured by the inherent characteristics of finite element modelling. The analytical model features the incorporation of non-linear material behaviour, aggregate interlock, dowel action, and the warping restraint and cross-sectional distortional stiffness of diaphragms.

To assist in the evaluation of the analytical model performance, seven double-celled prestressed concrete box girders were cast and tested, five beams having a rectangular cross-section and the remaining two a trapezoidal cross-section. All seven beams were subjected to various torque, bending moment, and shear load combinations.

Performance of the computer model was assessed through comparison with experimental and current theoretical results. The satisfactory outcome of the assessment verified the value of the analytical model as a flexible, sophisticated method of analysis.

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LIST OF SYMBOLS

In all equations presented in the text, the introduced symbols are defined immediately following the respective equations. The most commonly occurring symbols are listed below for reference purposes. Computer program symbolic names are provided in Appendix B.

A = aggregate interlock shear modulus

A_1, A_2 = coefficients

A', A_o = area enclosed by corner longitudinal reinforcement

A_c = cross-sectional area

B = matrix relating finite element strains to nodal displacement

b = beam width defined by corner stringers

b_n = net beam width

c = crack width

D = concrete constitutive matrix

D_b = bar diameter

D_f = dowel force at failure

d = beam depth defined by top and bottom flange stringers

E = initial elastic modulus for concrete

E_s = secant modulus for concrete

E_{st} = steel elastic modulus

F_{yl} = yield force of longitudinal stringers in bottom flange.

f_c' = ultimate concrete compressive strength

f_s = steel stress

G = concrete shear modulus

h = ultimate bending moment lever arm

K' = stiffness term

K_s = St. Venant torsion constant for closed cell

K_{s1} = St. Venant torsion constant for equivalent open cell

K_{s2} = St. Venant torsion constant proportional to additional torque arising from closing the equivalent open cell

k = finite element stiffness matrix

L_{av} = average crack width

M = bending moment

M_o = ultimate bending moment capacity

q = uniform shear flow

R_i = force at node i

r_i = displacement at node i

r = ratio of yield strength of top flange longitudinal reinforcement to bottom flange longitudinal reinforcement

S_y = yield force of hoop reinforcement

s = perimeter coordinate

T = St. Venant torque

T_o = ultimate torque capacity

T_{s1} = St. Venant torque resultant for an open cell

t = wall thickness

u, v = displacements in local x and y axis directions respectively

V = shear force

V_d = dowel force

V_o = ultimate shear force capacity

V_{po} = plastic shear force

v = uniform wall shear stress

W_{av} = average crack width

ΣZ_y = twice the sum of yield forces of longitudinal reinforcement in weaker flange

σ = direct stress

σ_p = ultimate concrete compressive strength

ε = direct strain

ε_c = concrete centroidal strain perpendicular to crack direction

ε_p = concrete strain corresponding to σ_p

λ' = constant

α = ratio of orthogonal stress to stress in direction considered

θ_{zi} = rotation at node i about Z axis

ϕ_i = shape function for node i

η, ξ = orthogonal dimensionless coordinates

ν = poisson ratio

Δ^-_{crack} = shear displacement across crack

α_s = angle of inclination of concrete compression struts

$\cot\alpha_t$ = variable associated with ratio of longitudinal to transverse reinforcement areas

CHAPTER I

INTRODUCTION

1.1 Need for Research

Cellular construction in reinforced concrete is commonplace in current civil engineering practice as this method of construction is both functional and economically competitive. In addition, the flexibility permitted by the cast-in-situ technique enables box girders to assume any desired alignment, as illustrated by the varied configurations of the large number of skew and curved highway bridges. Where large structures are exposed to public scrutiny, the aesthetic appearance of box girder design is an especially valuable asset.

As the application of concrete box girder design becomes more diverse, greater demands are made of the civil engineer in the design of structures of increasing complexity. Thus, the incentive for research in the obscure fields of structural and material behaviour has gained momentum. An example of two such fields of research that have attracted international interest in the past decade is the torsion and shear strength of reinforced and prestressed concrete members.

The complexity of box girder behaviour that has frustrated designers of the past has gradually been diffused since the advent of computer technology. The evolution of computer-orientated analytical design methods has developed rapidly to the current level of sophistication where the capabilities of structural analysis are bounded by few restrictions. The elastic analysis of concrete box girder structures is now well refined, and research effort is presently being focused on the non-linear, post-cracking region of behaviour.

To this point in time, non-linear analytical methods of analysis have been developed to trace the complete load deformation path from the elastic uncracked state through to failure. However, the constituent behaviour of concrete in its cracked form has only come under close scrunity in recent years, and a reasonably comprehensive understanding of its behaviour is only beginning to emerge. Thus, the logical progression is the development of an improved analytical computer model that incorporates and reflects the more accurate representation of material behaviour.

1.2 Thesis Objective and Scope

The principal objective of this thesis is the development of an analytical computer model that can analyze a prestressed concrete box girder of arbitrary cross-section for any loading combination of bending moment, torque, and shear. In addition to estimating cracking and ultimate loads, the complete stress-deformation description of the box girder is provided at any load level in the pre-cracked or post-cracked condition.

To assist in the assessment of the analytical model performance, seven prestressed double-cell concrete box girders were cast and tested under varying load combinations of torque, bending moment, and shear. Subsequently, each beam was analyzed by the computer model for the same respective loading conditions. The accuracy of the analytical modelling was evaluated upon comparison of the computer model results with the corresponding experimental test results and available theoretical predictions.

CHAPTER 2

RESEARCH EVOLUTION

2.1 Introduction

This section is not an exhaustive state of the art presentation, but simply an overview of research developments demonstrating the evolutionary progression that has occurred in experimentation and analysis in fields of study pertaining to this thesis topic. The five associated fields of research encompassed are the three loading types of bending moment, torque, and shear force, together with the nature of prestressed concrete and the characteristics of box girder behaviour.

2.2 Review of Developments to Current State of Art

2.2.1 Experimental Approach

In the infancy of examination of reinforced concrete behaviour, the complexity involved in formulating rational analytical solutions prompted an empirical approach. Detailed experimental programs were undertaken to develop an understanding of behavioural characteristics, and the recorded test data was used to develop and test methods of analysis and design.

The elastic performance and ultimate strength of reinforced concrete in bending have been investigated upon numerous occasions in the testing of scale models of geometrically complex structures. In certain instances, actual structures have been loaded to determine service load response. An example of a bending test to destruction of a complex scale model is the experimental study of Bouwkamp, Scordelis, and Wasti[1], in which a replica of a typical two-lane reinforced concrete bridge was cast and tested to failure.

An excellent example of a field of research where extensive experimental testing has been undertaken because of the difficulty in developing a rational analytical model, is the study of the shear strength of reinforced concrete members. Widespread interest in this topic has been sustained as the "shear failure" mechanism causes a reduction in the strength of structural elements below flexural capacity and a diminution of element ductility. To this point in time, the phenomenon is still not clearly understood, and consequently code provisions have been structured to prevent such a failure. In the past ten years, investigators have turned their attention away from the shear failure mechanism as an entity in itself, and have directed their research effort toward closer examination of the contributing shear components present across a concrete crack; aggregate interlock and dowel action. The aggregate interlock effect[2,3,4] has been researched extensively as has dowel action[4,5,6,7], and a better understanding of the physical mechanics of shear transfer has now emerged. However, the scope of experimentation in examination of the latter two phenomena has been restricted in several respects, and the numerous publications of experimental results and interpretation are not in complete agreement. A relatively recent state of the art paper on the shear strength of reinforced concrete members is that of the joint ASCE-ACI Task Committee 426[8], wherein all aspects of shear transfer are reviewed. Formulae in the latter paper, as well as those in the shear provisions of most building codes for reinforced concrete, have a purely empirical basis, thus demonstrating the valuable contribution of experimental research in complex structural fields.

Closely associated with the study of shear is that of torsion of reinforced concrete members. Only in the last ten years has this avenue of research attracted considerable attention. Indeed, the dramatic explosion in technological advance in this area is evident upon comparison of the torsion provisions of the 1963 ACI Code to those of the 1971 ACI Code and CSA Standard A23.3. In a similar procedure to that adopted for shear, the development of torsion code provisions of the 1971 ACI Code was partially dependent upon experimental results[9,10,11]. The corresponding torsion clauses in the more recently published CSA Standard A23.3 are quite dissimilar, a different theoretical model having been adopted as a basis for torsion derivations. The respective theories for the two codes will be addressed in the following section. From an experimental aspect, test results that were used to substantiate the two theories were in reasonable correspondence but were interpreted differently.

Of paramount importance to the designer is the nature of interaction of all three loading types bending, torsion, and shear. As in the case of shear and torsion, comprehensive experimental programs[11] were undertaken to establish interaction curves for torsion and bending, and torsion and shear. The only interaction formulae for torsion and bending that were developed on a purely theoretical basis are those of Lampert[12], and the ensuing discussion[13] of the proponents of the empirical and theoretical approach highlights the current state of the art of this topic.

Experimental programs[14,15,16] devoted to prestressed concrete research have been conducted in the same manner as that adopted for reinforced concrete. Essentially, the performance of the two reinforced concrete types is quite similar in that beams of concentrically prestressed and symmetrically reinforced concrete are analogous in behaviour, as are eccentrically prestressed and unsymmetrically reinforced concrete beams. Code provisions for torsion and shear of prestressed concrete members are empirically derived, and bear a close resemblance to the corresponding reinforced concrete clauses.

Behaviour of box girder members has come under closer scrutiny in recent years as a large percentage of concrete bridges are of the multi-cell box girder type. The box section has been favoured by many designers because of its aesthetic appearance, efficiency of cross-section, and high torsional rigidity. Since the experimental research of solid and single-celled reinforced concrete members is applicable in most facets of behaviour to box girder structures, research of multi-celled, hollow members has primarily been directed toward those behavioural aspects[17] peculiar to concrete box girder bridges. Such aspects include cross-section distorsion, warping, shear lag[18], and diaphragm action. Invariably, these aspects have not been isolated and studied individually, but have simply been incorporated collectively in the testing of scale models of concrete bridges[1,19].

2.2.2 Development of Theoretical Analyses

Up to the onset of cracking, stresses and deformations in a determinate prestressed concrete box girder of simple cross-section are readily evaluated for any loading combination. The assumptions that the uncracked concrete behaves as a homogeneous, isotropic, elastic material, and the contribution of the reinforcement to the girder stiffness is small, do not constitute a severe approximation of the actual structural behaviour. However, if the cross-section is more complex (multi-celled), longitudinal warping restraint is present (boundary conditions or diaphragm action), or the structure is indeterminate, the simplified approach using classical elastic theory is not accurate, or may indeed be non-applicable. For such a situation, a more sophisticated solution procedure is required, often in the form of a computer-orientated method of analysis.

Upon the onset of cracking, the reinforcement assumes a vital role in carrying tensile stresses and achieving stress redistribution. Of the three loading types, bending, torsion, and shear, only bending action can presently be analyzed theoretically to yield complete deformation behaviour. At the current state of knowledge, shear still defies rational theoretical analysis, whereas several theoretical postulates have been presented to predict torsional behaviour.

The first recognized theoretical model to predict the torsional strength of reinforced concrete was that of Rausch[20], whose model consisted of a network of bars to represent the action of reinforced concrete: compression concrete bars and tension bars for reinforcement. All reinforcement was assumed to yield simultaneously at failure. In time, this model was modified by Cowan[21] and Anderson[22] in assuming that the concrete carried a torque at failure equal to the cracking strength of an unreinforced beam. Subsequently, this approach was questioned by Hsu[23], who maintained that real behaviour lay between the two extremes of that proposed by Cowan and Anderson and 1958 German Code approach that assumed zero cracked concrete torsional strength. The basis of Hsu's research was the development of a modified skew bending model.

The first skew bending model was proposed by Lessig[24], and is illustrated in Fig. 2.1. The depicted failure surface is comprised of an inclined compression zone and a warped failure plane delineated by a diagonally inclined crack connecting compression zone ends, the inclination of the compression zone being dependent on cross-sectional geometry and ratio of longitudinal to transverse steel areas. In proposing his modified skew bending model, Hsu drew attention to the fact that the Lessig theory, being an upper bound solution, considerably overestimated ultimate strength. Hsu's principal modifications we adjusting the shorter sides of the failure surface to intersect the flange-web interface at 90°, and neglecting the torsional contribution of the shorter stirrup legs. Criticism[50] of Hsu's skew bending theory stemmed from three reservations:

1. Several constants were difficult to evaluate.
2. Theory had to be adjusted for square beams.
3. Dowel action of longitudinal bars was indeterminable.

In recognizing that the skew bending theory was an upper bound solution that more accurately predicted ultimate bending strength rather than ultimate torsional strength, several researchers directed their efforts toward refining the space truss theory postulated by Rausch. To this point in time, the most widely accepted space truss theory is that developed by Lampert[12], the theory presented in a form directly applicable to design by Collins and Lampert[25]. The space truss model adopted by Lampert is shown in Fig. 2.2, consisting of intermediate shear walls and longitudinal reinforcement considered to be concentrated into stringers at the hoop reinforcement corners. In the shear walls, the stirrups act as posts and the concrete between the inclined cracks provides the compression diagonals. The angle of the diagonals with respect to the beam axis is assumed to be constant for each side. In the walls that govern failure, the angle is such that both the longitudinal and stirrup reinforcement reach their respective yield points simultaneously. For this reason, the model is a space truss of variable diagonal inclination. The most attractive feature of Lampert's space truss theory is the simplicity of the design equations[25] that now form the basis of the torsion sections in the Canadian Code, CSA Standard A23.3. However, the theory is not completely general as

1. cross-section must be underreinforced,
2. model cannot accommodate shear,
3. only St. Venant torsion can be modelled (no support or load warping restraint)

Criticism of the Collins and Lampert theory is summarized comprehensively in the discussion[13] of the authors' paper[26] wherein they stated that the problem of torsion and bending was basically solved.

Thus far, only the strength predictions of theoretical analyses have been reviewed. The aspects of post-cracking stiffness and inelastic member deformation have not been addressed. The importance of these two latter aspects is paramount since the calculation of equilibrium torsional moments in a statically indeterminate structure requires not only statics but also compatibility conditions. As a point[25] of clarification, an equilibrium torque is one required to maintain equilibrium in a structure, in contrast to a compatibility torque that maintains structural compatibility. Both Hsu[27] and Collins and Lampert[25] have proposed post-cracking stiffness formulae that are derived on the basis of their respective theories. Two important qualifications of Hsu's theoretical approach are that the analysis hinges on the empirical derivation of an equivalent wall thickness for solid and "thick walled" cross-sections, and applicability of the theory to non-rectangular sections is not verified.

The theories of both Hsu and Lampert are restricted to the pure torsion condition.

2.2.3 Analytical Computer Solutions

In the study of complex modern structures such as box girder bridges, dams, and prestressed concrete nuclear containment vessels, the experimental approach adopted in the past is becoming an increasingly expensive method of investigation. Thus, the empirical approach is gradually being superseded by refined analytical computer-orientated methods. Not only does the analytical method offer a considerable saving in terms of time, but the progressive improvement in computer technology permits the development of computer programs of increasing complexity.

In the context of the analysis of box girder bridges, the numerous analytical methods and models that have been developed fall into two categories; those that model behaviour prior to cracking, and those that model the complete structural behaviour to failure.

Of those methods that have been developed for the uncracked section, approximate methods of analysis based on simplified structural behaviour, such as the equivalent beam grillage or anisotropic slab methods, have been used to model structural systems. Of a more precise nature, elaborate methods based on folded plate theory have been developed by several researchers, the most prominent sustained research program having been conducted by Scordelis et alia[28,29] at the University of California, Berkeley. In the adaptation of the folded plate theory to the analysis of box girder bridges, each component plate of the box girder is considered as an assemblage of individual elements, the bending of each plate normal to its plane being analyzed by plate flexure theory, and the in-plane bending analyzed by plane stress theory. These classical theories necessitate the representation of the applied loading by a Fourier Series, with the result that computational effort is considerable though less than that of a finite element solution. In an effort to reduce the number of equations and thus reduce computing time and programming effort, the 'finite strip method' has been proposed by Cheung[30]. In this method, the behaviour of each plate is approximated by an assemblage of longitudinal finite strips for which selected displacement patterns are assumed to represent the behaviour of the strip in the total structure. An additional advantage of this simplified method is that more complex material properties such as concrete anisotropy can be readily introduced. However, both the folded plate and finite strip methods are very much more restricted in their range of application than the finite element method as the two methods can only be applied to box girders of constant cross-sectional geometry. The greater flexibility is achieved at the expense of computational effort. In finite element studies, it should be recognized that the accuracy achieved is dependent upon assumptions of material properties and fineness of the structural mesh subdivisions. Generally, results closely satisfy compatibility, but not necessarily equilibrium in the continuum unless a sufficiently fine mesh is used. An array of computer programs[31] has been developed at the University of California, Berkeley, to analyze box girder bridges of arbitrary plan and general cross-section.

Beyond cracking, material behaviour and structural interaction are complex. Consequently, analysis is almost exclusively performed by the finite element method. Accurate analytical determination of the stress-deformation condition of a reinforced concrete structure at a certain stage of cracking is complicated by

1. Structural response is governed by the interaction of two component materials - steel and concrete.
2. Stress-strain relationships for concrete and reinforcement are non-linear, with the concrete exhibiting anisotropy.
3. Shear transfer phenomena of aggregate interlock and dowel action together with bond slip must be considered.
4. Failure criteria for concrete under biaxial stress conditions, dowel action, and bond slip have to be established.
5. Because of non-linear material properties, equilibrium is not easily maintained, and considerable iteration must be performed within load increments.
6. Transverse rigidity and warping resistance of actual diaphragms, as well as the intrinsic transverse rigidity of the box section, must be included.
7. Dramatic load reversal can occur upon application of the first load increment after prestress transfer.

The initial development of the application of the finite element method to the analysis of reinforced concrete beam behaviour was undertaken at the University of California. In the earliest publication of the application of the finite element method, Ngo and Scordelis[32] analyzed simple beams in which the concrete and steel reinforcement were represented by two-dimensional triangular finite elements, and special bond linkage elements were used to connect reinforcement to concrete. Linear elastic analyses were performed on the beams with predefined crack patterns to determine principal stresses in the concrete, and stress levels in the reinforcement and bond linkages. Since the initial publication of Ngo and Scordelis, the flexibility and capability of the finite element method in this particular application has advanced dramatically[33] to the point where most of the seven stipulated complications have now been successfully incorporated. Such an analytical computer model is that developed by Trikha and Edwards[34].

2.3 Definition of Thesis Approach

2.3.1 Analytical Model Aspect

In recognition of the complexity encountered in the analysis of prestressed concrete box girders in the uncracked and cracked states, a finite element analytical approach has been adopted in determination of both strength and deformation characteristics. In addition to evaluating strength-deformation values at both the cracking and ultimate loads, the analytical model yields a comprehensive description of the structural behaviour for the complete load range. Generality of loading is assured in that the external loads can reflect any ratio of bending moment to torque to shear, and the nature of the loading pattern can be altered at any point in the loading sequence.

Analytical flexibility of the proposed model in a structural mechanics context is exemplified by its ability to simulate:

1. Interaction between the two material components - steel and concrete.
2. Any reinforcement pattern of conventional steel and prestress strand.
3. Non-linear material behaviour.
4. Presence of diaphragms and the cross-sectional shear rigidity of the box girder section.
5. Shear transfer across concrete cracks.

In essence, the strength of the computer model approach is its comparative freedom from analytical and structural constraint.

2.3.2 Experimental Aspect

To evaluate the analytical model performance, a limited experimental program was undertaken in which the test specimens were subjected to varying combinations of bending moment, torque, and shear. To be more representative of concrete box girders employed in practice, all test specimen cross-sections were multi-celled, and of either rectangular or trapezoidal shape. The higher degree of complexity of test specimen cross-sectional geometry also provided a more rigorous basis of comparison for the computer model.

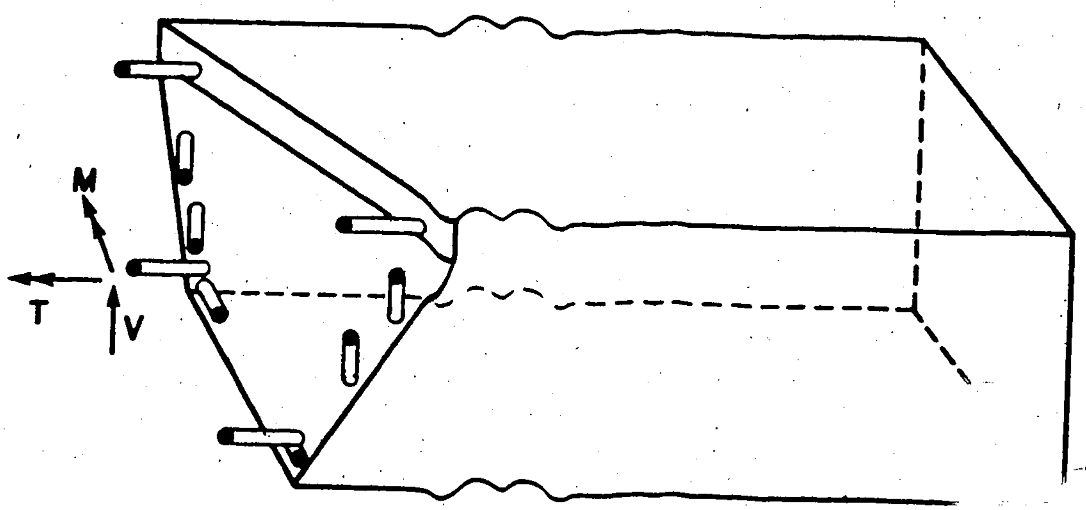


FIG. 2.1 SKEW BENDING MODEL

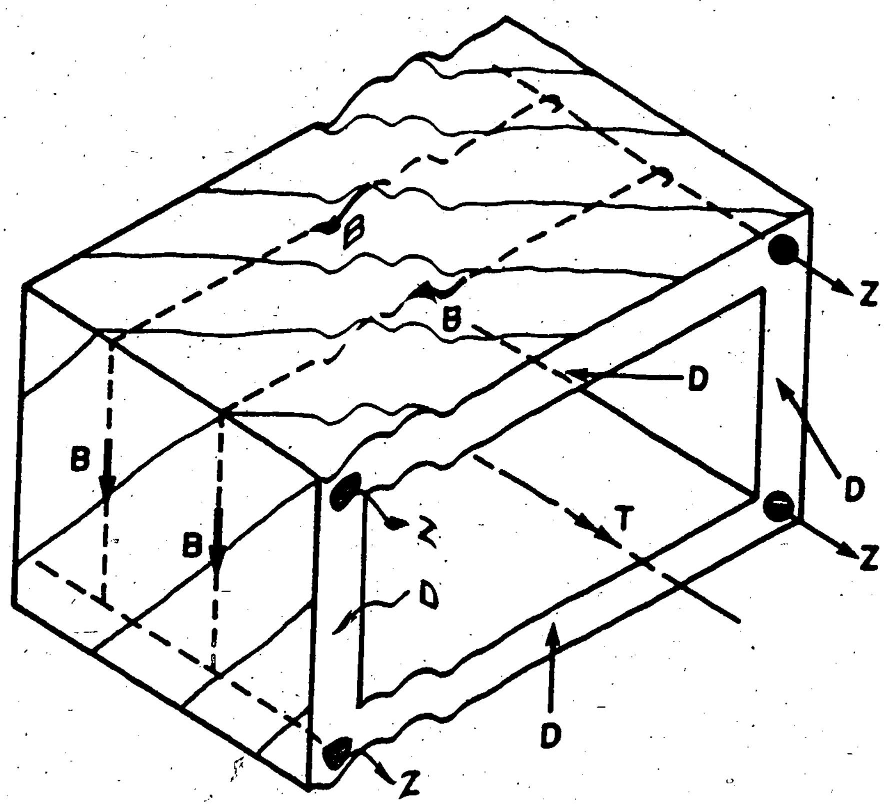


FIG. 2.2 SPACE TRUSS MODEL

CHAPTER 3

ANALYTICAL COMPUTER MODEL

3.1 Introduction

As the complexities of structural behaviour have come under closer scrutiny, more exacting analytical methods have been developed in an effort to overcome past analytical failings or restrictions. Research in structural engineering has often turned to the analytical model as the most appropriate method of investigating complex structural behaviour. Such is the approach in this thesis to the study of the action of prestressed concrete box girders under the combined loading of torque, bending moment, and shear.

Of the analytical methods that are most commonly used in the analysis of concrete box girders, only four are capable of representing the most important characteristics of box girder behaviour, those characteristics being longitudinal bending, St. Venant torsion, transverse distortion, longitudinal warping, and shear lag. Folded plate theory, finite strip theory, finite element theory, and shell theory all yield satisfactory results in an elastic analysis, but only the finite element method is capable of simulating the inelastic behaviour of concrete in its uncracked and cracked states. Primarily for this reason, a finite element approach has been adopted. A simplified finite element mesh similar to those employed in this analysis is exhibited in Fig. 3.1.

In the development of the computer model, the following capabilities have been incorporated:

1. Representation of any general, thin-walled cross-section comprised of linear segments.
2. Representation of any in-plane loading system (prestressing included).
3. Complete description of beam behaviour in the elastic and inelastic regions up to ultimate failure.
4. Accurate portrayal of material response to the action of torsion, bending, and shear in both uncracked and cracked concrete regimes.
5. Ability to model both diaphragm action and the transverse rigidity of a box girder without diaphragms.

The coding of the analytical computer model whose main program and subroutines are listed in Appendices B and C respectively, is written in the Fortran IV language compatible with IBM System/360 and System/370. All computer analyses were achieved through the use of the Amdahl 470 computer at the University of Alberta Computing Centre, and the several system subroutines utilized by the program are public library routines.

3.2 Finite Element Types Employed

3.2.1 Concrete Wall Element

The principal design characteristic of concrete box girders is the structural efficiency achieved by the absence of the central concrete core, thus significantly reducing the member's dead weight with a small loss of strength. As a consequence of the void, the thickness of concrete walls that define the girde s cross-sectional geometry is usually moderately small compared with the member's depth and width. When the wall thickness is comparatively small such that it can be defined as being "thin", the box girder cross-section can be envisaged as an assemblage of flat plates, or in the analytical model as a mesh of appropriate plane stress finite elements.

Since it is most desirable for the sake of economy and efficiency that the concrete box girder walls be modelled by plane stress elements, the classification as to whether a wall thickness is "thick" or "thin" is a prime concern. Thus, it is most appropriate that this issue be examined in detail. In the following derivation, a single cell, uniformly thick box girder will be considered, as exhibited in Fig. 3.2(a).

For a thin-walled single cell, the uniform shear flow is given by

$$q = \frac{T}{2A'} \tag{3.1}$$

in which T = applied torque, and A' = area enclosed by corner longitudinal bars.

Consequently, the uniform wall shear stress is defined by

$$v = \frac{T}{2A't} \tag{3.2}$$

in which t = wall thickness.

However, the St. Venant torsion constant K_s for a closed cell is comprised of two components; i.e.,

$$K_s = K_{s1} + K_{s2} \tag{3.3}$$

in which K_{s1} = torsion constant for the equivalent open cell, and K_{s2} = torsion constant proportional to the additional torque arising from closing an open cell.

Referring to Eq. 3.3, the St. Venant torsion constant for the uniform shear flow of the closed cell is expressed by

$$K_s = \frac{(2A')^2}{\oint \frac{ds}{t}} \tag{3.4}$$

in which s = perimeter length coordinate for the closed cell cross-section, and

$$K_{s1} = \frac{1}{3} \sum bt^3 \tag{3.5}$$

in which b = cross-sectional segment length of thickness t.

Therefore, for the closed single cell shown in Fig. 3.2(a), the St. Venant torsion resultant for the equivalent open cell is given by

$$T_{s1} = \frac{K_{s1}\ T}{K_s} \tag{3.6a}$$

Substituting for K_{s1} and K_s,

$$T_{s1} = \frac{\frac{1}{3}\ \Sigma t^3\ ds\ \oint \frac{ds}{t}\ T}{(2A')^2}$$

$$= \frac{t^2\ (b+d)^2\ T}{3b^2d^2} \tag{3.6b}$$

in which b and d are the cross-sectional dimensions as shown in Fig. 3.2(a).

With reference to Fig. 3.2(b), the torque per unit length developed by the equivalent open cell shear stress distribution is given by

$$\Delta T_{s1} = \frac{\Delta v \ t^2}{6} \tag{3.8}$$

in which Δv = variation in wall shear stress from mid-thickness to surface.

For the complete cell,

$$T_{s1} = \frac{1}{6} \oint \Delta v t^2 \, ds$$

$$= \frac{1}{3} \Delta v t^2 \ (b+d) \tag{3.9}$$

Equating the right hand sides of Eq. 3.7 and Eq. 3.9 yields

$$\Delta v = \frac{(b+d) \ T}{b^2 d^2} \tag{3.10}$$

From Eq. 3.2,

$$v = \frac{T}{2bd \ t} \tag{3.11}$$

Therefore, the ratio of the variation in the wall shear stress distribution from the mid-thickness to the surface, to the uniform shear stress is given by

$$\frac{\Delta v}{v} = \frac{2(b+d) \ t}{bd}$$

$$= \frac{A_c}{A'} \tag{3.12}$$

where A_c = cross-sectional area of cell.

Thus, the definition of a "thin-walled" box girder hinges on the degree of variation in the shear stress distribution that is acceptable. Many authorities postulate a threshold value of 10% that distinguish a "thick" from a "thin" box girder wall.

In employing plane stress rather than three-dimensional finite elements, the total number of degrees of freedom, and thus equations of equilibrium, is reduced considerably. As the computer model will be used to analyze only uniform beams of rectangular and trapezoidal cross-sections in this particular study, a rectangular finite element was chosen as the basic concrete wall element. The higher order rectangular finite element has three degrees of freedom per node, namely two in-plane translational degrees of freedom and a rotational degree of freedom orthogonal to the element plane.

To reduce computing costs, the behaviour of each rectangular concrete finite element is characterized by the stress-strain condition at the centroid of the element. Although this simplification approximates real behaviour, the degree of the approximation will be demonstrated to be of minor significance.

A detailed description of the rectangular finite element chosen for this analytical model is given in Section 3.3.

3.2.2 Reinforcement Element

The three distinct types of reinforcement represented in the computer model are:

1. Prestress reinforcement.
2. Conventional bar reinforcement.
3. Bond spring linkage.

All three are modelled by a one-dimensional constant strain finite element whose equations of equilibrium are given below:

$$\begin{Bmatrix} R_i \\ R_j \end{Bmatrix} = \begin{bmatrix} K' & -K' \\ -K' & K' \end{bmatrix} \begin{Bmatrix} r_i \\ r_j \end{Bmatrix} \tag{3.13}$$

where R_i, R_j = forces at nodes i and j respectively, K' = stiffness term, and r_i, r_j = displacements at nodes i and j respectively.

The nature and derivation of the reinforcement element stiffness elements K' will be treated in detail in Section 3.4.2.1.

3.2.3 Diaphragm Elements

Since the inception of box girder design, the need was recognized for the provision of transverse plates to strengthen the girders' cross-sectional rigidity, distribute shears from webs to bearings over supports, and to prevent excessive cross-sectional distortion. In practice, all box girders have diaphragms over supports, while the number of intermediary diaphragms varies considerably depending upon the span and cross-sectional shape.

Since the prime function of the diaphragm finite element in the analytical model is to provide cross-sectional shear rigidity, a lower-order element is quite satisfactory. To accommodate both the rectangular and trapezoidal cross-sectional shapes, the diaphragm element chosen is a bi-linear isoparametric serendipity element, as shown in Fig. 3.3. This element possesses two translational degrees of freedom per node, eight degrees of freedom in all per element, with the displacement vector defined by:

$$\begin{Bmatrix} u \\ v \end{Bmatrix} = \begin{bmatrix} \langle \phi \rangle & 0 \\ 0 & \langle \phi \rangle \end{bmatrix} \begin{Bmatrix} \underline{u} \\ \underline{v} \end{Bmatrix} \tag{3.14}$$

where shape function $\phi_i = \frac{1}{4}(1+\eta_i\eta)(1+\xi_i\xi)$ (3.15)

where i is the node number and (ξ_i, η_i) are the non-dimensional nodal coordinates.

The geometric coordinates (x, y) are mapped in the x-y plane using the same shape functions as above.

$$\langle x \; y \rangle = \langle \phi \rangle_g \; [\underline{x} \; \underline{y}] \qquad (3.16)$$

where

$$\phi_{ig} = \frac{1}{4}(1+\eta_i\eta)(1+\xi_i\xi) \qquad (3.17)$$

Thus, the analytical model is capable of representing the stiffness for a diaphragm of any quadrilateral geometry. Derivation of the stiffness of a bi-linear isoparametric serendipity element is detailed by Zienkiewicz[35]. In the course of the computer program logic, the stress-strain condition of the diaphragm elements is not monitored as all diaphragm elements are assumed to remain elastic. A listing of the coding of the appropriate subroutines is given in Appendix D.

In accurately representing diaphragm action, three distinct behavioural aspects are addressed. Firstly, the in-plane action of actual diaphragms is incorporated through the use of the bilinear finite element with a full concrete constitutive matrix. Secondly, the transverse rigidity of a box girder length without diaphragms is represented by equivalent diaphragm elements similar to the above, the distinguishing feature being that only a shear stiffness term is present in the constitutive matrix. This aspect is treated in detail in Section 3.4.2.9. The third and last aspect of diaphragm action introduced into the analytical model is the out-of-plane warping restraint developed by actual diaphragms. Within the computer model, those serendipity elements representing actual diaphragm segments contribute predetermined longitudinal stiffness terms to the total beam stiffness matrix at each of the element's four corner nodes. However, derivation of the warping restraint at the corner nodes is dependent upon the classification of the diaphragm thickness as being "thick" or "thin". The two respective warping derivations are fully treated in Sections 3.4.2.7 and 3.4.2.8.

3.3 Choice of Plane Stress Concrete Wall Element

Three plane-stress rectangular finite elements were conspicuous in a literature search for an appropriate concrete finite element to be utilized in the analytical model; the element's prerequisities were established in 3.2.1. The three elements were:

1. McCleod element[36]
2. Scordelis element[37]
3. Sisodiya and Ghali element[38]

Of the three plane-stress elements represented in the Sisodiya and Ghali paper[38], parallelogram element PQC3 was chosen for this comparative investigation.

The method of comparison adopted in choosing the most suitable element simply involved the modelling of identical single-cell concrete girders using each of the above elements, the box girders being subjected to loading patterns of torque, bending moment, and shear. The analysis was strictly elastic, with material behaviour simplified by omitting reinforcement and approximating concrete stiffness as being linear and isotropic. To assess convergence and accuracy, three finite element meshes of varying degrees of refinement, as shown in Fig. 3.4, were used.

As a basis of comparison, both stress-strain and deformation predictions were considered. The stress predictions for the application of a simple bending moment and a pure St. Venant torque, both uniform over the central length of the beam, are plotted in Figs. 3.5(a) and 3.5(b) respectively. All three elements perform equally well in both estimating direct stress and shear stress levels, and converging to the respective theoretical values. The vertical deflection of the central beam cross-section arising from the application of bending moment, and the differential rotation of the central beam length subjected to a uniform St. Venant torque, are given in Table 3.1 for each of the three element types. As in the first basis of comparison, no particular element was significantly superior to another in its accuracy.

In a closer scrutiny of the results, the Sisodiya-Ghali element was observed to have yielded an upper bound estimate to the theoretical bending deformation. Since deformation characteristics are of prime concern in the principal analytical model, this element was considered the least preferable. In choosing between the Scordelis and McCleod elements, inter-element displacement compatibility was the criterion selected as a basis of comparison. Whereas the Scordelis element does not consistently exhibit complete displacement compatibility[37], the choice of the nodal rotational degrees of freedom and the displacement functions for the McCleod element is such that full boundary compatibility is achieved. Thus, the McCleod element was chosen as the marginally preferable plane stress finite element for this application.

To satisfy the requirement of complete displacement compatibility, two McCleod finite element types are employed. For each element type, the rotational degrees of freedom are defined alternately as either

$\theta_{zi} = (\frac{\partial v}{\partial x})_i$ or $\theta_{zi} = (\frac{-\partial u}{\partial y})_i$, the rotation at node 1 of element type 1 being defined as $\theta_{z1} = (\frac{\partial v}{\partial x})_1$, and the rotation at node 1 of element type 2 defined as $\theta_{z1} = (\frac{-\partial u}{\partial y})_1$. By allocating element types such that adjacent elements have different type designations, each element node will have a unique rotation. Figures 3.6(a) and 3.6(b) illustrate element types 1 and 2 respectively, and an element assemblage for a two dimensional beam is displayed in Fig. 3.6(c).

The McCleod displacement functions chosen to satisfy boundary compatibility are

$$u = A_4 + A_7x + A_6y + A_{12}xy + A_1y^2 + A_{10}xy^2 \tag{3.18}$$

$$v = A_5 + A_8x + A_2y + A_3xy + A_9x^2 + A_{11}x^2y \tag{3.19}$$

in which A_i = coefficients numbered in such an order to facilitate inversion of [A] matrices.

Maintenance of boundary compatibility is demonstrated as follows. For y = constant = k, the displacements will have the form

$$u = (A_4 + kA_6 + k^2A_1) + (A_7 + kA_{12} + k^2A_{10})x \tag{3.20}$$

$$v = (A_5 + kA_2) + (A_8 + kA_3)x + (A_9 + kA_{11})x^2 \tag{3.21}$$

In this example, u is a linear function of x and can be defined by 2 nodal translations in the x direction, and v is a quadratic function of x and can be defined by two nodal translations in the y direction together with the single edge rotation $\theta = \frac{\partial v}{\partial x}$. Similarly, nodal displacements are uniquely defined along x = constant boundaries. Thus, displacement compatibility is maintained across element boundaries and throughout the structure.

The stiffness formulation of the McCleod finite element chosen differs from the original derivation through shifting the origin of the element's local axes to the centroid, and changing the node numbering sequence. In adopting these two changes, the matrices $[A]^{-1}$ for element types 1 and 2 (Table 1)[36] have had to be reformulated. The redefined matrices are given in Table 3.2. It should be noted that the element dimensions have been redefined as (2A x 2B) as illustrated in Figs. 3.6(a) and 3.6(b).

The plane stress constitutive relationship is defined as

$$\{\sigma\} = [D]\{\varepsilon\} \tag{3.22}$$

Thus, the element stiffness matrix [k] is given by the relationship

$$[k] = [A^{-1}]^T[\bar{k}]\ [A^{-1}] \tag{3.23}$$

in which

$$[\bar{k}] = \int_{vol} [B]^T\ [D]\ [B]\ dxdydz \tag{3.24}$$

[B] is the matrix relating element strains to element nodal displacements.

3.4 Computer Program Description

3.4.1 General Characteristics

This finite element computer program has been developed as an analytical model to predict the behaviour of prestressed or reinforced concrete box girders subjected to torsion, bending, and shear. Complete stress-strain and deformation information is provided at any specified load level in both the uncracked and cracked states up to the point of girder failure.

Finite elements employed in the program consist of rectangular plane stress concrete elements, one-dimensional reinforcement elements, and plane stress quadrilateral diaphragm elements. Provision is included within the program to represent the presence of a reinforcing steel mesh within any of the concrete elements. Those reinforcement elements whose bond with the adjoining concrete elements is suspect to deterioration during loading, are connected to adjacent concrete nodes through bond spring linkages whose stiffnesses are modified as loading progresses. At every beam cross-section delineated by concrete element nodes, equivalent diaphragm elements are introduced to simulate the box beam's transverse cross-sectional rigidity.

Any loading pattern can be superimposed upon the analytical model through the judicious choice of nodal force combinations, the only restriction being that force directions cannot be orthogonal to plane stress concrete elements. The presence of prestress forces is readily accommodated.

To accurately reproduce concrete behaviour, the program is capable of modelling non-linear material characteristics. Naturally, non-linearity is not restricted solely to concrete, as the prestress and conventional reinforcement and bond spring linkages all exhibit non-linear behaviour. The loading sequence is an incremental one consisting of the superposition of successive load increments. Following the application of each load increment, all material elements are checked for deviation from their pre-defined behavioural paths, the probability of deviation occurring being reduced through the use of the Runge-Kutta method. If significant deviation is detected, equilibrium is restored using the modified Newton-Rapson iterative method. Probability of material deviation within a subsequent load increment is reduced by modifying all element stiffnesses at the end of the current load increment using a tangent stiffness formulation. A detailed description of the latter method of stiffness adjustment and the Runge-Kutta method is given in Section 3.4.4.

To solve the large set of equilibrium equations efficiently, a banded-block solution process has been adopted. At any instant of the solution process, only two stiffness blocks of half-band width are stored in core. Efficient transmission of the stiffness blocks in and out of core storage is achieved through the use of public library system subroutines provided by the University of Alberta Computing Services.

The non-linear behaviour of a rectangular concrete element under bi-axial stresses is characterized by the stress-strain condition at the centroid of the element. Once the principal tensile centroidal stress exceeds the concrete tensile strength, the concrete element is designated as cracked, the crack direction thereafter remaining fixed. The crack inclination is derived from the centroidal stress conditions at cracking. If the monitored crack width should close immediately following prestress transfer, the element is once again designated as an uncracked element. In reconstituting the stiffness of the concrete element following cracking, the aggregate interlock and dowel effects are taken into account in determining the shear rigidity across the crack. At all but the lowest stress levels, concrete behaves as an anisotropic material.

At the conclusion of each load increment, all elements are checked to detect local failure. Structural failure occurs when one of the principal prestress strand elements fails in tension or a concrete element crushes. In reality, the modelled structure might well not have failed under the above circumstances, but its full load carrying capacity will have been attained and its subsequent highly inelastic behaviour will indicate imminent collapse. Should failure not have occurred following the application of the load increment, a comprehensive summary of stress, strain, and deformation values will be printed out.

To enable large program runs to be monitored during their execution, several duplicate print statements were inserted in the output subroutine. The fully comprehensive output for each load increment is assigned to a high speed printing device, whereas simultaneously, a dramatically smaller representative output is viewed on a terminal screen that can subsequently assume the role of the controlling device. Termination of execution of the program can be prompted once irregular behaviour is observed.

3.4.2 Simulation of Material Behaviour

3.4.2.1 Concrete Stiffness Under Bi-axial Stresses:

Derivation of the concrete constitutive matrix under bi-axial stress conditions closely follows the research of Liu, Nilson, and Slate[39]. The form of the stress-strain equations for concrete under uniaxial or biaxial stress conditions, and the resulting constitutive matrix proposed by the above authors are easily incorporated in an incremental iterative finite element analysis as developed in this thesis.

The stress-strain relationship for concrete under biaxial stresses is given by the equation

$$\sigma = \frac{\varepsilon E}{(1-\nu\alpha)\left[1+\left(\frac{1}{(1-\nu\alpha)}\frac{E}{E_s}-2\right)\left(\frac{\varepsilon}{\varepsilon_p}\right)+\left(\frac{\varepsilon}{\varepsilon_p}\right)^2\right]} \qquad (3.25)$$

in which σ = stress in direction considered, ε = strain in direction considered, E = initial modulus, ν = Poisson's ratio, α = ratio of orthogonal stress to stress in direction considered, σ_p = ultimate compressive strength, ε_p = strain at point of ultimate strength, and $E_s = \frac{\sigma_p}{\varepsilon_p}$.

Liu, Nilson, and Slate state that the equation is applicable to concrete in biaxial compression only, but in this analysis, the equation is used in biaxial compression and compression-tension conditions, where in the latter state the orthogonal principal stress is tensile.

For uniaxial loading, the above equation simplifies to:

$$\sigma = \frac{\varepsilon E}{1+\left(\frac{E}{E_s}-2\right)\left(\frac{\varepsilon}{\varepsilon_p}\right)+\left(\frac{\varepsilon}{\varepsilon_p}\right)^2} \qquad (3.26)$$

This theoretical equation was compared to the experimental results of a concrete cylinder tested during the experimental program. The two curves exhibited in Fig. 3.7 correspond closely, demonstrating the reasonably accurate approximation of the above equation to real behaviour.

The two distinct load conditions encountered in the analytical model are the incremental and total load cases. In the formulation of the concrete constitutive matrix, the stiffnesses in the two orthogonal principal directions are derived on a tangent moduli basis for incremental loading, and a secant moduli approach during the iterative process when the total loading condition prevails. Thus, the concrete constitutive matrix is expressed in the form:

$$\begin{Bmatrix} \sigma_1 \\ \sigma_2 \\ \sigma_{12} \end{Bmatrix} = \begin{bmatrix} \lambda' \frac{E'_{1b}}{E'_{2b}} & \lambda'\nu_1 & 0 \\ \lambda'\nu_1 & \lambda' & 0 \\ 0 & 0 & \frac{E'_{1b}\,E'_{2b}}{E'_{1b} + E'_{2b} + 2E'_{2b}\,\nu_1} \end{bmatrix} \begin{Bmatrix} \varepsilon_1 \\ \varepsilon_2 \\ \varepsilon_{12} \end{Bmatrix} \qquad (3.27a)$$

in which

$$\lambda' = \frac{E'_{1b}}{\left(\frac{E'_{1b}}{E'_{2b}} - \nu_1^2\right)} \qquad (3.27b)$$

and where in the total loading condition,

$$E'_{1b} = \frac{E}{1 + \left[\frac{1}{(1-\nu\alpha_1)}\frac{E}{E_s} - 2\right]\left(\frac{\varepsilon_1}{\varepsilon_p}\right) + \left(\frac{\varepsilon_1}{\varepsilon_p}\right)^2} \qquad (3.27c)$$

for biaxial compression and compression tension. However, in a uniaxial compression state

$$E'_{1b} = \frac{E}{1 + \left(\frac{E}{E_s} - 2\right)\left(\frac{\varepsilon_1}{\varepsilon_p}\right) + \left(\frac{\varepsilon_1}{\varepsilon_p}\right)^2} \qquad (3.27d)$$

while for biaxial tension, tension compression, and uniaxial tension conditions,

$$E'_{1b} = E \qquad (3.27e)$$

During the application of load increments,

$$E'_{1b} = \frac{E[1 - (\frac{\varepsilon_1}{\varepsilon_p})^2]}{\{1 + [\frac{E}{(1 - \nu\alpha_1)\, E_s} - 2]\, (\frac{\varepsilon_1}{\varepsilon_p}) + (\frac{\varepsilon_1}{\varepsilon_p})^2\}^2} \tag{3.27f}$$

for biaxial compression and compression tension. For the uniaxial compression state

$$E'_{1b} = \frac{E[1 - (\frac{\varepsilon_1}{\varepsilon_p})^2]}{[1 + (\frac{E}{E_s} - 2)(\frac{\varepsilon_1}{\varepsilon_p}) + (\frac{\varepsilon_1}{\varepsilon_p})^2]^2} \tag{3.27g}$$

while for biaxial tension, tension compression, and uniaxial tension conditions,

$$E'_{1b} = E \tag{3.27h}$$

E'_{2b} is defined in a similar manner to E'_{1b}.

In defining concrete moduli under biaxial stress conditions, the nature of the two associated orthogonal principal stresses is the criterion used in establishing the biaxial state. However, such a definition is misleading if one principal compressive stress is much larger than its orthogonal compressive stress such that the strain in the orthogonal direction is tensile. Since the behaviour of concrete is characterized in such a conflicting situation by its strain state, not stress state, a condition of biaxial compression only prevails when the strain in the two principal strain directions is compressive. In its application to the analytical model, use of a principal strain rather than principal stress criterion changes the concrete moduli little, as the tangent stiffness of concrete in compression at low stress levels is close to the initial tangent modulus.

3.4.2.2 <u>Concrete Crack Width</u>: Janjua and Welch[40] propose that the concrete crack width be given by:

$$W_{av.} = L_{av.} \cdot \frac{(f_s - 3)}{E_s} \cdot R \qquad (3.28)$$

where $L_{av.}$ = average crack spacing, R = ratio of extreme fibre distance from neutral axis to distance of steel centroid to neutral axis, f_s = steel stress, and E_s = steel elastic modulus.

$$L_{av.} = 1.5t + 3D \qquad (3.29a)$$

in which t = concrete cover and D = bar diameter.

All the above variables are in kip and inch units.

Unfortunately, the above formulation does not lend itself readily to inclusion within the analytical model logic as the term R is not easily determined. Moreover, the average crack spacing expression shown above does not accurately predict the test beam observations. Consequently, elaborate formulations were discarded in preference for the simple expression:

$$W_{av.} = \varepsilon_c \cdot L'_{av.} \qquad (3.29b)$$

in which ε_c = finite element centroidal direct strain perpendicular to crack direction and $L'_{av.}$ = observed average crack spacing.

3.4.2.3 <u>Aggregate Interlock</u>: Once plain concrete cracks, shear is still able to be transmitted across the crack through interlock of the two adjacent rough surfaces. The level of shear that can be transferred, however, has been a subject of constant conjecture and research in the development of analytical models. The wide diversity of opinion is reflected in the two opposing schools of thought that support the Space Truss and Skew Bending Analogies.

Whereas a common approach[41] has been to assume that a constant percentage of the concrete shear strength is retained after cracking, the treatment of the aggregate interlock effect in this analytical model is based on the research conducted by Houde and Mirza[4]. The influence of the three parameters of crack width, concrete strength, and maximum aggregate size on the shear rigidity modulus was examined, and the results lead to the development of the relationship:

$$A = 57 \cdot \left(\frac{1}{c}\right)^{3/2} \cdot \sqrt{\frac{f_c'}{5000}} \qquad (3.30)$$

where A = shear rigidity modulus and c = crack width.

All of the above variables are expressed in inch pound units. As observed in the above expression, the effect of aggregate size was found to be negligible.

The expression above was derived from experimental measurements made in the range of crack widths of 2×10^{-3} to 20×10^{-3} inches. Crack widths smaller than 2×10^{-3} were difficult to accurately control. In the uncertain region beyond $\frac{1}{c} = 500$, the authors have suggested that the curve for the rigidity modulus is asymptotic to the line A = G (shear modulus of uncracked concrete) for large values of $\frac{1}{c}$. Such a supposition does seem severe, however, and thus the equation for A has been applied to the range of $\frac{1}{c} > 500$ in the absence of research that indicates otherwise. The curve for the rigidity modulus intersects the line $A = G$ close to $\frac{1}{c} = 1000$.

3.4.2.4 <u>Dowel Effect</u>: In bridging across the concrete crack, reinforcement not only restricts the widening of the crack such that substantial aggregate interlock can develop, but also offers shear resistance normal to its axis. This shear resistive force developed in the reinforcement is termed the "dowel force". Since the dowel effect is only significant across cracks that have experienced considerable shear displacement, reinforcement must be highly stressed in tension to develop dowel action. Principal longitudinal tension reinforcement in an underreinforced beam is such an example.

The research of Houde and Mirza[4] is used in quantitatively defining dowel stiffness. The dowel load-displacement relationship is given by:

$$V_d = 2000.\, D_f.\, \Delta_{crack} \tag{3.31}$$

where V_d = dowel force, Δ_{crack} = shear displacement across crack, and D_f = dowel failure force.

$$D_f = 40\, b_n\, (f'_c)^{1/3} \tag{3.32}$$

in which b_n = net beam width.

All units are in pounds and inches. Embedment length, bar size or arrangement, and axial stress in reinforcement below yield do not have a pronounced effect upon dowel action.

Appraisal of the effect of parameters in addition to those already mentioned is given in other publications. The effect of inclination of reinforcement to crack direction on dowel strength was investigated by D[illegible]ska and although an expression was derived for the dowel failure force, a simple theoretical relationship could not be found for deformations. Bauman's[42] research demonstrated that positioning of stirrups between a diagonal crack and the support did not increase the dowel strength if the distance between crack and stirrup exceeded 2.5 cms. Also, the same author stated that dowel action was not influenced by crack width or concrete cover.

Upon the commencement of splitting along the reinforcement, dowel action deteriorates. The level of residual dowel action in such circumstances is very much dependent on stirrup spacing, but the precise behaviour is difficult to define. In this analytical model, dowel strength after splitting is considered negligible. Dowel failure occurs when the dowel displacement Δ_{crack} exceeds 5×10^{-4} inches.

3.4.2.5 Cracked Concrete Stiffness: The concrete constitutive matrix for a cracked finite element is of the form:

$$\begin{Bmatrix} \sigma_1 \\ \sigma_2 \\ \sigma_{12} \end{Bmatrix} = \begin{bmatrix} 0 & 0 & 0 \\ 0 & E_2 & 0 \\ 0 & 0 & G' \end{bmatrix} \begin{Bmatrix} \varepsilon_1 \\ \varepsilon_2 \\ \varepsilon_{12} \end{Bmatrix} \qquad (3.33)$$

where σ_1 is the principal tensile stress that acts in the direction normal to the crack. Since the concrete stiffness in this direction has been set to zero, the stress in the direction of σ_1 will consequently be zero. In the direction of the principal compressive stress, the stress-strain relationship is given by Equations 3.27d and 3.27g in 3.4.2.1 as a uniaxial loading condition now prevails.

The cracked shear modulus G' is comprised of two contributions;

the aggregate interlock and dowel stiffnesses. The former is readily evaluated at the centroid of a concrete element as described in Sections 3.4.2.2 and 3.4.2.3. In its form presented in the preceding section, the dowel force stiffness developed in the reinforcement is not in the units of shear modulus. Therefore, the following procedure has been adopted. Upon calculation of the crack width and centroidal shear strain parallel to the crack, the dowel displacement Δ_{crack} is evaluated. The subsequently calculated dowel force V_d is then considered uniformly distributed over the element's cracked concrete surface whose area is given by the product of the element thickness and the length of the inclined crack that passes through the centroid and extends from element boundary to boundary. The equivalent dowel shear modulus is added to the aggregate interlock rigidity modulus to define the equivalent cracked concrete shear modulus G'.

In the formulation of G', the presence of stirrups is neglected, and all dowel stiffness contributions are calculated disregarding deviation from perpendicular inclination of reinforcement to the crack direction.

3.4.2.6 Warping Resistance of Thin Concrete Diaphragms: The out-of-plane warping resistance at the corners of a thin concrete plate is derived in most theory of elasticity texts[43]. For a square diaphragm element, the out-of-plane warping stiffness matrix is of the form below:

$$\begin{Bmatrix} R_{1x} \\ R_{2x} \\ R_{3x} \\ R_{4x} \end{Bmatrix} = \begin{bmatrix} X & -X & X & -X \\ -X & X & -X & X \\ X & -X & X & -X \\ -X & X & -X & X \end{bmatrix} \begin{Bmatrix} r_{1x} \\ r_{2x} \\ r_{3x} \\ r_{rx} \end{Bmatrix} \qquad (3.34)$$

where R_{1x} = force at node 1 in a direction perpendicular to the element's plane, X = corner warping stiffness, and r_{1x} = displacement at node 1 corresponding to force R_{1x}.

$$X = \frac{Et^3}{3(1 + \nu)\ k^2} \tag{3.35}$$

in which E = concrete elastic modulus, t = thickness, and k = length of element's diagonal.

The off-diagonal stiffness terms in Eq. 3.34 have the same magnitude as the main diagonal terms since the plate was visualized as being simply supported at each of its four corners in the derivation of the warping restraint forces. Rectangular or quadrilateral diaphragm elements are treated as square elements of the same area in computing warping restraint.

3.4.2.7 Warping Restraint of Thick Concrete Diaphragms: Since the classical theory of plates formulation for diaphragm warping resistance given in the preceding section is valid only for "thin" plates, the distinction between "thick" and "thin" plate thicknesses must be made. Theoretically, no method of distinction is currently available. An additional qualification of the classical approach is that the derivation is developed for a square plate. Thus, an approximation is immediately introduced if the formulation is applied to non-square diaphragm shapes.

To analytically simulate actual diaphragm action, a finite element model was developed that permitted complete generality of diaphragm shape and thickness. To accommodate a complete range of possible diaphragm thicknesses, an assemblage of three dimensional bi-quadratic serendipity finite elements was used to represent the presence of diaphragms. Upon comparing the corner end warping displacements of two identical double cell box beams (illustrated in Fig. 3.8), one beam being restrained longitudinally by end diaphragms, the diaphragm warping stiffness at its four corners was derived as a function of the box beam warping resistance. The warping resistance equation is of the form:

$$R_{wd} = \left(\frac{w_b}{w_d} - 1\right) R_{wb} \tag{3.36}$$

where R_{wd} = corner warping stiffness of diaphragm, w_b = warping displacement of unrestrained box beam corner, w_d = corresponding warping displacement of box beam corner restrained by diaphragm, and R_{wb} = corner warping stiffness of unrestrained box beam.

The analytical model results and the corresponding classical plate derivations for a particular test beam are illustrated in Fig. 3.9. As expected, the classical approach dramatically overestimates the warping resistance for thick diaphragms, but even for smaller thicknesses, there is not good agreement between the two formulations. For thicknesses smaller than 2 inches, the dramatic divergence of the approaches is not significant as both predict negligible diaphragm warping restraint compared with that of the box beam. For thicknesses exceeding 2 inches, it is apparent that the classical approach does not give accurate correspondence with actual diaphragm behaviour as represented with a reasonable degree of accuracy by the finite element method. Consequently, for the beam cross-sectional geometry used in this comparison, the geometry being that of the five rectangular beams tested in the experimental program, the finite element approach is the more preferred method for modelling diaphragm behaviour.

The diaphragm corner out-of-plane warping stiffnesses derived by the above approach are readily incorporated in the principal analytical computer model. Throughout the analysis, it is assumed that the diaphragms behave elastically.

3.4.2.8 Shear Rigidity of Equivalent Diaphragms: In modelling a rectilinear box girder cross-section as an assemblage of plane stress finite elements, no account has been taken of the girder's intrinsic cross-sectional distortion rigidity. At a cross-section in the analytical model where an actual diaphragm is not provided, externally applied loads will not be distributed correctly unless an equivalent diaphragm is introduced. Evaluation of the shear rigidity of an equivalent diaphragm is treated in detail by Sawko and Cope[44]. In the formulation of the element stiffness matrix, the only non-zero term in the constitutive matrix is the shear modulus. Thus, the function of the equivalent diaphragm is twofold in preserving both cross-sectional geometry and real structural performance.

3.4.2.9 Concrete-Reinforcement Bond: Assumption of perfect bond between reinforcement and concrete can result in significant error[45] in analytical modelling. Thus, the concept of bond spring linkages connecting reinforcement and concrete elements was developed, wherein the loss of adhesion between concrete and steel is represented by a softening of the spring stiffness. Nilson[45] was the first to introduce a non-linear bond-slip relationship into a finite element analysis, and subsequent extensive research investigations in this field, as that conducted by McCutcheon, Mirza and Mufti[49], used similar modelling systems to that of Nilson. The adopted equation relating bond stress to bond spring elongation used in stiffness formulation is that given by Houde and Mirza[4]. In both the incremental and total load cases, the bond spring linkage stiffness is the product of the bond stress curve slope[4] and the contributing circumferencial area of the reinforcement bar to which the bond linkage is attached. Failure of the bond spring occurs when the spring elongation exceeds .0012 inches.

3.4.2.10 Reinforcement Stiffness: To simplify the analytical representation of reinforcement bars, bi-linear stress-strain curves have been assumed for both conventional and prestress reinforcement, as shown in Fig. 3.10.

Below yield, the stiffness of both reinforcement types in the incremental and total load cases is given by the respective moduli of elasticity. Beyond yielding, the slope of the strain hardening segment of the stress-strain curve defines the incremental reinforcement stiffness. However, in the total load condition that prevails in the iterative process, the method of stiffness derivation for the conventional and prestress reinforcement differs. Sir the slope of the strain hardening section of the conventional reinforcement stress-strain curve is highly inelastic, an excessive number of modified Newton-Rapson iterations are often required to restore equilibrium. If the set of equations were large, this approach could be highly impractical. Therefore, once a conventional reinforcement bar has yielded in the total load condition, it is represented in the analytical model by a bar of zero stiffness, the load carried by the bar being represented by equivalent external loads applied at the bar nodes. Following such a stiffness change, The strain in the yielding bar will increase as the load is maintained, but the magnitude of the equivalent bar loads will increase only slightly. Thus, at a particular load level, no iteration is required to restore an acceptable degree of equilibrium to the yielding bars. This approach cannot be applied, however, to the yielding of the prestress reinforcement. Prestress reinforcement constitutes the underreinforced beam's last reserve of strength, and setting of its stiffness to zero will immediately produce instability. Thus, the modified Newton-Rapson method is retained for modelling yielding prestress reinforcement in the total load condition. Use of a relaxation factor has been introduced to improve the rate of convergence of prestress strand deviation.

3.4.3 Failure Criteria

Of the two material constituents, concrete and reinforcing steel, the failure characteristics of concrete are complex and warrant clarification. In establishing the tensile and compressive strengths of concrete under biaxial stress conditions, Kupfer, Hilsdorf, and Rüsch[46] have postulated a biaxial stress envelope illustrated in Fig. 3.11. In addition to specifying biaxial stress failure combinations, the authors also established that the Poisson ratio in biaxial tension, biaxial compression, and tension-compression were .18, .2, and an average value of .19 respectively. The simplified envelope used in the analytical model is shown in Fig. 3.12. Beyond cracking, the shear rigidity across a concrete crack is developed by the aggregate interlock and dowel mechanisms. In the absence of relevant information, the failure stress level for the aggregate interlock mechanism has been set equal to the uniaxial concrete compressive strength. Under actual test conditions, the evaluated aggregate interlock stress has not been found to rise beyond 750 p.s.i.[4]. Dowel linkage and the concrete-related bond spring linkage failure criteria are specified in preceding Sections 3.4.2.6 and 3.4.2.9 respectively.

In a total structure context, failure in the analytical model is defined as having occurred when a concrete element has crushed or a major prestress reinforcement element has failed in tension.

3.4.4 Numerical Methods

The three numerical methods that perform vital functions in the analytical model are the block-by-block gaussian elimination solution process, Runge-Kutta, and modified Newton-Rapson methods.

To reduce the size of the allocated core storage area used in the equation solution process, only two stiffness blocks are retained in core at any one time. After a stiffness block has been reduced by gaussian elimination, it is written onto auxiliary disc storage, and the following unreduced stiffness block is subsequently read into core. If an efficient method of transfer of the large data sets in and out of core storage is used, this block-by-block solution process is ideally suited to the solution of large equation systems. Logic details are given in the listing of subroutine SOLVE in Appendix D.

The area of application of the Runge-Kutta method is the progressive adjustment of the structural stiffness as the external loads are applied incrementally. Figure 3.13(a) illustrates the procedure. Using the stiffness of the material element in the previous (n - 1)th. load increment, defined by the slope of line AB, the stress-strain state upon application of half of the current nth. load increment is calculated, denoted by point C. The tangent stiffness at the corresponding point D on the predefined stress-strain curve for the material is subsequently adopted as a reasonable prediction of the average material element stiffness for the nth. load increment. Upon superposition of the full nth. load, the stress-strain state at the end of the nth. increment is established, denoted by point E. Thus, the Runge-Kutta method of stiffness predicti[illegible] program efficiency through minimizing the need for the time c[illegible] iterative process that must be invoked if significant material [illegible]ation occurs.

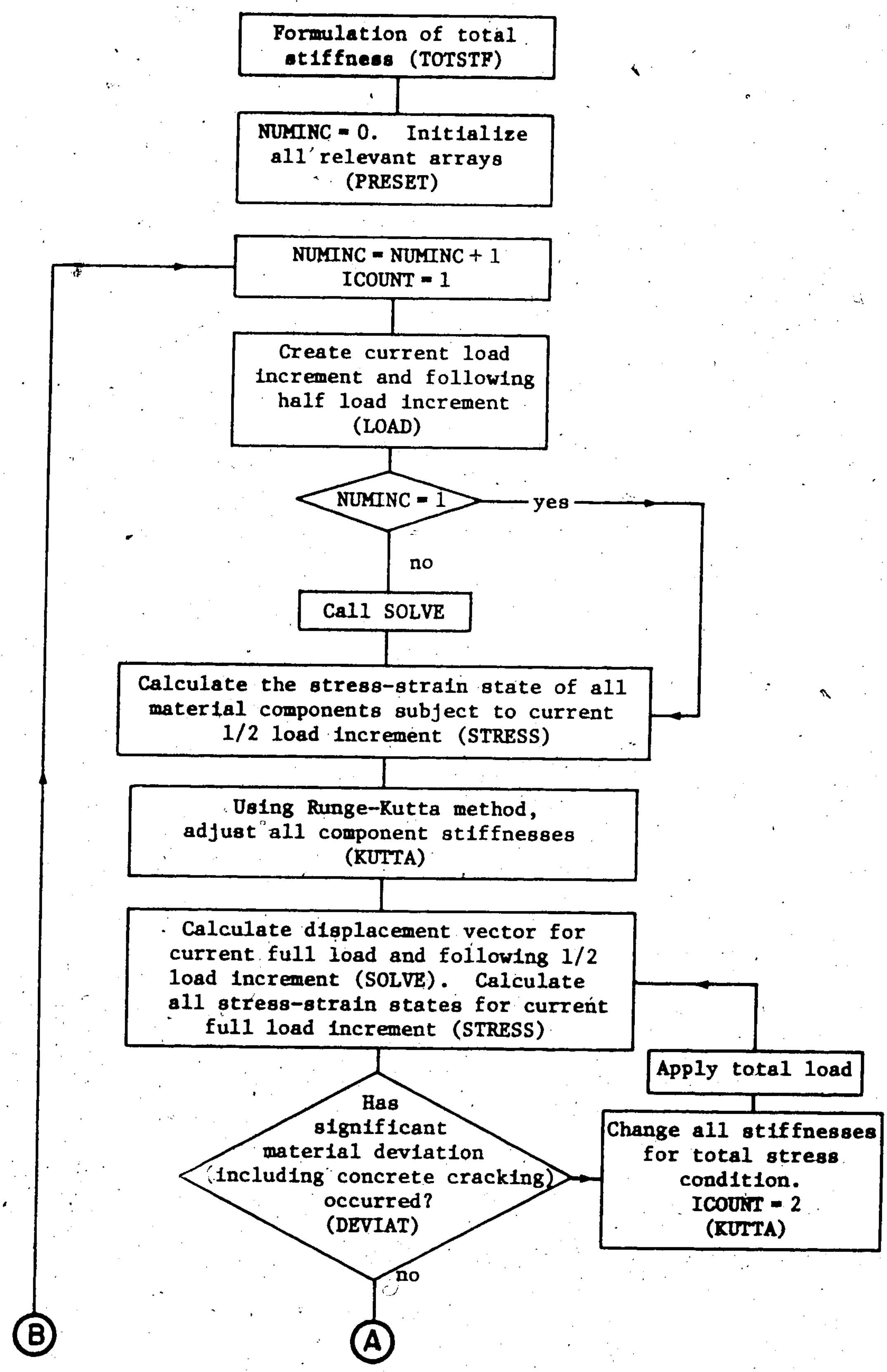
Should deviation at the end of a load increment be significant, equilibrium is restored by employing the iterative modified Newton-Rapson method. After reducing the deviation of point Q in Fig. 3.13(b) to an acceptable level, denoted by point R, the tangent stiffness at point S is the initial estimate of increment stiffness used in the Runge-Kutta stiffness adjustment for the nth. load increment.

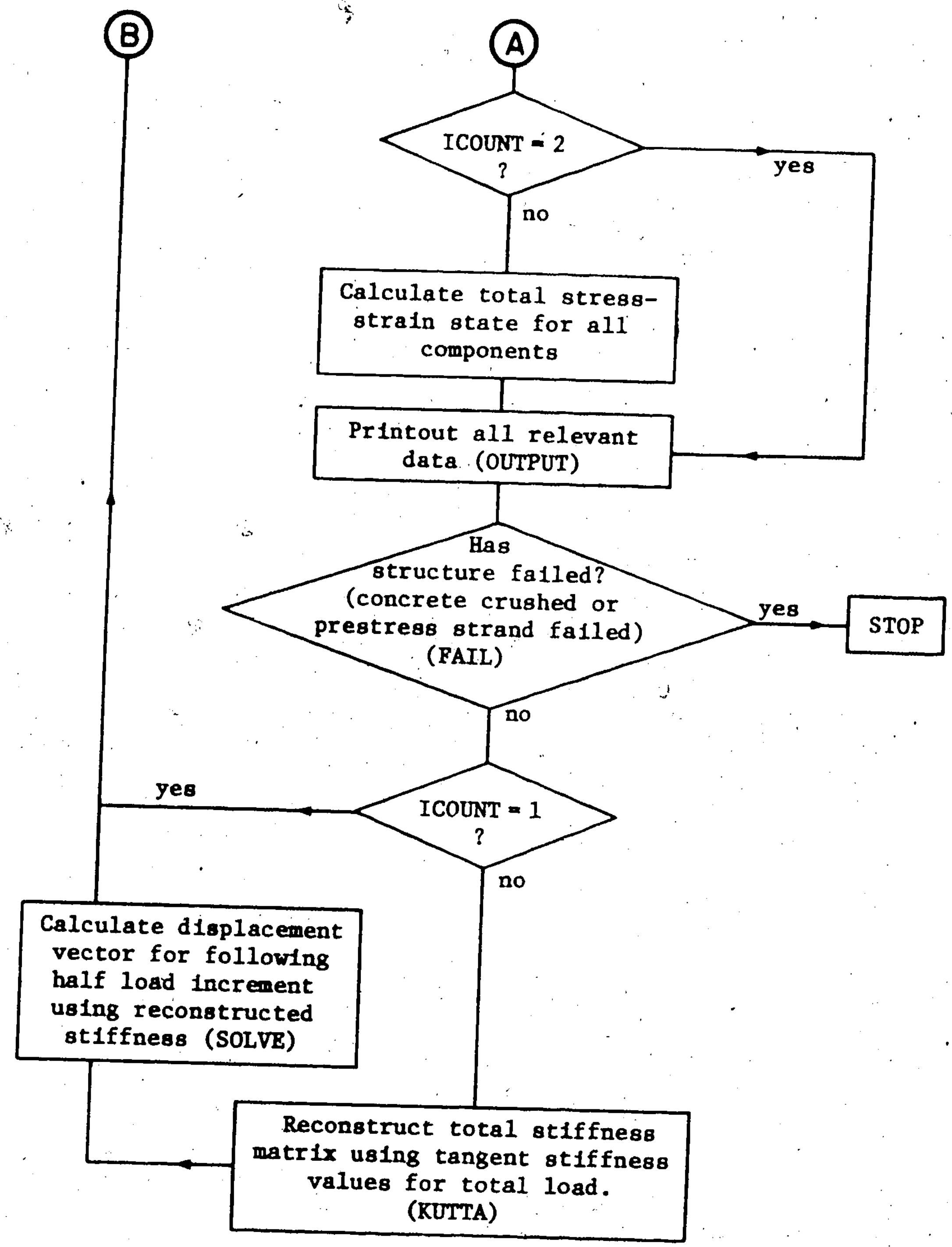
Both the Runge-Kutta and modified Newton-Rapson techniques are treated in detail in most applied mechanics texts[47] that address non-linear behaviour.

3.4.5 Flow Chart for Main Program

In the flow chart that follows, the logic skeleton of the main program is presented, the symbolic names in brackets designating the subroutines that accomplish the defined logic steps. Further logic clarification is supplied by the numerous comment cards distributed through the main program, whose listing is given in Appendix C.

FLOW CHART FOR MAIN PROGRAM




B
A
ICOUNT = 2
?
yes
no
Calculate total stress-strain state for all components
Printout all relevant data (OUTPUT)
Has structure failed? (concrete crushed or prestress strand failed) (FAIL)
yes
STOP
no
yes
ICOUNT = 1
?
no
Calculate displacement vector for following half load increment using reconstructed stiffness (SOLVE)
Reconstruct total stiffness matrix using tangent stiffness values for total load. (KUTTA)

3.4.6 Derivation of Subroutine Logic

Logic development notes of the more complex subroutines comprise Appendix E.

3.5 Program Usage

3.5.1 Capabilities and Restrictions

(A) Beam Geometry

The program has been developed for the analysis of concrete box girder structures with modest wall thicknesses, but any wall thickness can be accommodated with the qualification that accuracy of solution will be prejudiced for "thick" walls, as detailed in 3.2.1. Cross-sectional geometry can assume any form that can be represented by linear segments, but in the program's present form, no geometry change is permitted along the beam's length. Such a shortcoming can be remedied by replacing the rectangular concrete finite element by a general quadrilateral element[38].

(B) Material Behaviour

To reduce computational effort, the behaviour of a concrete element is characterized by the stress-strain condition at the element's centroid. For modestly fine meshes of high order elements, this approach is an acceptable approximation. Uncracked concrete is modelled as a heterogeneous, anisotropic, non-linear material, but cracking introduces two phenomena associated with the definition of shear rigidity across a crack, namely aggregate interlock effect and dowel action. As cracking progresses, the stiffness of these two contributions are modified in addition to the concrete stiffness in the parallel crack direction. The closing of cracks produced by dramatic load reversal in the load increment following prestress transfer, and the superposition of a different loading type (torque upon bending load) is also within the program's capability.

The two principal reinforcement types, conventional steel and prestress strand, are modelled by one-dimensional finite elements. Consequently, a continuous reinforcement bar is represented by a series of segments of constant stress, producing a step-like variation in bar force. In direct contrast, steel mesh reinforcement is considered to be uniformly distributed throughout a concrete element, and its stiffness is formulated in an identical manner to that for a rectangular concrete finite element. The stress-strain curves for all three reinforcement types are approximated by multi-linear functions

In the development of the finite element mesh, the mechanism of force transfer at the steel-concrete interface is simulated by bond spring linkages connecting adjacent steel and concrete nodes. As loading progresses, the stiffness of the bond spring is adjusted to reflect the gradual deterioration in concrete bond.

Should significant material behavioural deviation be detected within a load increment, an iterative process is automatically initiated, and proceeds until equilibrium is restored.

(C) Loading and Boundary Conditions

Any loading or boundary condition combination can be imposed upon the analytical model provided that the appropriate degrees of freedom are present. In specifying load magnitudes, the subsequent number of load increments should not be small such that the computationally expensive iterative process is invoked frequently. Provision for application is not restricted to external test loads, since both prestressing and post-stressing techniques can be represented by equivalent model forces.

(D) Diaphragms

The presence of diaphragms of any thickness and the intrinsic cross-sectional deformation resistance of the hollow beam a e two important components of box girder behaviour, and both characteristics are incorporated in the program logic. Throughout the entire load range, diaphragm action is assumed to be elastic.

(E) Printout Information

Through the use of output control parameters, the precise nature of the printed output can be specified in a selective qualitative and quantitative manner, as detailed in 3.5.4.

(F) Program Monitoring

Provision is made within the program for transmission of output information to a monitoring device such as a CRT display terminal. In the running of large problems, the user has the option of observing program progress and controlling the rate of execution in the event that erroneous results may prompt run termination, thus reducing unnecessary complete run expense.

3.5.2 Structure Discretization

Figure 3.13 illustrates the finite element mesh chosen to model three double-cell prestressed concrete rectangular box beams tested in the experimental program. In specifying structural geometry, the global x axis must be parallel to the beam's longitudinal centroidal axis.

3.5.3 Input Specifications

In the preparation of the input data file, reference should be made to Appendix H in which a detailed card-by-card description of the input data is given, together with implicit logic assumptions that have a direct bearing on data derivation. For ease of preparation, the input format is semi-free field, as illustrated in Appendix F which lists a sample input data file.

3.5.4 Output Description and Interpretation

The form of the printed output is governed by the choice of the output control variables described in Appendix H. Independent of specified controls, increment headings are printed, together with local material failure messages and progressive CPU and program cost estimates. For each load increment, structure deformations, concrete centroidal stresses and strains in the global and principal axes directions, steel mesh and bar reinforcement stresses and strains, and increment and total load levels can be selectively printed on an element or category basis. Units of pounds wt., inches, and radians are used throughout.

In the interpretation of output deformations, all values correspond to displacements in structural degree of freedom directions. The results must be processed further to determine cross-sectional rotation, beam curvature, and other such descriptive deformations. For both concrete and steel mesh elements, all stress and strain estimates are derived for the centroidal element location. Since the reinforcement bar element is one-dimensional, the respective stress-strain values are constant for the element's length. Structural failure is defined as occurring when a concrete element crushes or a prestress strand fails in tension. Thus, the failure statement that immediately precedes run termination is only of two possible forms. However, structural failure can develop through instability, such a failure mode resulting in the formation of an ill-conditioned set of equilibrium equations. Under such a situation program progress ceases when a negative term is encountered on the main diagonal of the total stiffness matrix during the reduction process.

Since the success of the finite element method of analysis is reflected in the quality of the program input, careful consideration should be given to the evaluation of the numerous strength of materials parameters, in particular those that have a strong influence on post-cracking behaviour.

Element Type	McCleod	Scordelis	Sisodiya-Ghali
% Below Theoretical Bending Deflection	+1.55	+ .6	-3.43
% Rotation Variation w.r.t. Theoretical Value	1.97	1.04	2.0

TABLE 3.1 DEFORMATIONS CONSIDERED IN CONCRETE FINITE ELEMENT SELECTION

$$
\begin{bmatrix}
\frac{1}{8B^2} & & & \frac{-1}{8B^2} & & \frac{-1}{4B} & \frac{1}{8B^2} & & & \frac{-1}{8B^2} & & \frac{1}{4B} \\
& \frac{-3}{8B} & \frac{-A}{4B} & & \frac{1}{8B} & & & \frac{3}{8B} & \frac{-A}{4B} & & \frac{-1}{8B} & \\
& \frac{1}{4AB} & & & \frac{-1}{4AB} & & & \frac{1}{4AB} & & & \frac{-1}{4AB} & \\
\frac{1}{8} & & & \frac{3}{8} & & \frac{B}{4} & \frac{1}{8} & & & \frac{3}{8} & & \frac{-B}{4} \\
& \frac{3}{8} & \frac{A}{4} & & \frac{1}{8} & & & \frac{3}{8} & \frac{-A}{4} & & \frac{1}{8} & \\
\frac{-1}{4B} & & & \frac{1}{4B} & & & \frac{1}{4B} & & & \frac{-1}{4B} & & \\
\frac{-1}{8A} & & & \frac{-3}{8A} & & \frac{-B}{4A} & \frac{1}{8A} & & & \frac{3}{8A} & & \frac{-B}{4A} \\
& \frac{-1}{4A} & & & \frac{-1}{4A} & & & \frac{1}{4A} & & & \frac{1}{4A} & \\
& \frac{-1}{8A^2} & \frac{-1}{4A} & & \frac{1}{8A^2} & & & \frac{-1}{8A^2} & \frac{1}{4A} & & \frac{1}{8A^2} & \\
\frac{-1}{8AB^2} & & & \frac{1}{8AB^2} & & \frac{1}{4AB} & \frac{1}{8AB^2} & & & \frac{-1}{8AB^2} & & \frac{1}{4AB} \\
& \frac{-1}{8A^2B} & \frac{1}{4AB} & & \frac{1}{8A^2B} & & & \frac{-1}{8A^2B} & \frac{1}{4AB} & & \frac{-1}{8A^2B} & \\
\frac{1}{4AB} & & & \frac{-1}{4AB} & & & \frac{1}{4AB} & & & \frac{-1}{4AB} & &
\end{bmatrix}
$$

TABLE 3.2(A). $[A]^{-1}$ FOR ELEMENT TYPE 1

$$
\begin{bmatrix}
\frac{-1}{8B^2} & & \frac{1}{4B} & \frac{1}{8B^2} & & & \frac{-1}{8B^2} & & \frac{-1}{4B} & \frac{1}{8B^2} & & \\
& \frac{-1}{8B} & & & \frac{3}{8B} & \frac{A}{4B} & & \frac{1}{8B} & & & \frac{-3}{8B} & \frac{A}{4B} \\
& \frac{1}{4AB} & & & \frac{-1}{4AB} & & & \frac{1}{4AB} & & & \frac{-1}{4AB} & \\
\frac{3}{8} & & \frac{-B}{4} & \frac{1}{8} & & & \frac{3}{8} & & \frac{B}{4} & \frac{1}{8} & & \\
& \frac{1}{8} & & & \frac{3}{8} & \frac{A}{4} & & \frac{1}{8} & & & \frac{3}{8} & \frac{-A}{4} \\
\frac{-1}{4B} & & & \frac{1}{4B} & & & \frac{1}{4B} & & & \frac{-1}{4B} & & \\
\frac{-3}{8A} & & \frac{B}{4A} & \frac{-1}{8A} & & & \frac{3}{8A} & & \frac{B}{4A} & \frac{1}{8A} & & \\
& \frac{-1}{4A} & & & \frac{-1}{4A} & & & \frac{1}{4A} & & & \frac{1}{4A} & \\
& \frac{1}{8A^2} & & & \frac{-1}{8A^2} & \frac{-1}{4A} & & \frac{1}{8A^2} & & & \frac{-1}{8A^2} & \frac{1}{4A} \\
\frac{1}{8AB^2} & & \frac{-1}{4AB} & \frac{-1}{8AB^2} & & & \frac{-1}{8AB^2} & & \frac{-1}{4AB} & \frac{1}{8AB^2} & & \\
& \frac{-1}{8A^2B} & & & \frac{-1}{8A^2B} & \frac{-1}{4AB} & & \frac{1}{8A^2B} & & & \frac{1}{8A^2B} & \frac{-1}{4AB} \\
\frac{1}{4AB} & & & \frac{-1}{4AB} & & & \frac{1}{4AB} & & & \frac{-1}{4AB} & &
\end{bmatrix}
$$

TABLE 3.2(B) $[A]^{-1}$ FOR ELEMENT TYPE 2

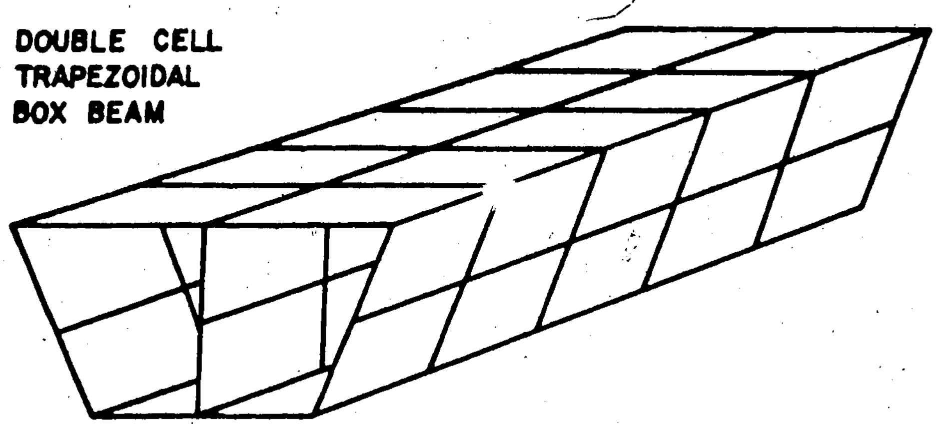
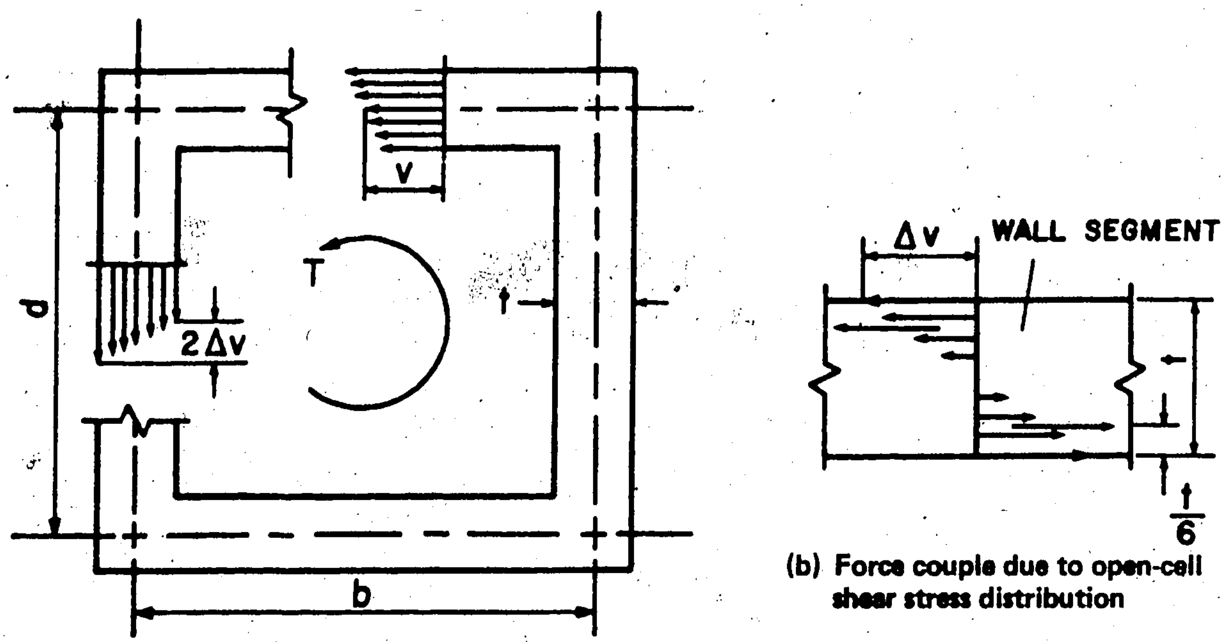


FIG. 3.1 SIMPLIFIED REPRESENTATIVE FINITE ELEMENT MESH



(a) Single-celled box girder

(b) Force couple due to open-cell shear stress distribution

FIG. 3.2 NOTATION FOR DERIVATION OF ST. VENANT SHEAR STRESS DISTRIBUTION

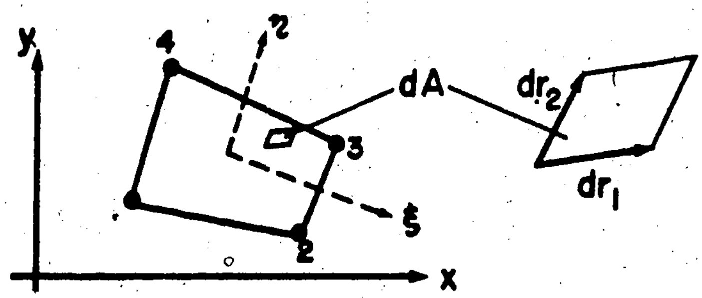
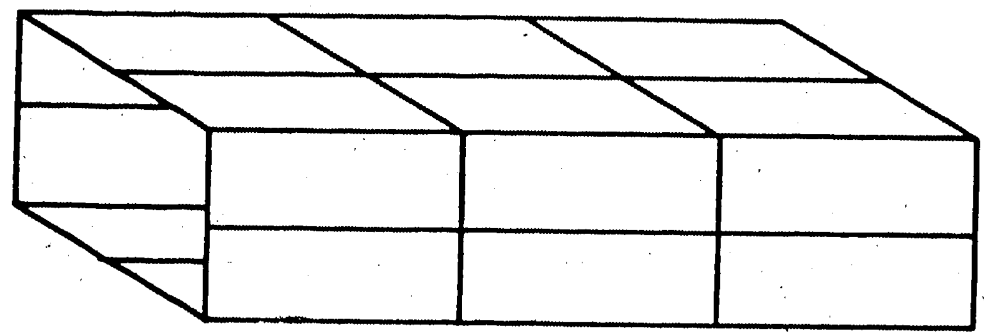
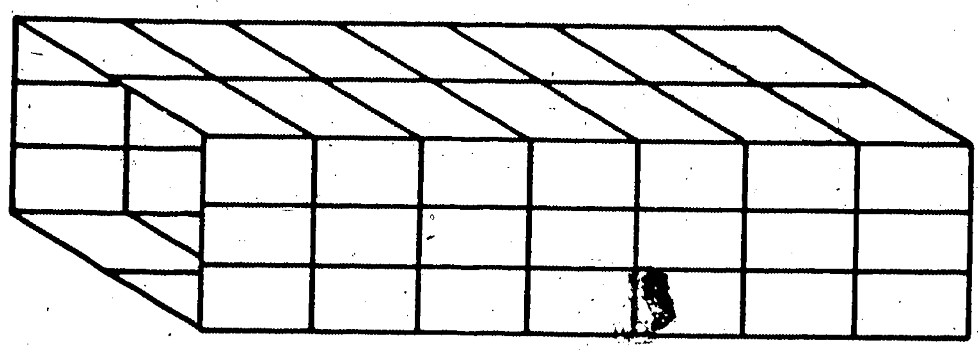


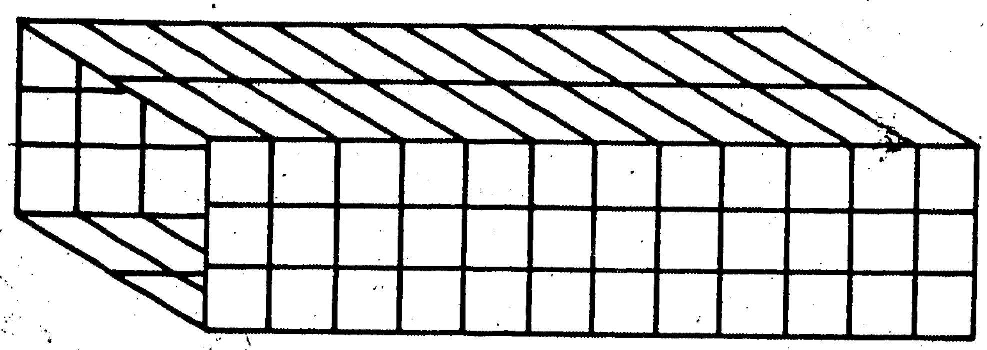
FIG. 3.3 BI-LINEAR ISOPARAMETRIC SERENDIPITY FINITE ELEMENT



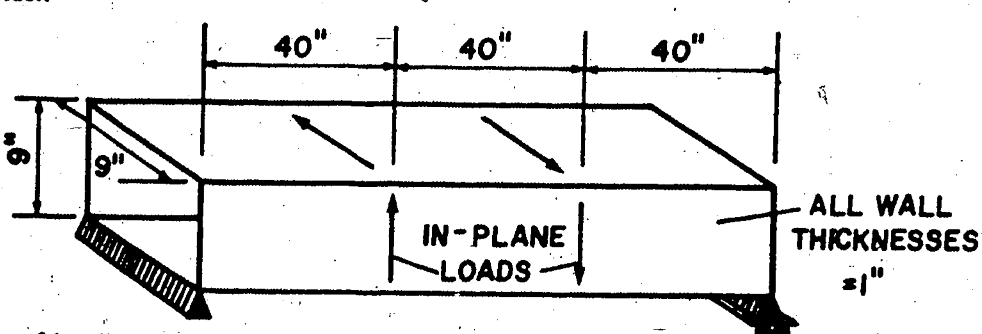
(a) Coarse mesh



(b) Finer mesh



(c) Finest mesh



(d) Location of loading cross-sections

FIG. 3.4 THREE FINITE ELEMENT MESHES USED IN CONCRETE SELECTION PROCESS

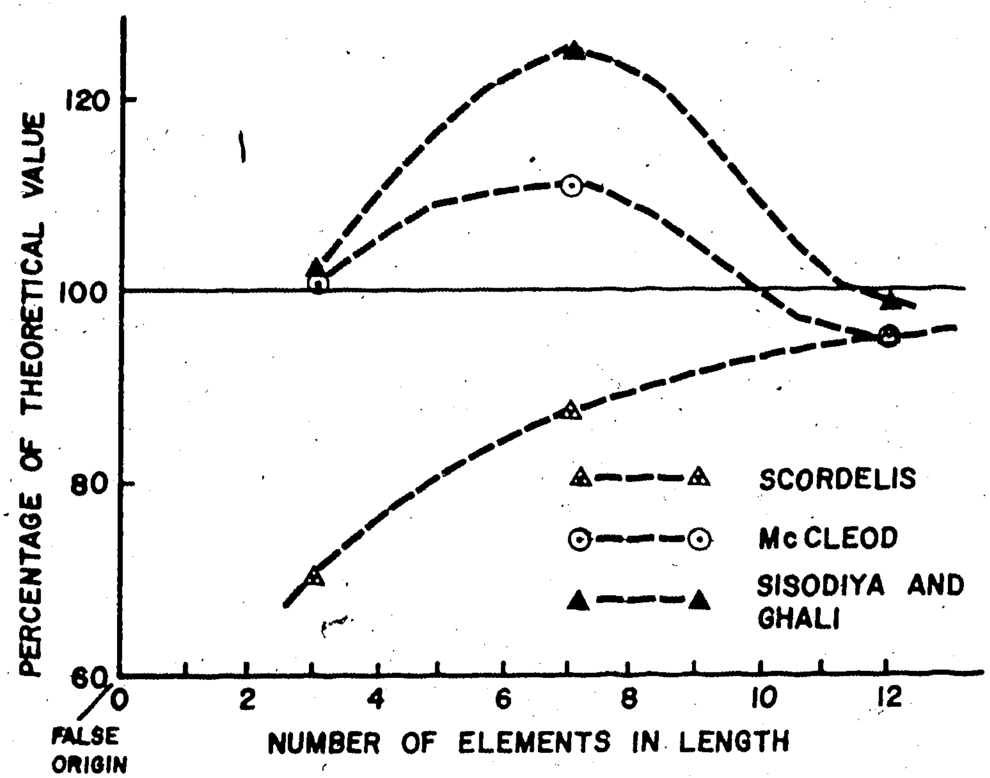


FIG. 3.5(A) DIRECT BENDING STRESSES FOR THREE CONCRETE FINITE ELEMENTS

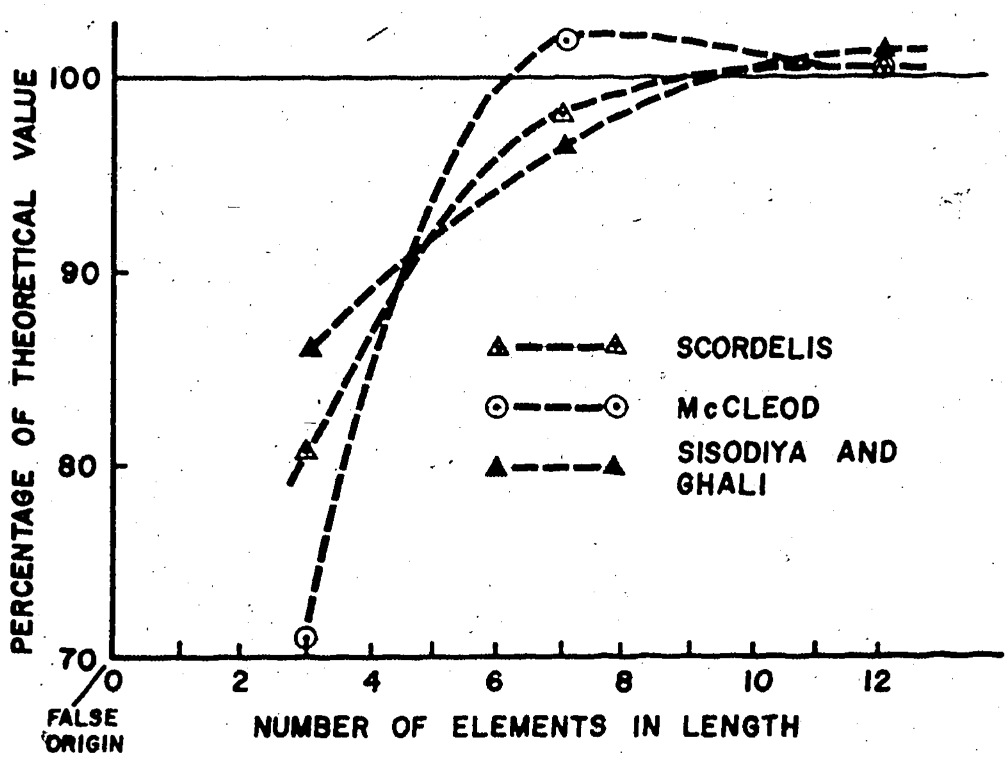


FIG. 3.5(B) ST. VENANT TORSION SHEAR STRESSES FOR THREE CONCRETE FINITE ELEMENTS

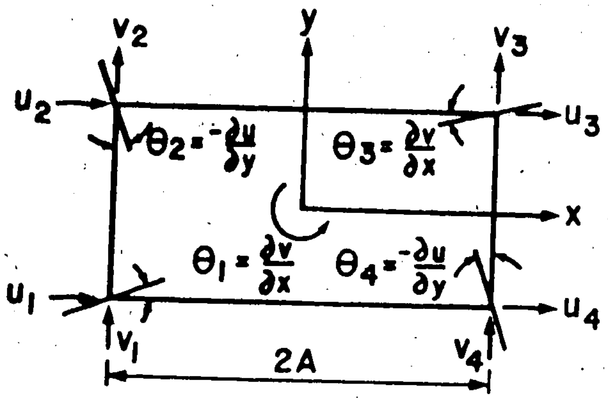


FIG. 3.6(A) ELEMENT TYPE 1

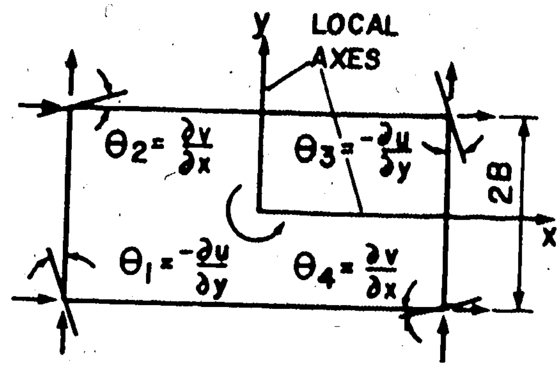


FIG. 3.6(B) ELEMENT TYPE 2

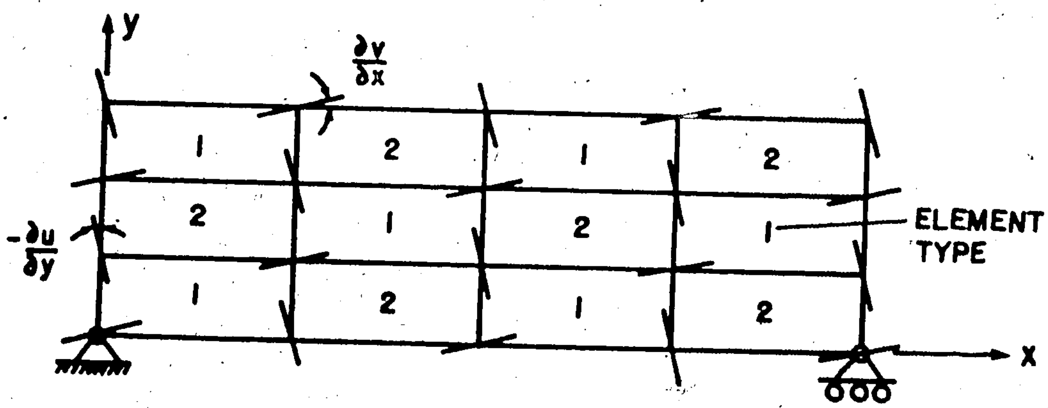


FIG. 3.6(C) TWO-DIMENSIONAL BEAM ASSEMBLAGE OF McCLEOD FINITE ELEMENTS

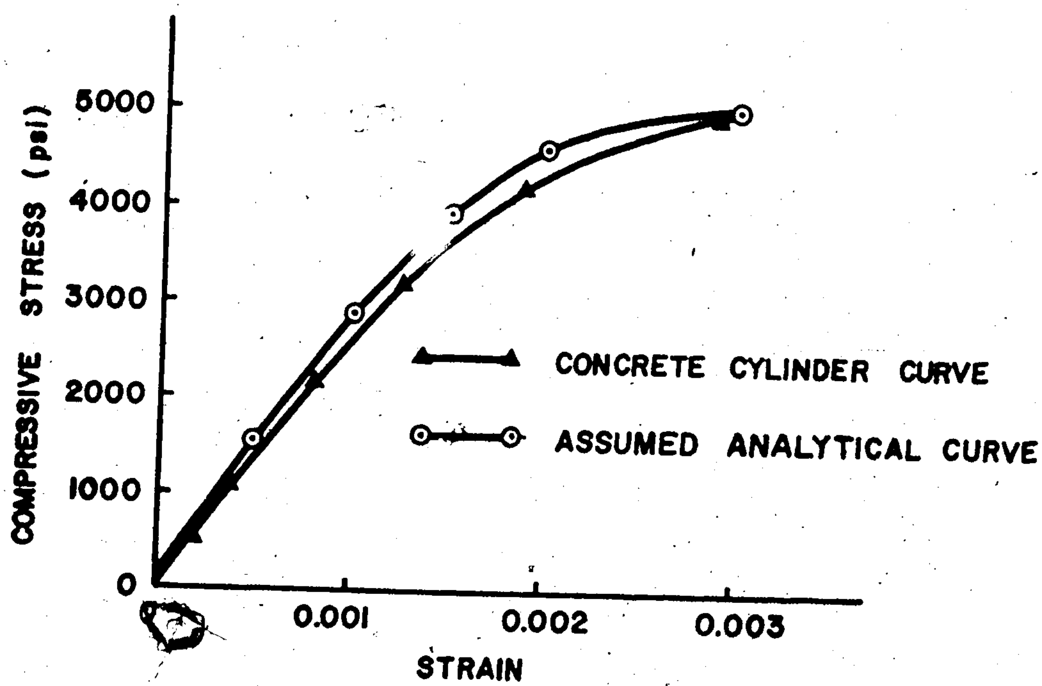


FIG. 3.7 THEORETICAL AND EXPERIMENTAL CONCRETE COMPRESSION STRESS-STRAIN CURVES

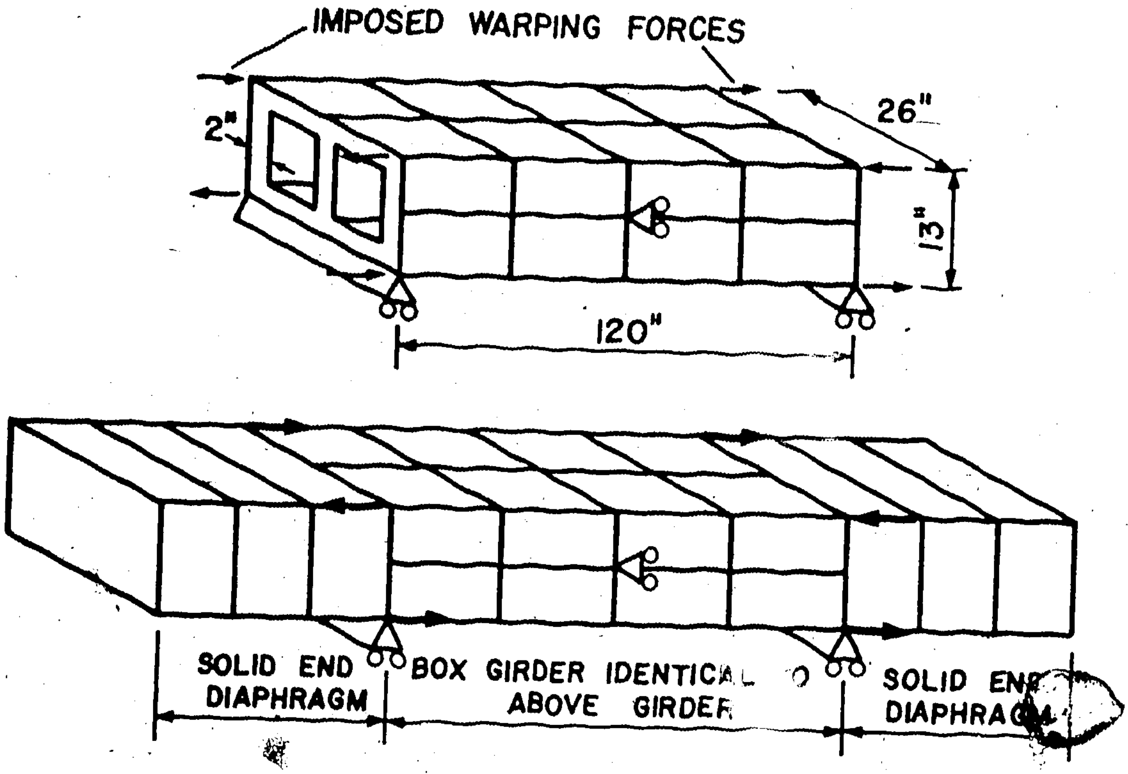


FIG. 3.8 BOX BEAM MODELS WITH THICK DIAPHRAGMS

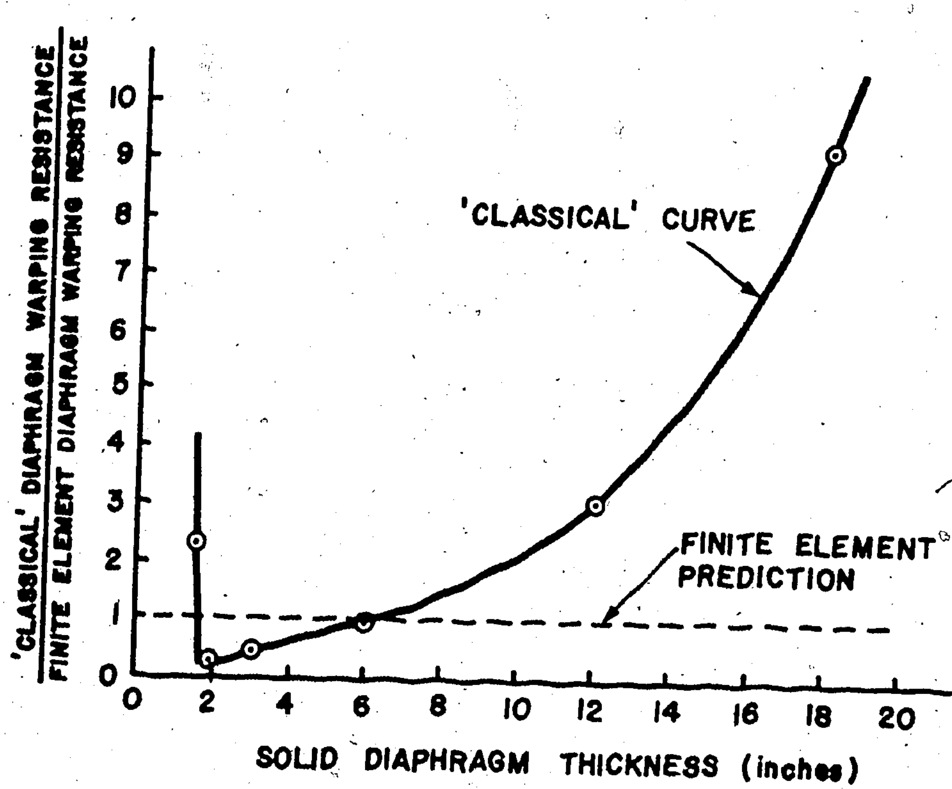


FIG. 3.9 DIAPHRAGM WARPING RESTRAINT CURVES

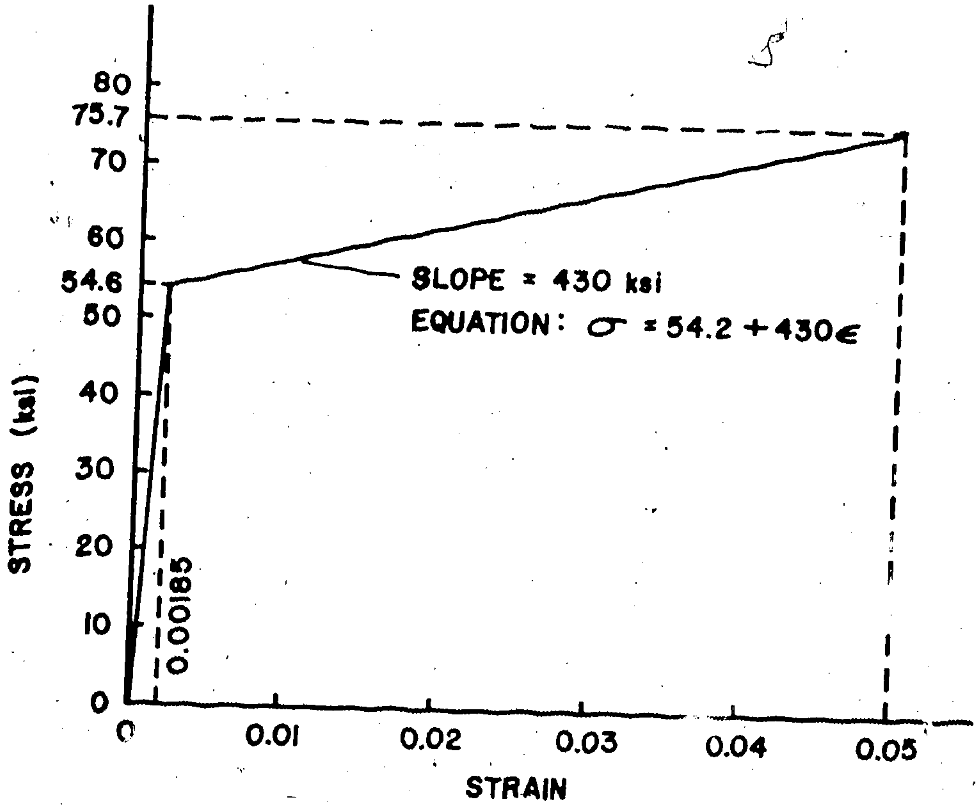


FIG. 3.10(A) CONVENTIONAL REINFORCEMENT STRESS-STRAIN CURVE

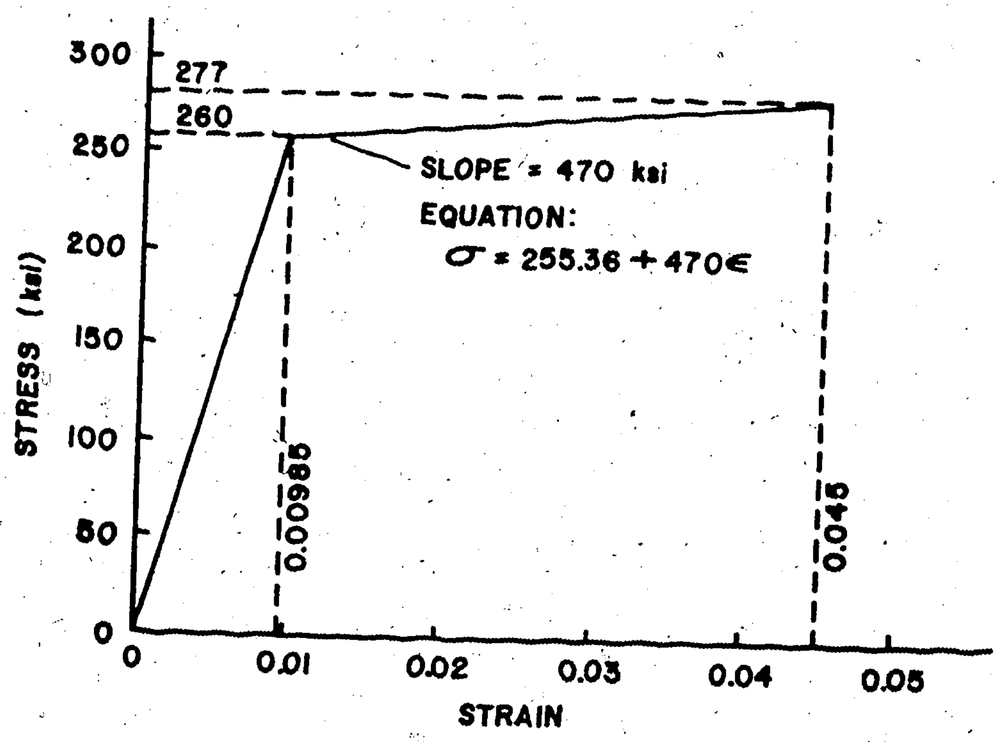


FIG. 3.10(B) PRESTRESS REINFORCEMENT STRESS-STRAIN CURVE

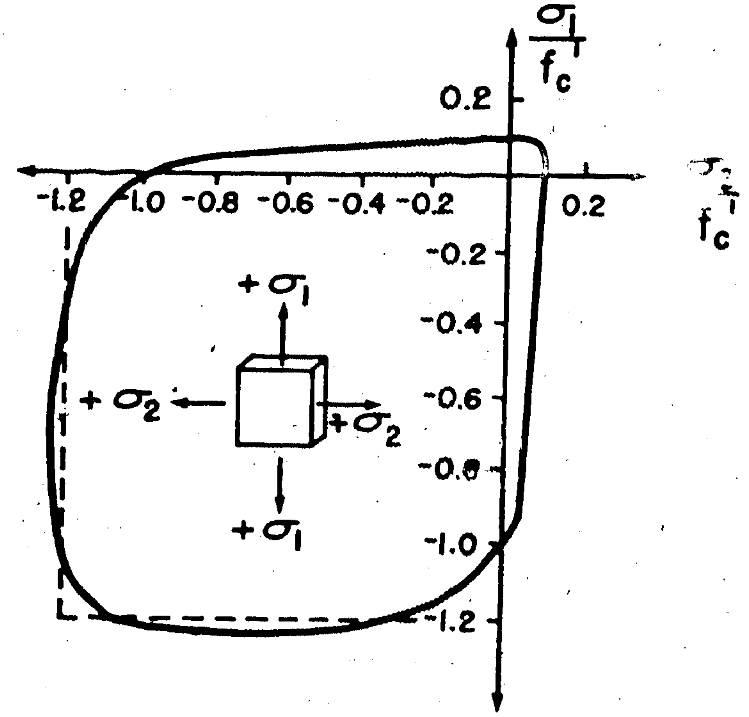


FIG. 3.11 KUPFER, HILSDORF, RUSH BIAXIAL STRESS ENVELOPE

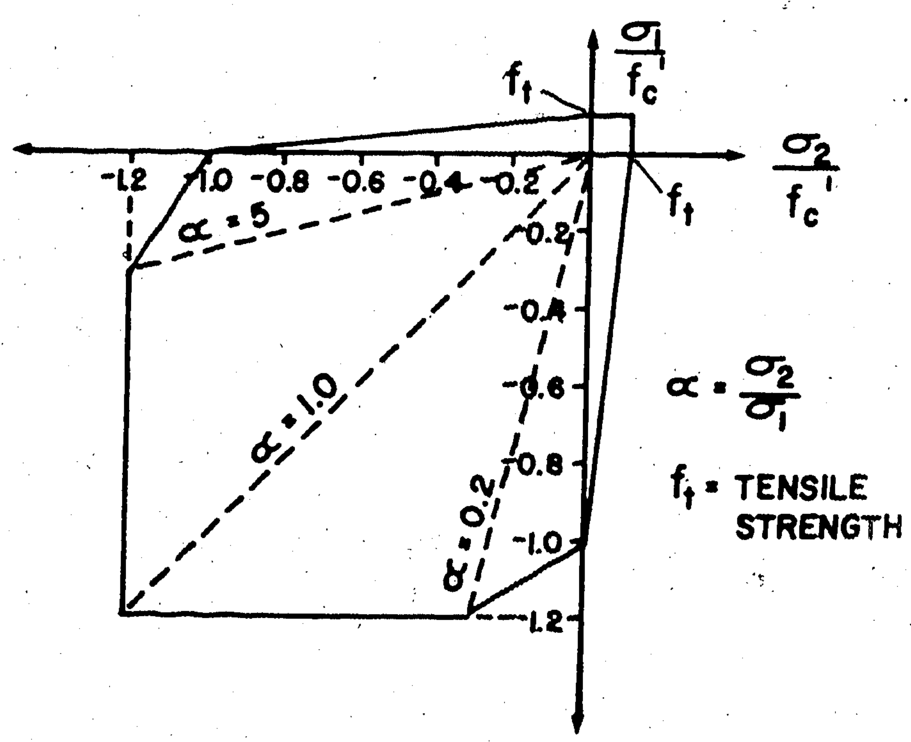


FIG. 3.12 SIMPLIFIED ANALYTICAL MODEL BIAXIAL STRESS ENVELOPE

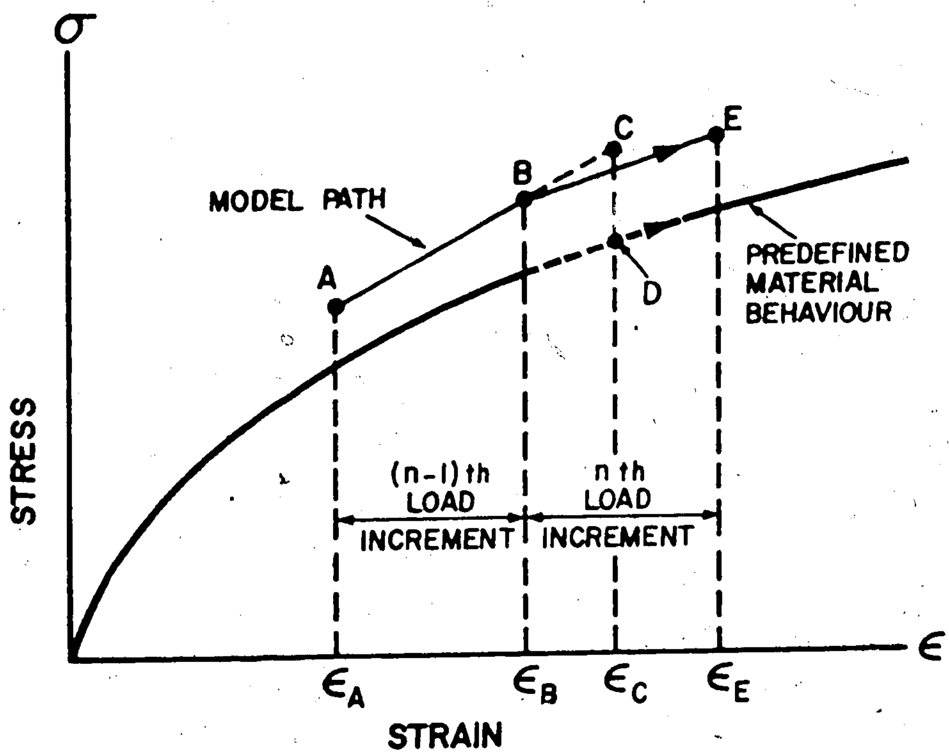


FIG. 3.13(A) RUNGE-KUTTA METHOD

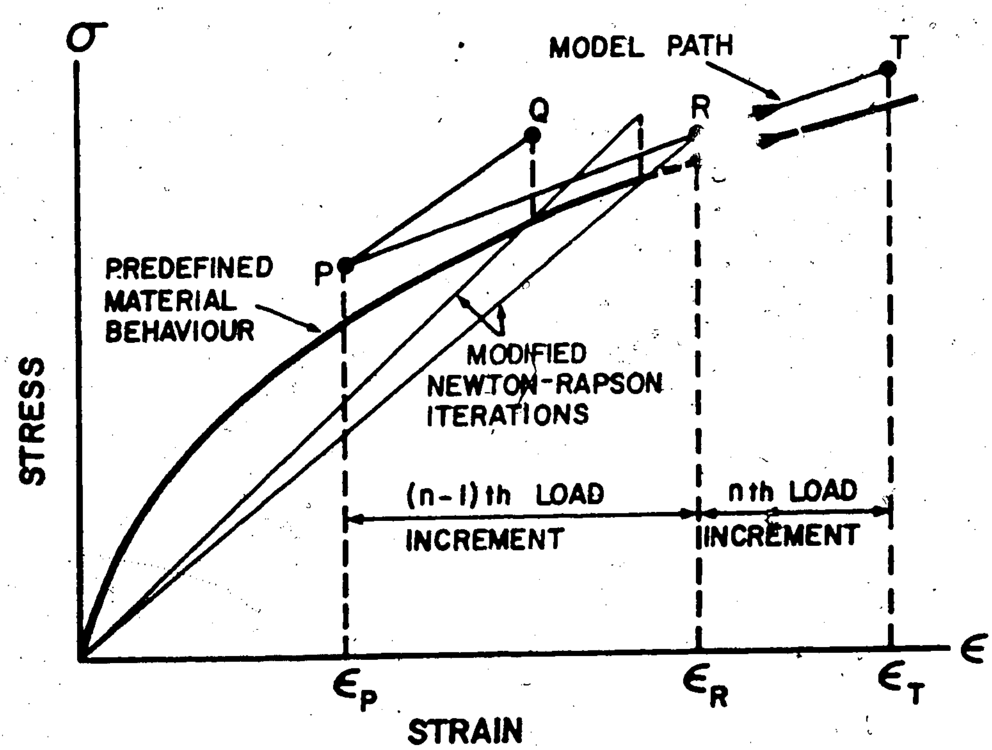


FIG. 3.13(B) MODIFIED NEWTON RAPSON METHOD

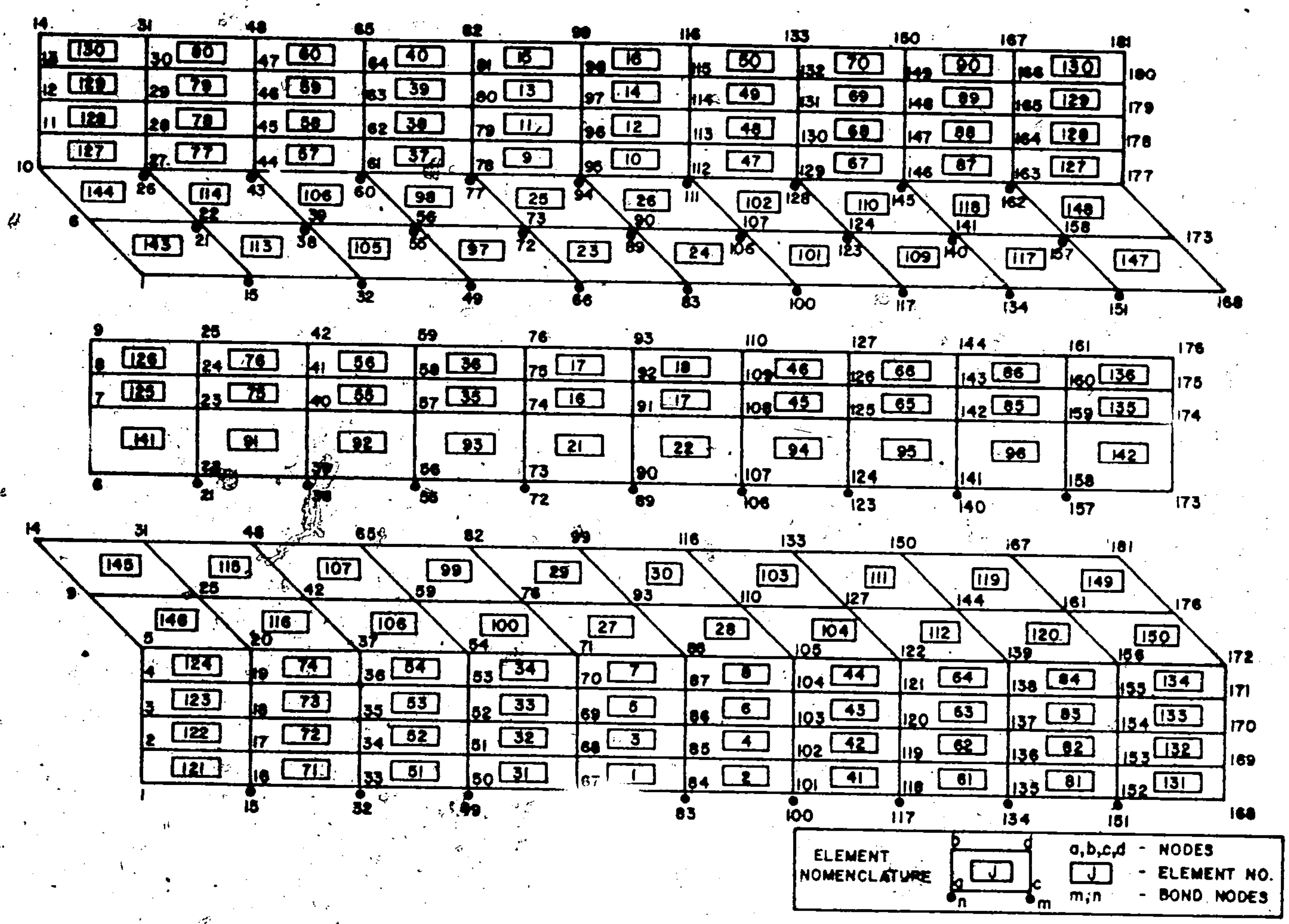


FIG. 3.14 FINITE ELEMENT MESH DISCRETIZATION

CHAPTER 4

EXPERIMENTAL PROGRAM

4.1 Introduction

Consistent with the defined scope, the scale of the experimental program was modest in the number of beam specimens tested and the degree of parameter variation. In all, seven prestressed box beams were cast and tested to failure, with each beam subjected to different ratios of torque to bending moment to shear. All facets of the experimental program are dealt with in this chapter, and the results are presented in the following chapter.

4.2 Definition of Basic Experimental Parameters

In essence, all the test beam specimens were hollow, precast, prestressed concrete members. At the preliminary design stage, it was recognized that, if the analytical model was to be tested rigorously, complex beam cross-sections of more than one cell should be tested. An extensive literature survey revealed that little if any experimental research had been conducted on the post-cracking behaviour of multi-celled members under the action of torsion, bending, and shear. From an analytical stance of generality, little benefit would have been derived in having more than two cells per beam, and practical expedience in the casting and testing sequences dictated against an excessive number of voids. Thus, the decision was made that all beams would be cast with two closely spaced voids.

Since the experimental tests were to be used strictly for comparative purposes, the number of beams cast for the testing program was small. The only principal experimental parameter whose variation was investigated in the series of tests was the load parameter. The more significant aspect of this parameter was not the level of individual loads, but the ratio of bending moment to torque to shear. In all, seven beams were cast, with five beams being subjected to varying ratios of torque to bending moment with little shear present. The remaining two beams were subjected to significant levels of shear, in conjunction with the more predominant torque-bending moment loading combination.

The primary region of concern in the full range of test beam behaviour was the post-cracking region. Thus, the initial design of the test specimens had to meet the important prerequisite that the inelastic behavioural path beyond cracking was of reasonable length compared with the respective elastic path. This condition was assured by not permitting the ratio of ultimate strength to cracking strength to fall much below the value of two.

To introduce a degree of generality of cross-sectional shape in the experimental study, two geometrical shapes were adopted in the beam design; rectangular and trapezoidal. Of the total of seven beams, five were cast with rectangular sections.

The one overriding consideration in the selection of void sizes was that all wall segments must be classified as "thin". The need for thin wall segments arose from the use of plane stress finite elements to model the beam walls. The theoretical classification as to whether a beam cross-section is "thick" or "thin", is treated in detail in 3.2.1.

4.3 Design of Test Specimens

All beams in the test program were precast, prestressed concrete members with modest conventional reinforcement present. Beams were categorized as belonging to either of the two test beam series according to their cross-section shape, those rectangular beams belonging to the "R" series and those trapezoidal assigned to the "T" series.

In all seven beams, a design concrete strength of 5,000 p.s.i. was adopted. To achieve the appropriate workability that would allow the small wall thicknesses to be cast free of air voids, the mix design was made considerably richer than was generally required, the actual concrete component proportions being given in Table 4.1. The respective compressive and tensile concrete strengths for the seven beams are given in Table 4.2. The load versus elongation curves for the remaining two material components, the prestress strand and the conventional reinforcement, are displayed in Figures 4.1 and 4.2 respectively. As a point of clarification, the prestress strand was 250K grade, 1/4" diameter seven strand stress relieved cable, whereas the conventional reinforcement used throughout was #3 deformed mild steel.

In defining full beam geometry, beam length, wall thickness, number of cells, cross-sectional shape and dimensions have to be specified. Of the three variables that are dimensionally functions of length, the independent variable was the wall thickness. This beam dimension had to be sufficiently large such that the prestress and conventional reinforcement could be accommodated without drastically impeding the flow of concrete during casting. As mentioned previously, a wall thickness of 2" was chosen for all beams. Having specified the wall thickness, the dependent variables of cross-section dimensions and beam length were determined upon recognition of their respective constraints. The web height and flange width had to be a considerable order of magnitude larger than the wall thickness to ensure a reasonable degree of plane stress action of the concrete beam walls, and the beam length such that a central test section for deformation measurement and the appropriate loading and support apparatus could be accommodated. The remaining two geometrical variables of cross-sectional shape and number of cells have been treated in Section 4.2. Complete beam dimensioning is given in Figures 4.3 and 4.4.

In design, the appropriate level of prestress was chosen such that, at transfer four days after casting, the maximum allowable tensile stresses were developed in the top flange. In all beam designs, the bottom flange compressive stresses did not approach the allowable limits.

In the calculation of the ultimate bending moment capacity, the level of reinforcement was such that the concrete crushed just prior to the prestress strand developing its ultimate strength. In establishing the ultimate torsional capacity, the space truss analogy equations as presented by Collins and Lampert[25] were used to estimate beam strength.

As mentioned in the previous section, the three distinct loading types represented in this experimental program were bending moment, torque, and shear. Of the five rectangular beams cast, three were subjected to combined bending and torsion only, the remaining two acted upon by complete bending-torsion-shear loading combinations. The two trapezoidal beams were restricted to bending and torsion loads. In predicting beam strength under combined loading, the interactive equations given in the Collins and Lampert[25] paper were utilized, such equations given in Fig. 4.5 together with the design curves for the rectangular and trapezoidal beams. The predicted beam strengths and failure loads are given in Table 4.3.

Premature shear failure was prevented in the beam length outside the central test section by the provision of substantial shear reinforcement, achieved in most beams by extending torsion hoop reinforcement beyond the central test section. Reinforcement details are provided in Figures 4.3 and 4.4. Also shown in the latter figures are the locations of the points of application of the bending, torsion and shear loads.

4.4 Specimen Fabrication

To enable both rectangular and trapezoidal beams to be cast with the same forming unit, the timber formwork consisted of two identical reversible form segments placed adjacent to one another, as shown in Fig. 4.6.

Provision of the two adjacent cells within each beam was simply achieved by the use of styrofoam blocks held in place by the reinforcement cage. In previous research conducted at this university, the styrofoam was removed by piping acetone into the interior of the cast beams. This added precaution, however, was not deemed necessary as the styrofoam strength contribution was negligible compared with that of the prestressed beam. In all beams, the void lengths did not extend as far as the points of support, as exhibited in Figure 4.4.

As indicated above, the presence of the reinforcement cage was utilized to maintain the positions of the styrofoam blocks. To prevent the styrofoam blocks from floating during cast, restraining beams were placed across the top of the forms to inhibit upward cage movement. Since the cage tolerances were quite small arising from the selection of narrow wall thicknesses, the reinforcement bars and hoops were lightly tack-welded in position to minimize movement and facilitate handling. As a matter of necessity, the two top corner longitudinal bars had to be moved 1" from their corner positions to allow a small vibrating needle to pass down the outside web walls during casting.

Despite the small wall thickness, the rich concrete mix whose proportioning is given in Table 4.1 did exhibit good workability with the result that very few air voids were evident upon examination after formwork stripping.

The complete casting bed before concrete pouring is shown in Plate 4.1.

4.5 Loading Apparatus Design

4.5.1 Beam Supports

In addition to their customary role of simple supports, the beam reactions permitted the development of a uniform St. Venant torque applied across the central test section. Consequently, each support had to allow the beam to rotate freely about its centre of rotation. Lateral stability of the beam was ensured by the very nature of the torsion loading arms, as described in Section 4.5.3. Figure 4.7 illustrates all end support details. The roller housing shown in this figure was bolted to the top of the conventional roller and hinge supports to achieve the desired beam support conditions.

4.5.2 Application of Bending Moment and Shear

Both bending moment and shear force were developed by the application of downward vertical concentrated loads applied by 50 ton Amsler jacks. If a vertical concentrated load was not to create secondary torsion during the course of a beam test, the centre line of the Amsler jack had to pass through the shear centre of the beam. This alignment was achieved through incorporating a design that was identical to that of the rotational end support with the apparatus simply inverted. The one additional design provision not present in the end support design was the roller housing bracing whose sole function was to provide horizontal stability to the roller housing under all loading extremes. As the beam deflected under load, the bracing arms attached to the housing by a central pin maintained their horizontal posture upon adjustment of the bracing jack. The apparatus is displayed in Fig. 4.8.

4.5.3 Application of Torque

Only the central test section was subjected to torque, more specifically a uniform St. Venant torque developed by a pair of equal and opposite forces whose placement along the test beam delineated the central test section. Each torsion force was applied by a cable that draped over a curved torsion arm (as in Fig. 4.9) and passed through the testing floor where it was anchored by a hydraulic jack loading system. The curved arm geometry ensured that the torsion lever arm dimension remained constant as the beam rotated. Stability of a test beam, once the beam was placed in the testing arrangement, was provided by minimal tension in the torsion cables.

4.6 Instrumentation

To plot beam behaviour throughout the test program, both stress and deformation data were recorded. As the seven strand prestress cable was of small diameter (1/4"), strain gages were difficult to attach securely, and thus could not be relied upon to accurately monitor prestress strain levels. However, the strain states of the conventional longitudinal and central transverse hoop reinforcement were monitored by strain gages to yield a representative record of stress versus load behaviour.

More emphasis was placed in the instrumentation phase on the accurate measurement of cross-section deformation within the central test section rather than extensive monitoring of reinforcement strain. By surveying the vertical and horizontal deflections of three cross-sections within the central test section, the central vertical deflection, beam curvature, and rate of twist were able to be evaluated. To facilitate fast test measurement recording, linear displacement transducers were used in preference to conventional dial gages. Plate 4.2 displays several linear displacement transducers operating under test conditions.

In recognition of the fact that the number of both load increments and individual instrument readings was large, an automatic data-recording system was used. The electrical signals from the linear displacement transducers and the strain gages were monitored, translated, and stored on disc by a Data General Corporation Nova 210E Computer. During testing, the depression of a console key prompted the complete data set of load increment measurements to be recorded and written on disc.

All locations of strain gages and linear displacement transducers are described in Chapter 5.

4.7 Testing Procedure

Upon application of each load increment, the entire regime of instrument measurement was recorded automatically as described in the preceding section. If cracking had commenced, crack propagations were clearly marked and the load intensity indicated at the furthest point of crack propagation, as seen in the following chapter's photographic plates of tested beams. As a precautionary measure, the transverse alignment of the Amsler jacks was checked at regular intervals and corrected when necessary by adjustment of the bracing turnbuckles and jacks. All cracking patterns and mode of failure were photographed upon completion of the test.

Ingredient	Wt. in lbs./batch
Cement	211
Sand	420
Coarse Aggregate (3/8")	540
Water	120

Approximate batch volume = 10 ft.^{3}
slump = 3" + 4"

TABLE 4.1 CONCRETE MIX PROPORTIONS

Beam Designation	Concrete Compressive Strength (psi)	Concrete Tensile Strength (psi)	Concrete Young's Modulus (psi)
R1	4816	396	2.36×10^6
R2	4580	462	2.33×10^6
R3	4545	479	2.6×10^6
R4	4362	288	2.69×10^6
R5	4562	329	1.84×10^6
T1	3985	354	3.0×10^6
T2	4262	338	2.79×10^6

TABLE 4.2 CONCRETE STRENGTH

	Beam Designation	R1	R2	R3	R4	R5	T1	T2
	Initial Prestress	79.2	79.2	79.2	79.2	79.2	72	72
	Final Prestress	61.3	61.3	61.3	61.3	61.3	58.2	58.2
	Cracking Bending Moment	715	715	715	715	715	429	429
	Cracking Torque	454	454	454	454	454	229	229
	Ultimate Bending Capacity	1515	1515	1515	1515	1515	985	985
	Ultimate Torsion Capacity	380	380	380	380	380	246	246
Average Values Over Central Test Section at Failure	Bending Moment	1010	1250	960	1150	680	618	811
	Torque	500	372	450	250	639	304	200
	Shear	0.0	0.0	±11	±18	0.0	0.0	0.0
	Ratio of $\frac{\text{Bending Moment}}{\text{Torque}}$	2.02	3.36	2.13	4.6	1.064	2.03	4.05

Note: All units are in Kips and inches.

TABLE 4.3 BEAM DESIGN TABULATION

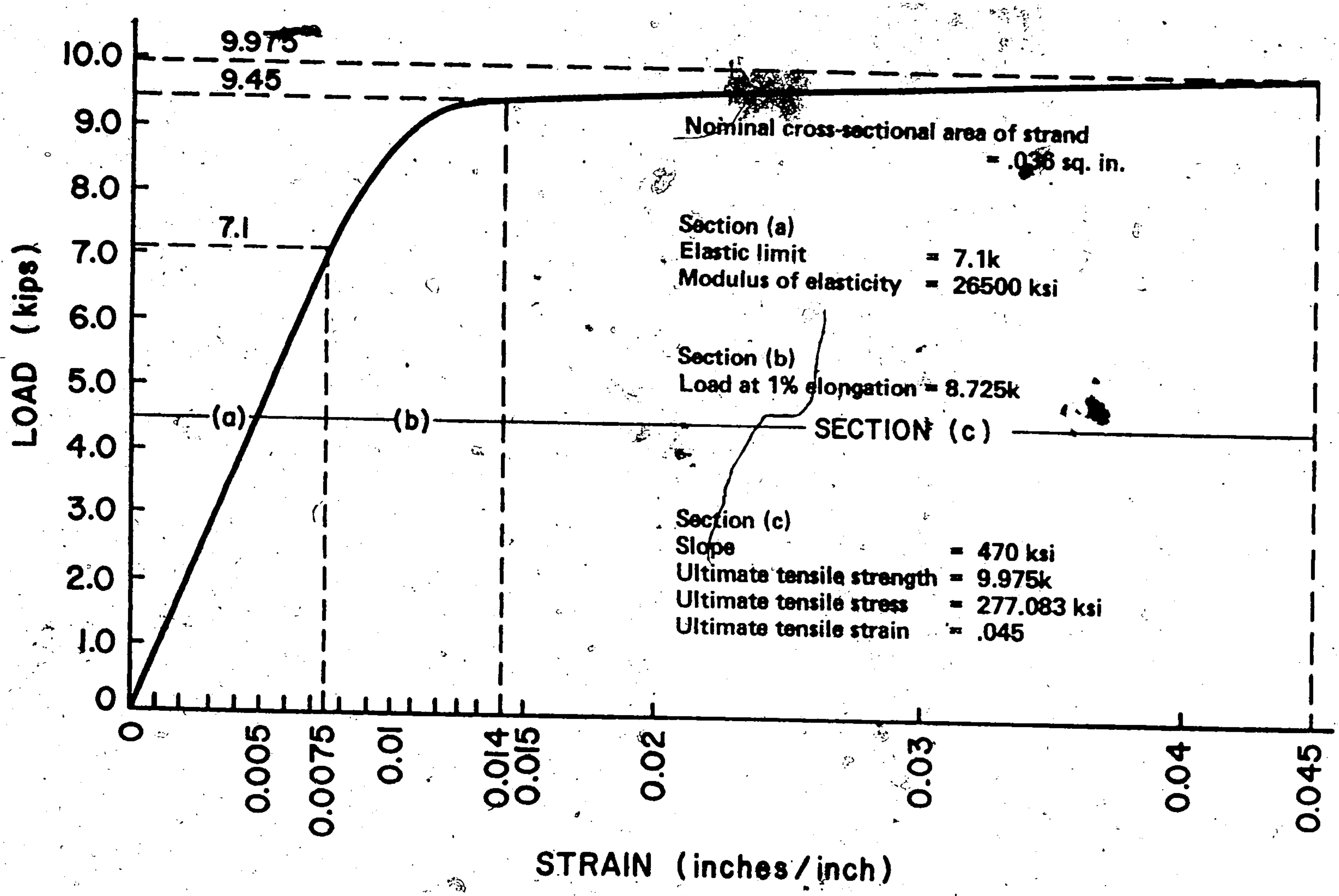


FIG. 4.1 LOAD-ELONGATION CURVE FOR PRESTRESS STRAND

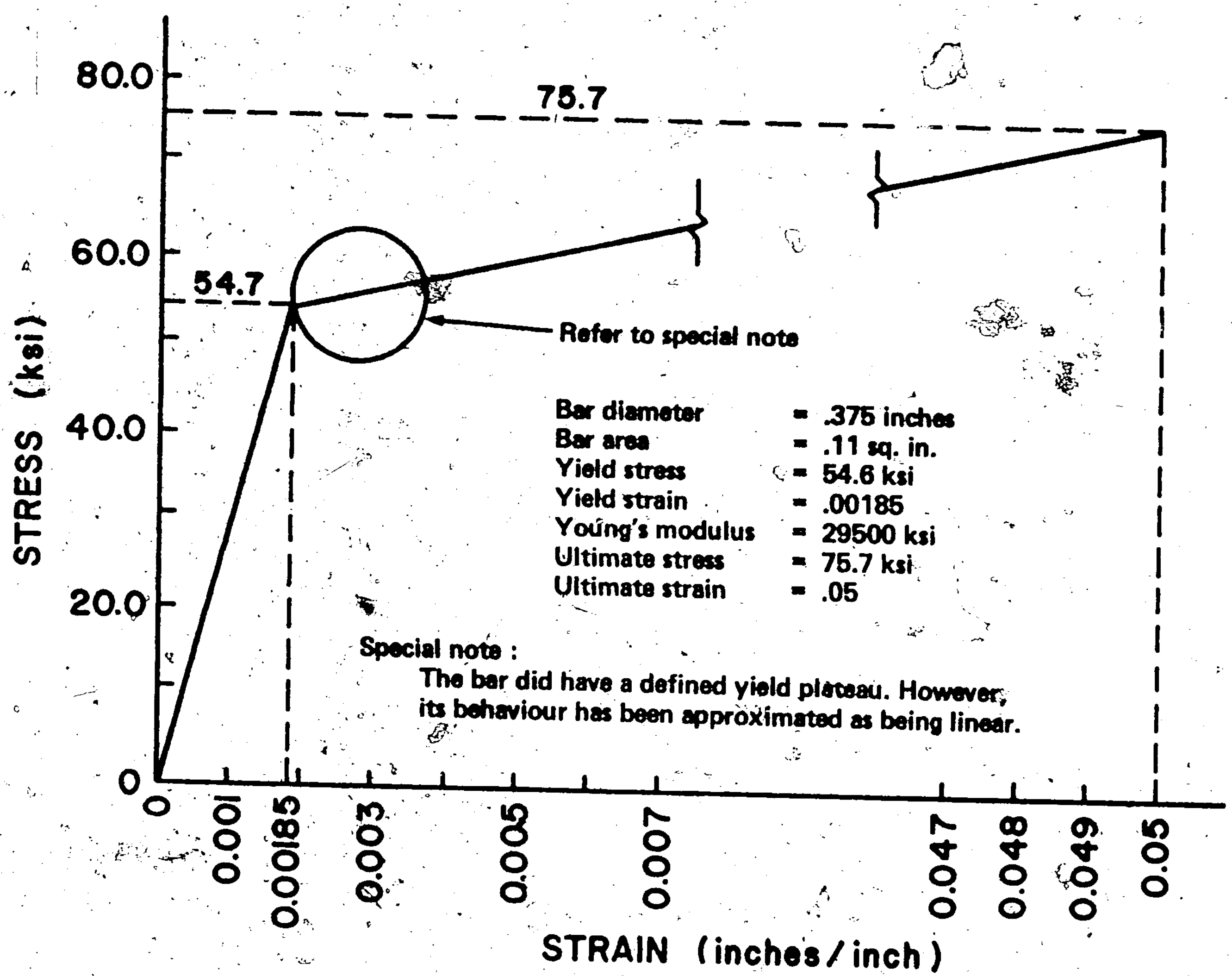


FIG. 4.2 STRESS-STRAIN CURVE FOR #3 DEFORMED BAR REINFORCEMENT

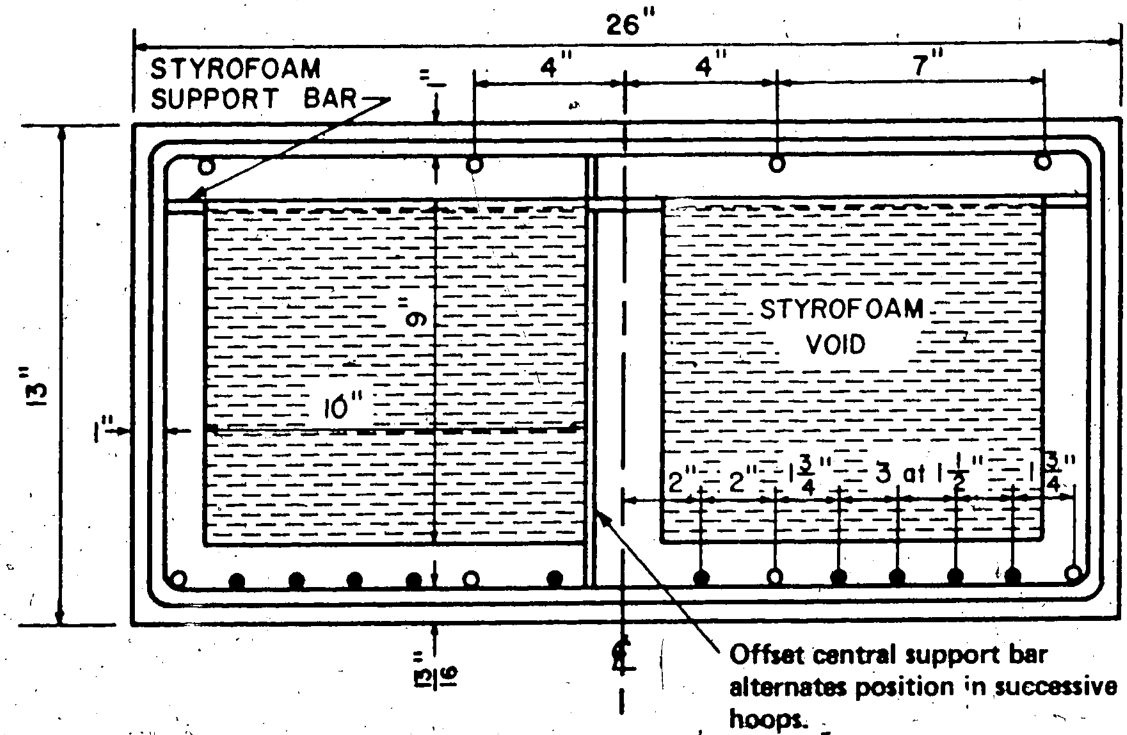


FIG. 4.3(A) RECTANGULAR BEAM CROSS-SECTION

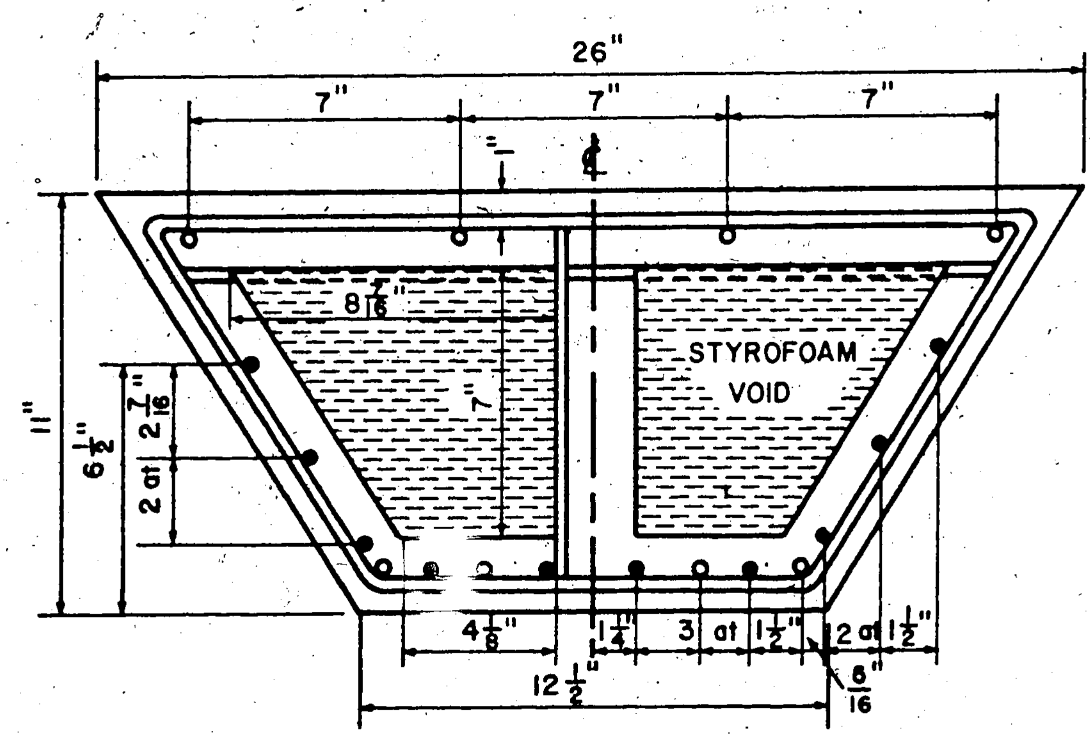
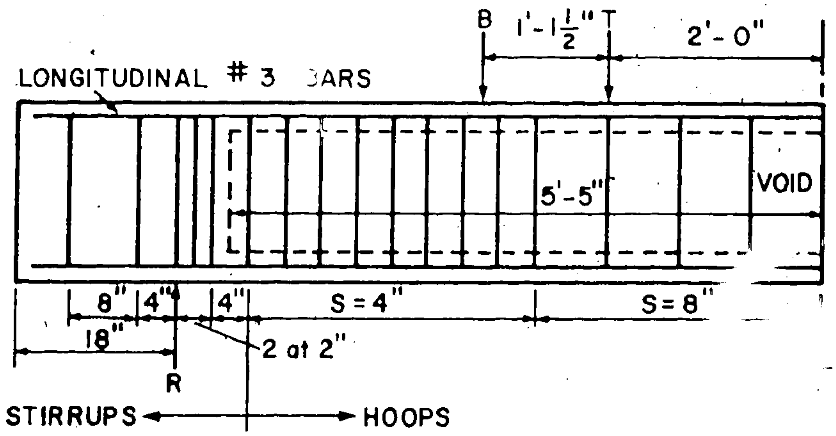


FIG. 4.3(B) TRAPEZOIDAL BEAM CROSS-SECTION

Notes: (1) All wall thicknesses = 2"
(2) ○ Conventional #3 bar ● .25" Prestress strand
(3) All beams are symmetrical w.r.t. vertical centreline



Note: Total void length for beam R1 = 10'- 0"

FIG. 4.4(A) REINFORCEMENT FOR BEAMS R1 AND R2

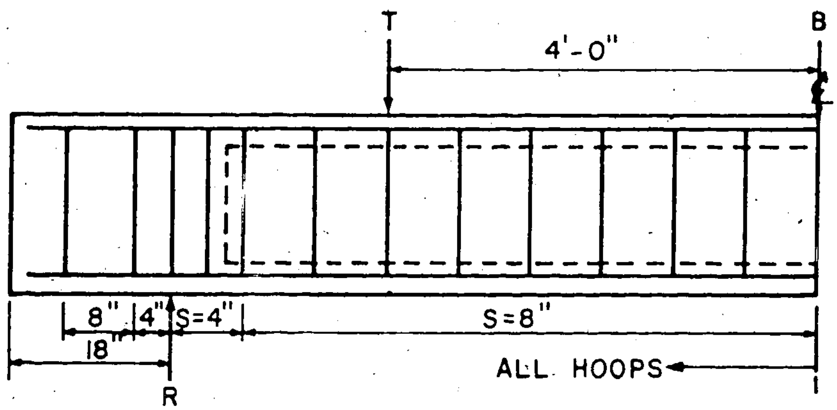


FIG. 4.4(B) REINFORCEMENT FOR BEAMS R3, R4, R5

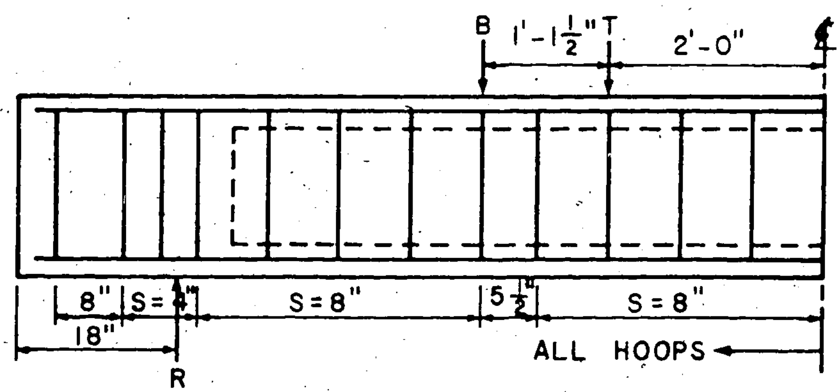


FIG. 4.4(C) REINFORCEMENT FOR BEAMS T1 AND T2

Notes: (1) All bars are #3
(2) All half beam lengths = 7'- 7.5"
(3) R = support B = jack T = torsion
(4) All void lengths = 11'- 0" unless noted otherwise
(5) All longitudinal steel is 1.5" short of beam ends
(6) Drawings are not to scale

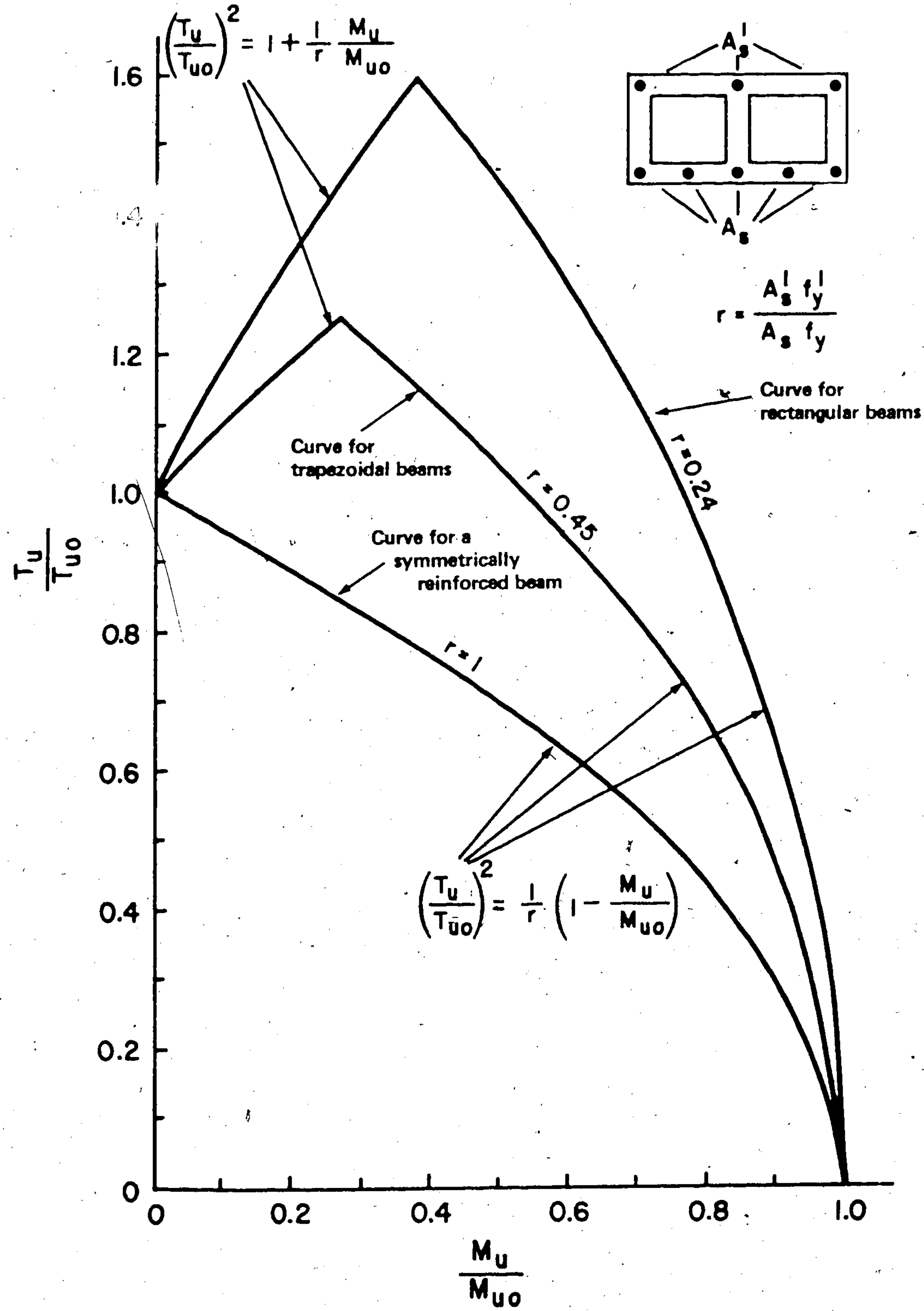
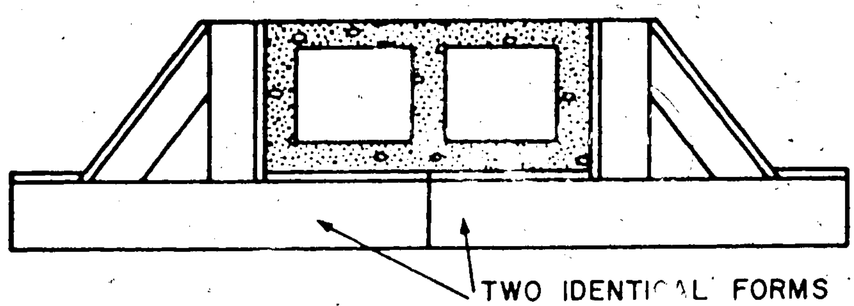


FIG. 4.5 INTERACTION EQUATIONS FOR TEST BEAMS UNDER COMBINED BENDING AND TORSION

Juxtiposition for casting rectangular beams



Juxtipostion for casting trapezoidal beams.

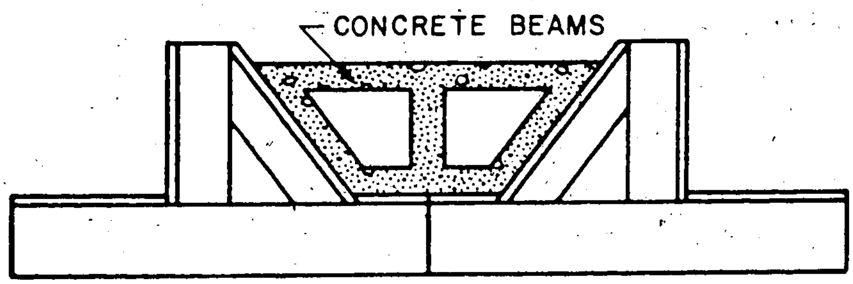


FIG. 4.6 FORMWORK SECTION

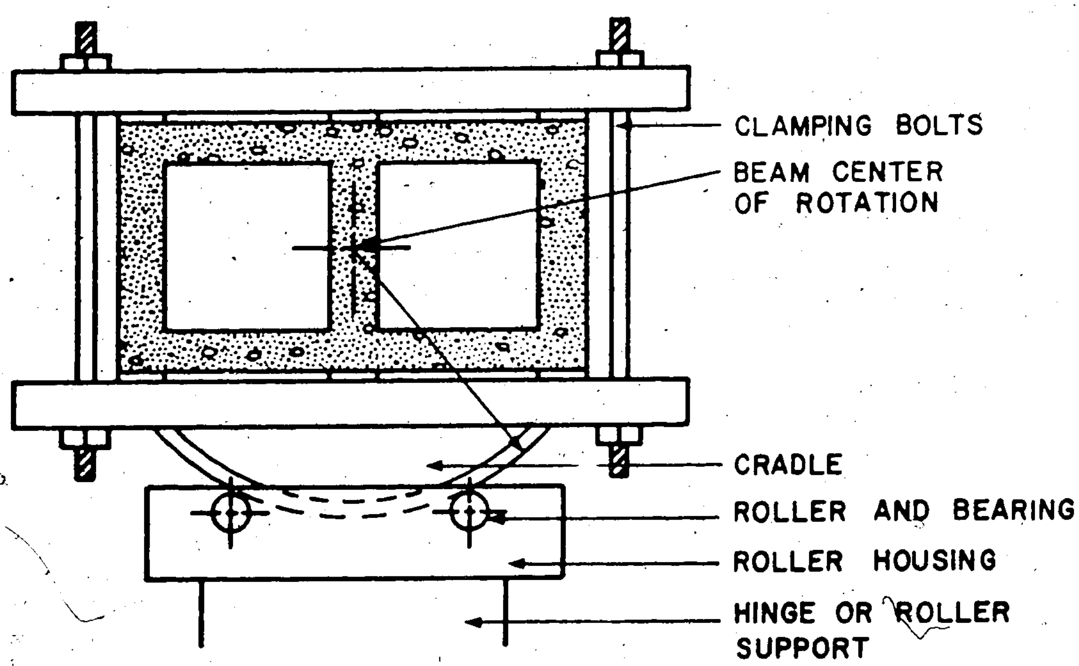


FIG. 4.7 TEST BEAM SUPPORTS

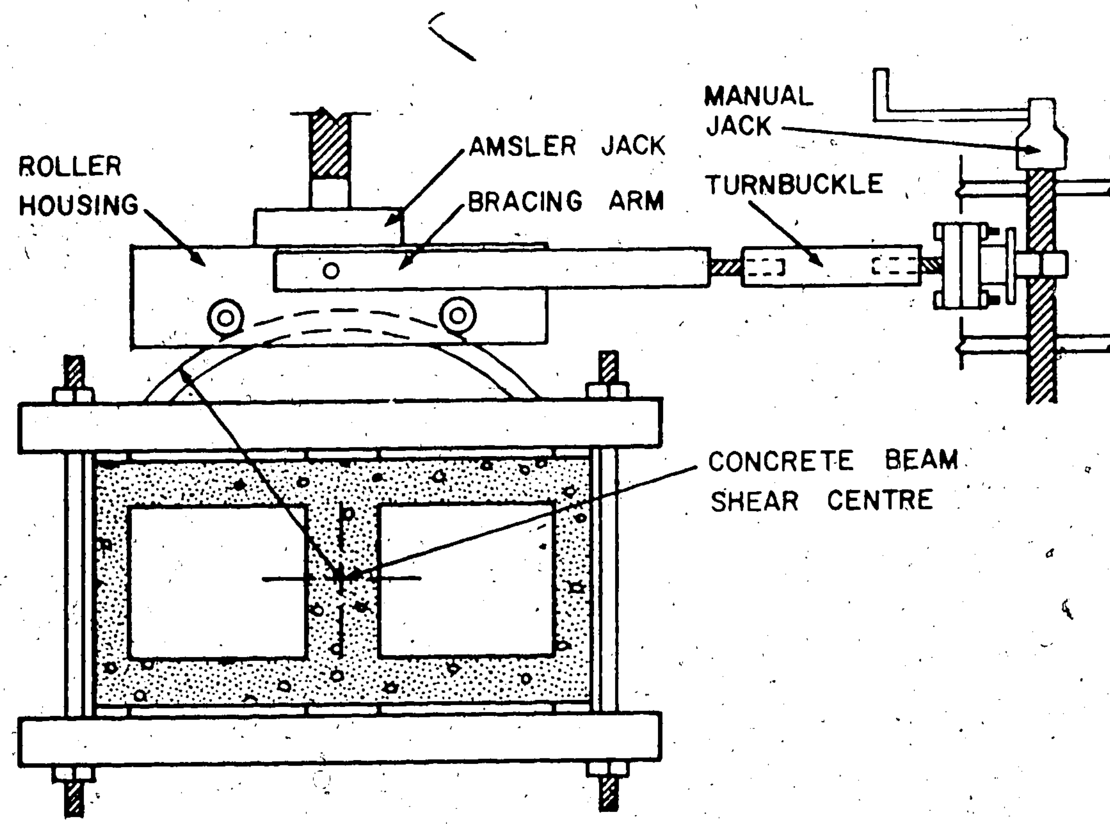


FIG. 4.8 POINT LOAD APPARATUS WITH ROLLER HOUSING

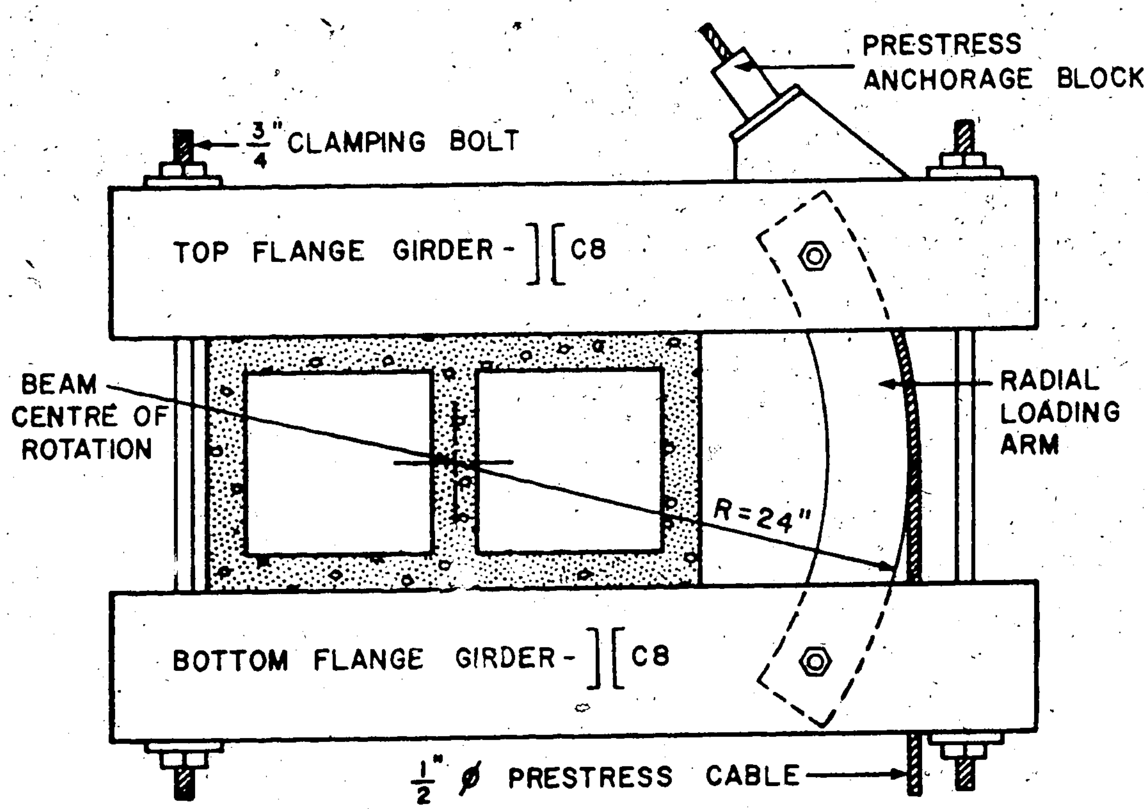


FIG. 4.9 TORSION LOAD ARM

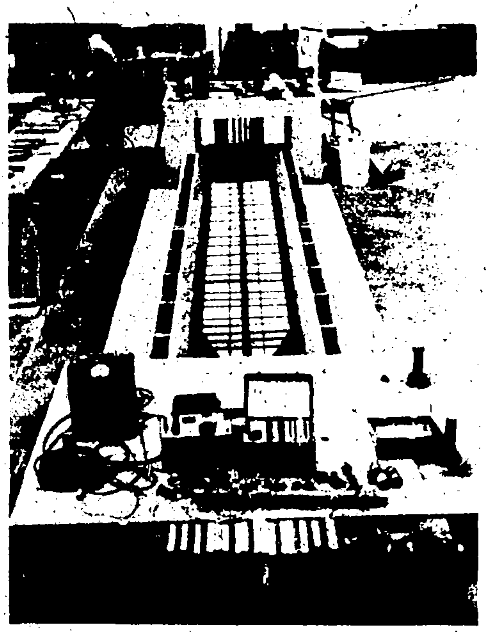


PLATE 4.1 FORMWORK BEFORE CASTING

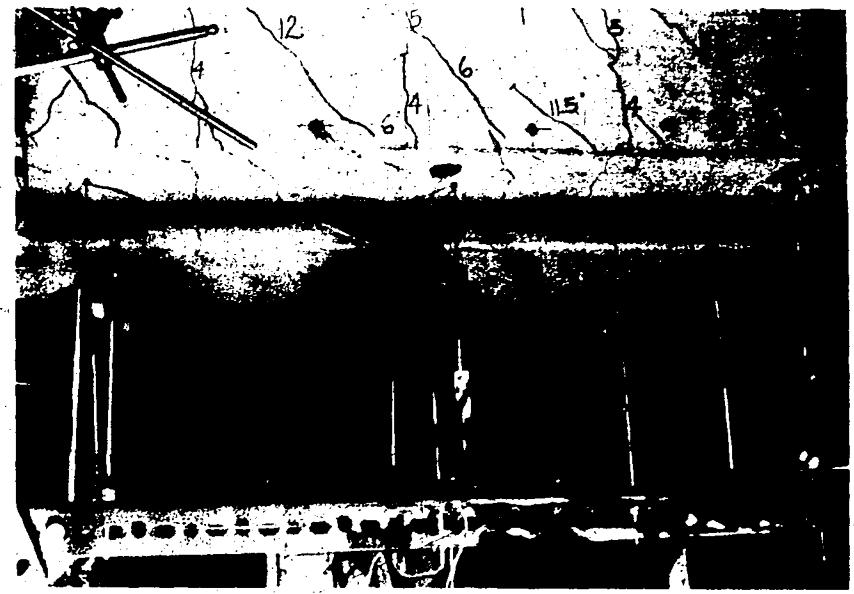


PLATE 4.2 LINEAR DISPLACEMENT TRANSDUCERS

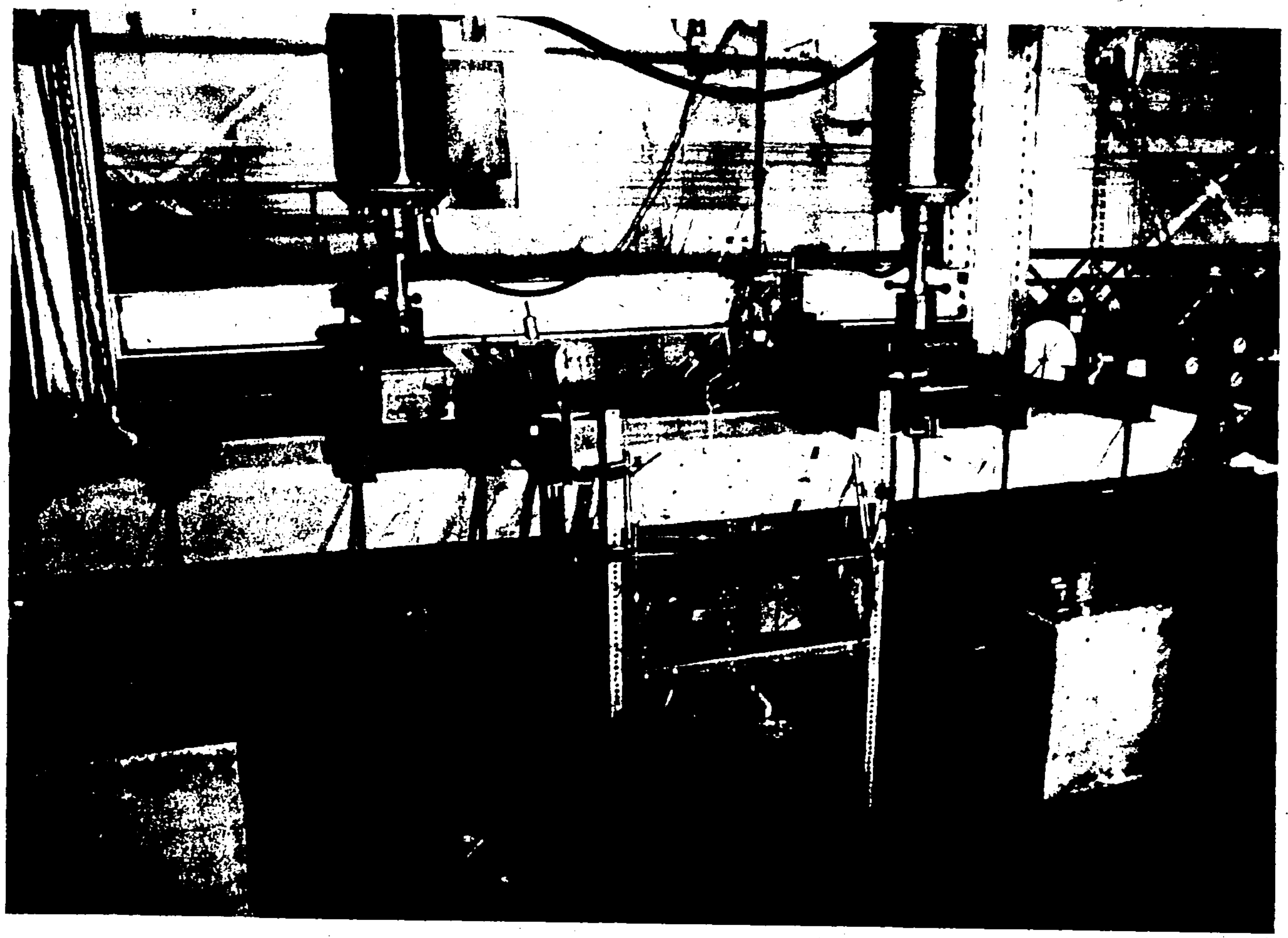


PLATE 4.3 COMPLETE EXPERIMENTAL TESTING SETUP

CHAPTER 5

EXPERIMENTAL RESULTS

5.1 Format of Presentation of Results

The complete spectrum of experimental results for the seven beams tested is presented in this chapter. The two distinct aspects of the testing program, test measurements and observed behaviour, are treated separately in the following Sections 5.2 and 5.3.

In the measurement of beam response, greater emphasis was placed on the recording of member deformation than reinforcement stress levels. For each beam, graphs of torque versus differential rotation and bending moment versus central deflection describe complete deformation characteristics. In the only representation of monitored stresses, stress levels of the longitudinal and transverse conventional reinforcement at the central cross-section are presented in the respective plots. Beam strength is tabulated fully in the specification of prestress levels, elastic stiffness, cracking and failure loads.

Experimental observations are confined to modes of failure and distinctive cracking patterns.

In the concluding section of this chapter, the potential sources of experimental error and irregularities in the experimental program are identified and evaluated.

A complete detailed tabulation of experimental results is provided in Appendix I.

5.2 Test Measurements

The recorded numerical data in its processed form is presented under the three categories of beam strength, beam stiffness, and reinforcement stress.

The beam strength components of initial level of prestress, final level of prestress, cracking load, and loading combination at failure are summarized in Table 5.1. At both the cracking and failure values the bending moment specified is the total bending moment arising from the torsion arm and Amsler jack loads.

To provide a suitable basis of comparison with analytical model results, the load-deformation relationships for each test specimen have been plotted for the complete loading range. Beam response in the pre-cracked state is elastic to within a reasonable degree of accuracy, and the respective torsion and bending stiffness constants for the seven beams are given in Table 5.2. Beyond cracking, beam behaviour is highly inelastic as exhibited in Figures 5.1 to 5.5. The torque versus rotation curve for beam R4 is not presented as it closely retraces the appropriate segment of the curve for beam R3. In Figures 5.1, 5.2, and 5.3, the abscissa is the differential rotation over the central 30 inch length of the beam. The deflection ordinate of the moment-deflection curves is the vertical displacement of the central cross-section due to bending action only. Thus, in all tests where torque was present, the initial deflection measurements had to be appropriately adjusted to reflect pure bending behaviour.

In monitoring reinforcement stress levels, strain gages were attached only to the longitudinal and transverse conventional reinforcement at the beam's central cross-section. In most cases, the bottom and top longitudinal reinforcement stress values are averages of three readings, whereas the hoop stress levels are derived from single gage measurements. In the test setup, the beams were aligned longitudinally close to a north-south direction. Thus, differentiation of the beam's east and west sides in elevation was established. The respective stress plots are illustrated in Figures 5.6 to 5.12, with identifying curve nomenclature given in Table 5.3. All stress levels are tensile with the exception of those of the top longitudinal bars.

5.3 Observed Behaviour

In almost all cases, beams were not tested to complete destruction, but were loaded just beyond their observed maximum load carrying capacity. Upon approaching the load capacity of all beams, primary cracks widened markedly, especially in those tests where there was a high ratio of bending moment to torque. Thus, failure appeared to be precipitated by excessive yielding of conventional reinforcement, yielding being most pronounced in the bottom tension flange. In the testing of beam R1 to destruction, failure occurred upon the crushing of a compression diagonal across the width of the top flange. As indicated by gaping crack widths, the strands had undergone considerable inelastic strain, but complete disintegration of the beam was prevented by the ductility of the bottom prestress strands. Close to the failure of beam R4, the test in which the highest ratio of bending moment to torque was applied, a similar mode of failure was observed, but well defined splitting cracks at the level of the bottom longitudinal reinforcement were also apparent, as shown in Plate 5.1. The only beam failure that differed from this general mode of failure was that of beam T1. Excessive local crushing and a punching shear effect became apparent beneath one torsion arm as displayed in Plate 5.2. Initiation of the local failure was primarily due to the absence of the plaster of paris pad between beam and torsion arm, thus resulting in extremely severe load stress concentrations. However, the load at failure was close to the predicted level.

Representative cracking patterns for two of the beams are illustrated in Fig. 5.13. Beams R4 and R5 were selected as they represented the extremes of highest and lowest ratio of bending moment to torque respectively. The numbering adjacent to the crack paths in Fig. 5.13 corresponds to the torsion load at which the particular crack was formed. The suffix B designates a bending load. In the testing of beam R4, the ratio of bending moment to torque at failure was close to five, and thus the resultant cracking pattern is much as expected. At the other end of the testing spectrum, the ratio of bending moment to torque for beam R5 was slightly greater than one. Consequently, the formation of parallel compression diagonals is well defined. Generally, most primary cracks were 7 to 8 inches apart, with secondary cracks more closely spaced at 2 to 4 inches.

5.4 Potential Sources of Anomalies

In the casting of test specimens, dimensional tolerances are inevitably introduced, and their severity must be accounted for in the estimation of accuracy of strength and deformation predictions. The most significant source of potential dimensional inaccuracy unique to the seven beams cast was the presence of the styrofoam voids. Any substantial movement of the styrofoam blocks during casting would have introduced dramatic variation in the thickness of the thin concrete flanges and webs. Presence of the closed torsion hoop reinforcement was used to advantage to secure the position of the voids, and post-test examination revealed that little variation in wall thickness was apparent.

The nature of the beam supports was such that a uniform St. Venant torque could be accommodated along any length of the beam. However, the presence of the 18 inch long solid beam ends beyond the supports offered longitudinal warping restraint to the small out-of-plane warping displacements that were generated at the beam ends during torsion tests. As a result, the beam length along which deformation measurements were taken was centrally located such that the influence of the solid ends would have diminished to a negligible level.

For the sake of expediency in ease of handling and positioning, the hoops and bars were lightly tack-welded during construction of the reinforcement cages. The welding was sufficiently light such that no brittle joints would be formed. The rigidity thus introduced into the cages had no effect on the pre-cracked beam stiffness, and contributed little to the post-cracked stiffness. Of greater significance concerning the reinforcement cage design was the location of the top longitudinal bars one inch in from the hoop corners. This feature was necessary to facilitate casting. Although Collins and Lampert[25] state that the positions of the corner longitudinal bars define cross-section geometry of the cracked concrete beam subjected to torsion, the movement of the top corner bars did not have a marked effect on the torsion failure loads for the seven beams tested. This was primarily due to the ratio of bending moment to torque remaining sufficiently high such that the uncracked state of the top flange was preserved up to failure in most instances.

The presence of the clamping bolts that secured the torsion arms, Amsler point loading and beam support apparatus did restrain the propagation of cracking beyond the central test section in each beam. Individual bolts were 3/4" in diameter and effectively acted as oversized stirrups.

In the lower range of elastic behaviour where beam deformations were small, the linear displacement transducers did not yield consistently accurate results in the calculation of beam twist. This inaccuracy arose partly through the method of attaching to the concrete the smooth metal plates on which the transducer needles impinged, and partly because the measuring instrument was being used at the lower limit of its range of accuracy.

		Beams						
		R1	R2	R3	R4	R5	T1	T2
	Initial Prestress	74.4	75.8	72	72.9	71.6	65.8	65.73
	Final Prestress	63.4	64.1	61.1	62.9	63.7	56	56.2
At Cracking	Torque	200	100	140	140	270	130	75
	Bending Moment	460	660	760	820	330	300	300
	Shear	0.0	0.0	10.0	10.0	0.0	0.0	0.0
At Failure	Torque	520	372	532	336	614	290	196.5
	Bending Moment	1080	1291	1357	1624	640	598	801
	Shear	0.0	0.0	11.0	18	0.0	0.0	0.0

TABLE 5.1 BEAM STRENGTH

Bending Stiffness*	4485.5	3560	4950	6340	2280	2531	2092
Torque Stiffness**	13.76×10^6	-	-	-	5.184×10^6	4.91×10^6	-

TABLE 5.2 PRE-CRACKED BEAM STIFFNESS

* Units of Kips (inch K/inch)
** Units of inch Kips/radian/in.

⊙———⊙ ●	Experimental Bottom Longitudinal Steel Model Bottom Longitudinal Steel
⚠———⚠ ▲	Experimental* Top Longitudinal Steel Model* Top Longitudinal Steel
⊡———⊡ ■	Experimental West Hoop Leg Model West Hoop Leg
◇———◇ ◆	Experimental East Hoop Leg Model East Hoop Leg
▽———▽ ▼	Experimental Bottom Hoop Leg Model Bottom Hoop Leg

* Note: In Figures 5.6 and 5.12, the designation for the ordinates of the experimental top longitudinal steel curves is the symbol ▲

TABLE 5.3 NOMENCLATURE FOR CONVENTIONAL REINFORCEMENT STRESS-LOAD CURVES

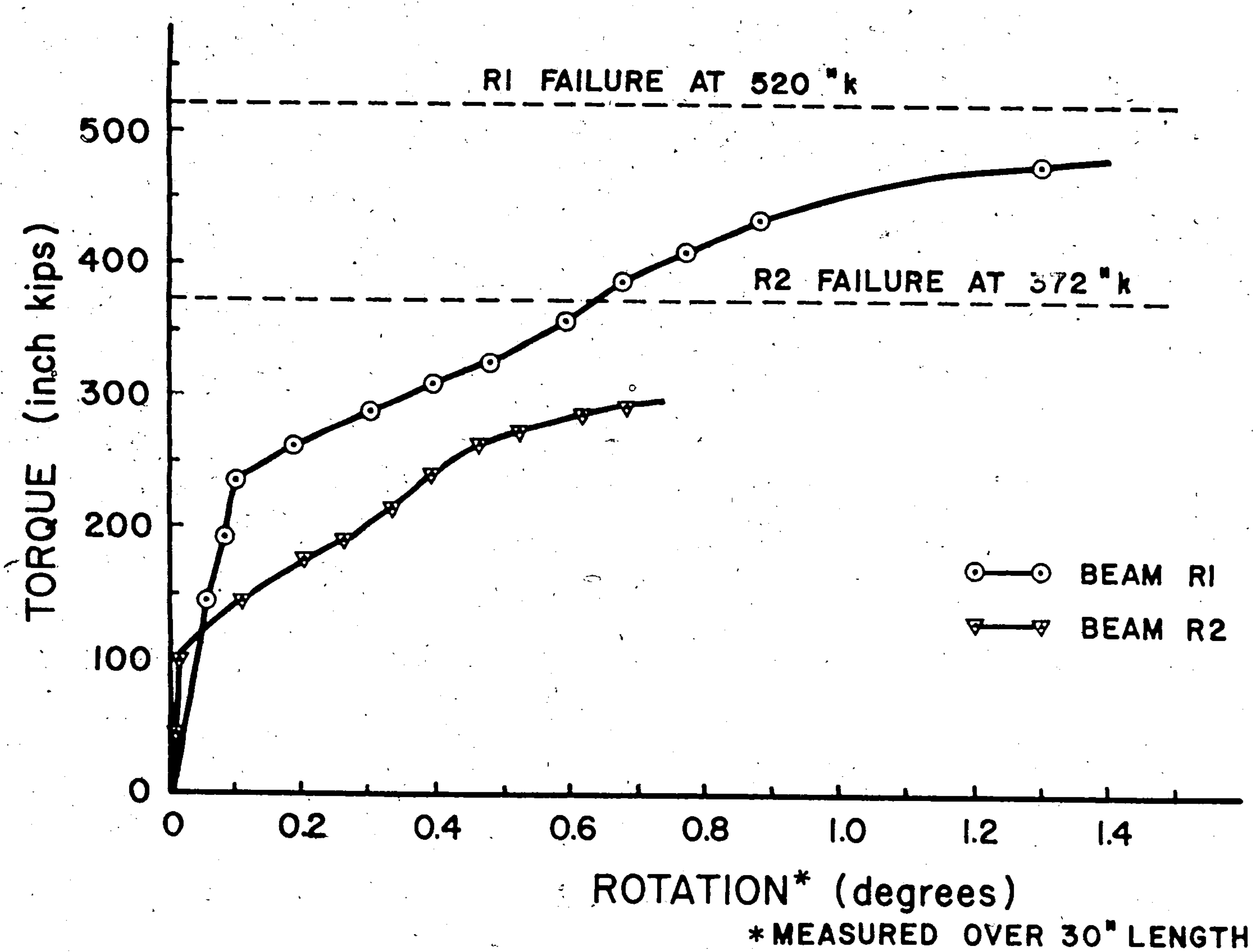


FIG. 5.1 TORQUE-ROTATION CURVES FOR BEAMS R1 AND R2

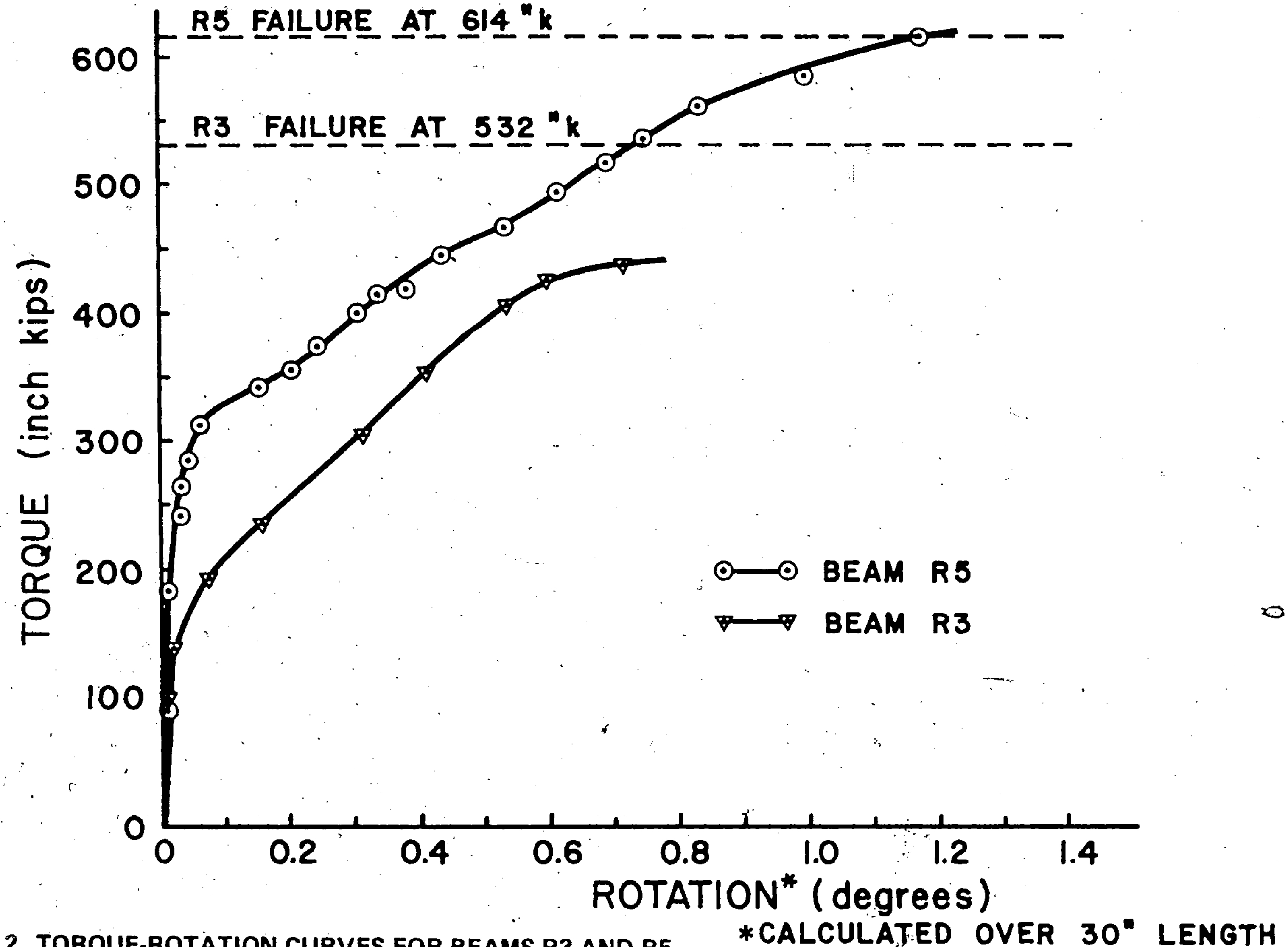


FIG. 5.2 TORQUE-ROTATION CURVES FOR BEAMS R3 AND R5

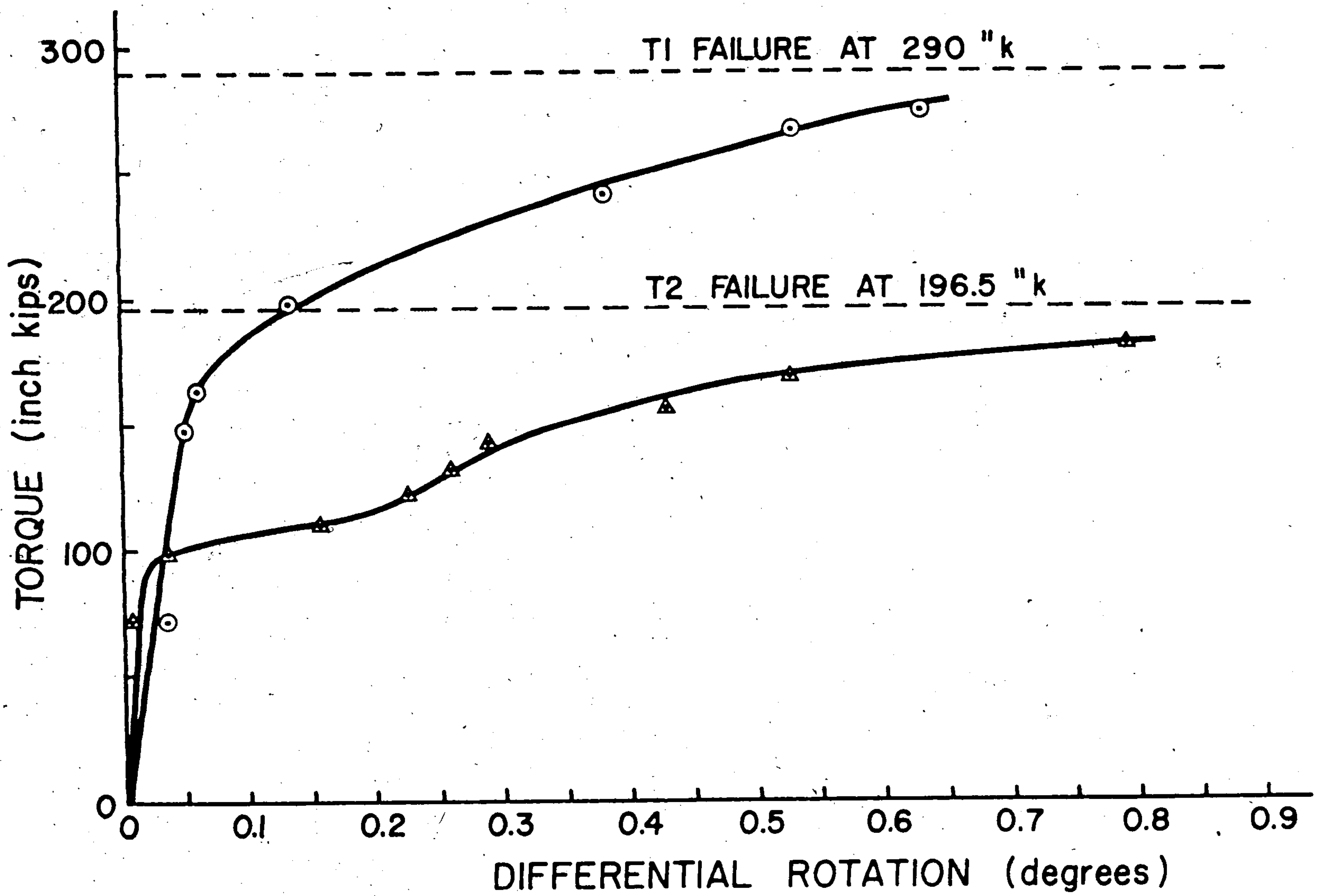


FIG. 5.3 TORQUE-ROTATION CURVES FOR BEAMS T1 AND T2

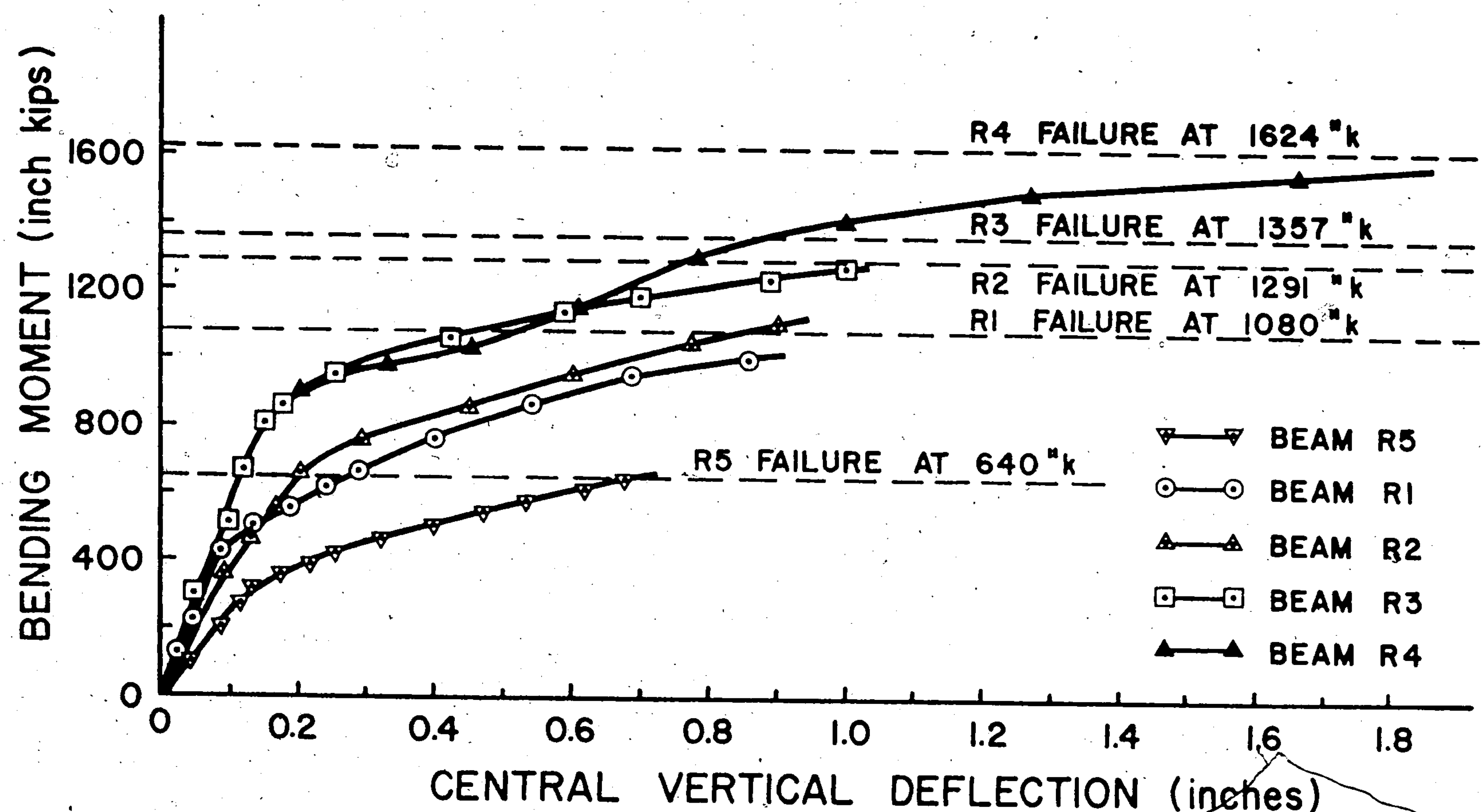


FIG. 5.4 MOMENT DEFLECTION CURVES FOR BEAMS R1, R2, R3, R4, R5

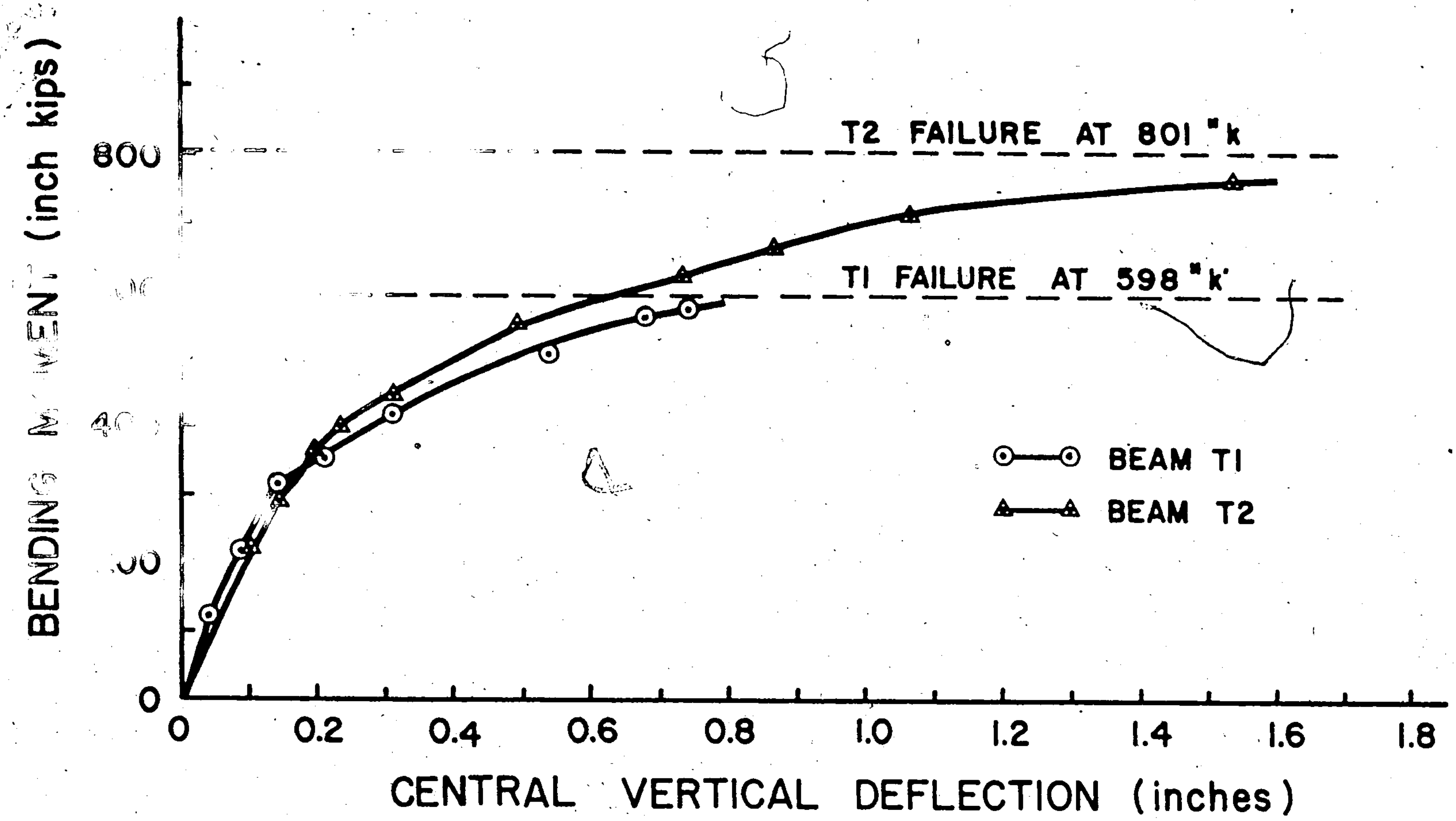


FIG. 5.5 MOMENT DEFLECTION CURVES FOR BEAMS T1 AND T2

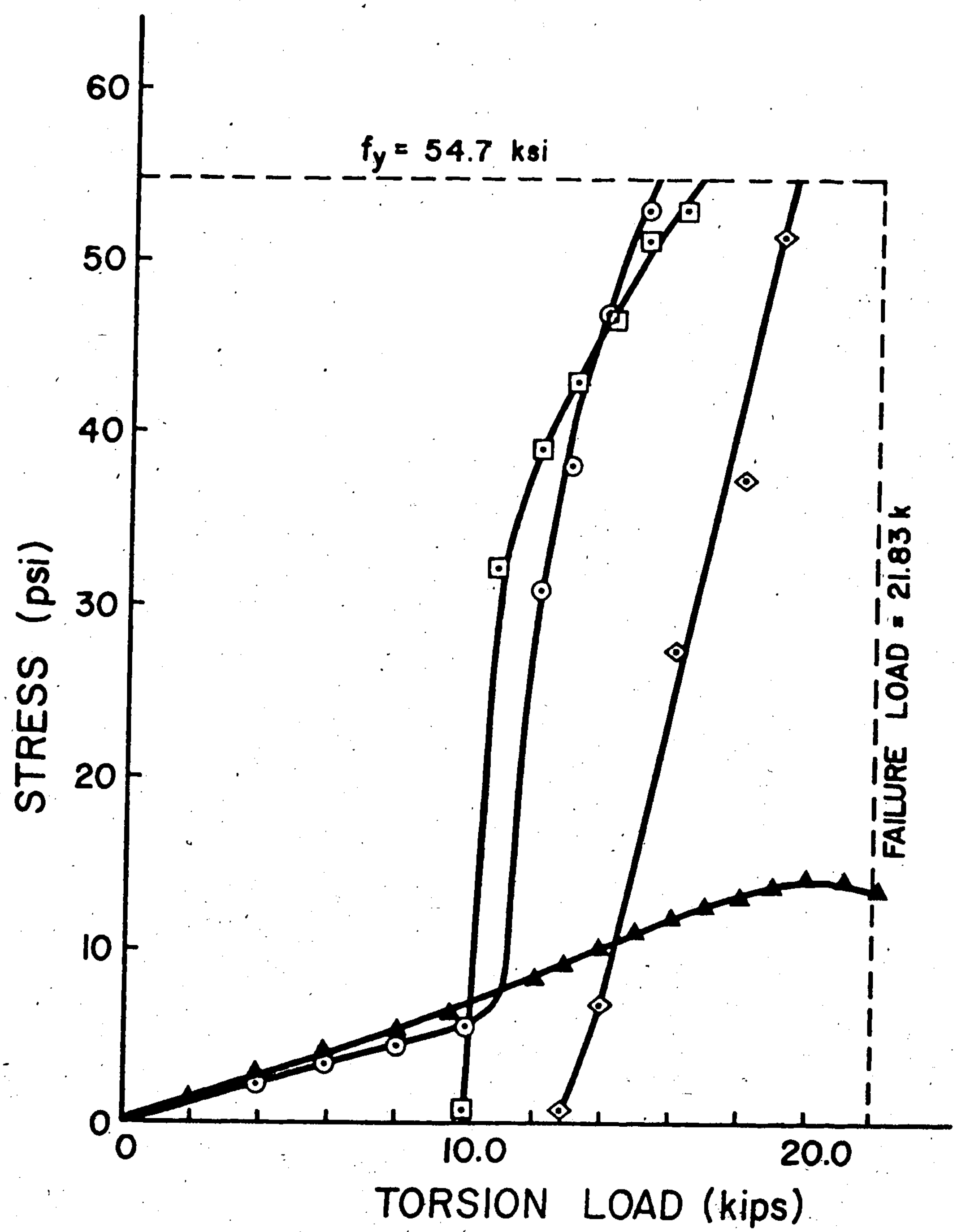


FIG. 5.6 STRESS-LOAD CURVES FOR BEAM R1 REINFORCEMENT

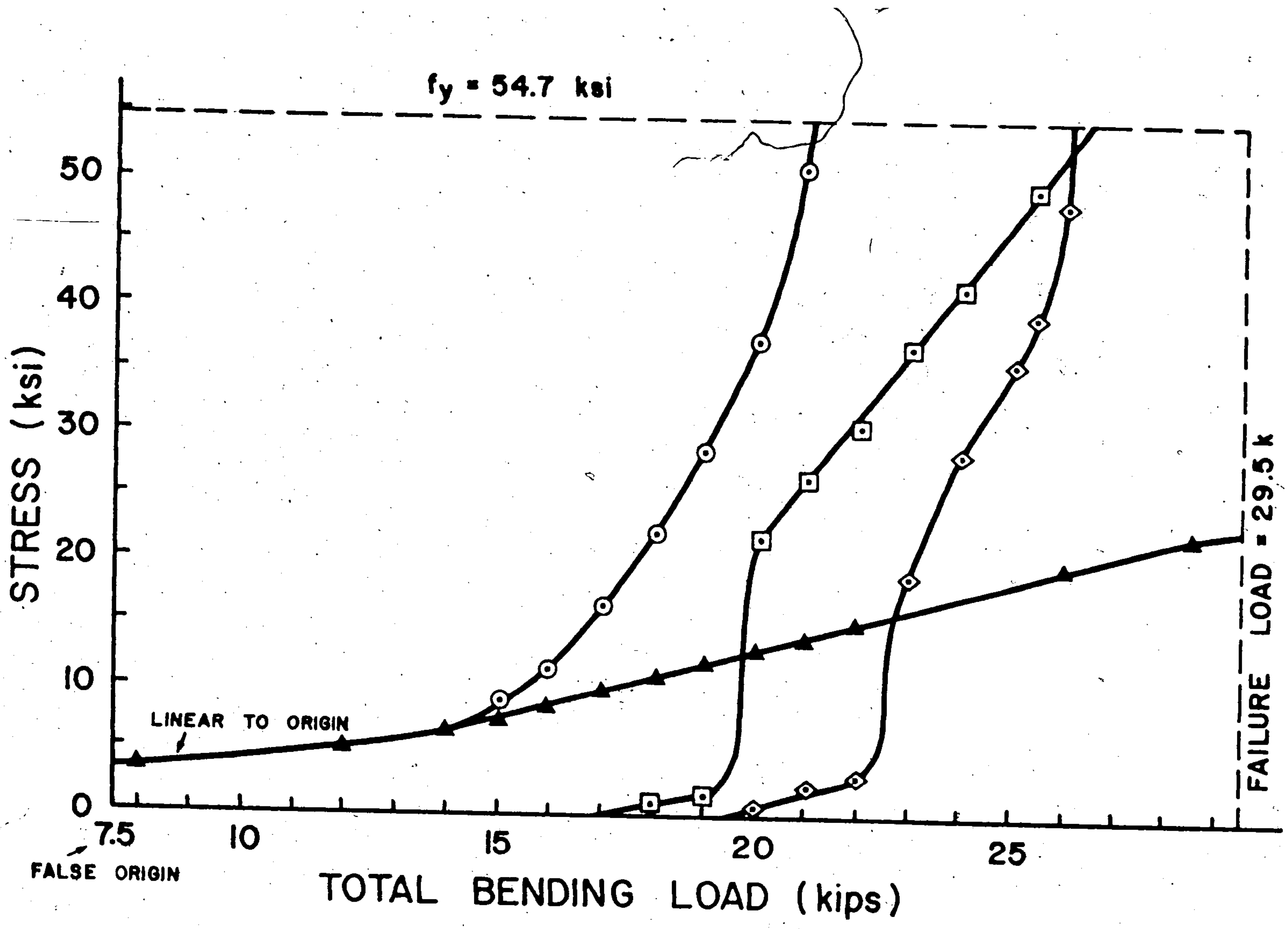


FIG. 5.7 STRESS-LOAD CURVES FOR BEAM R2 REINFORCEMENT

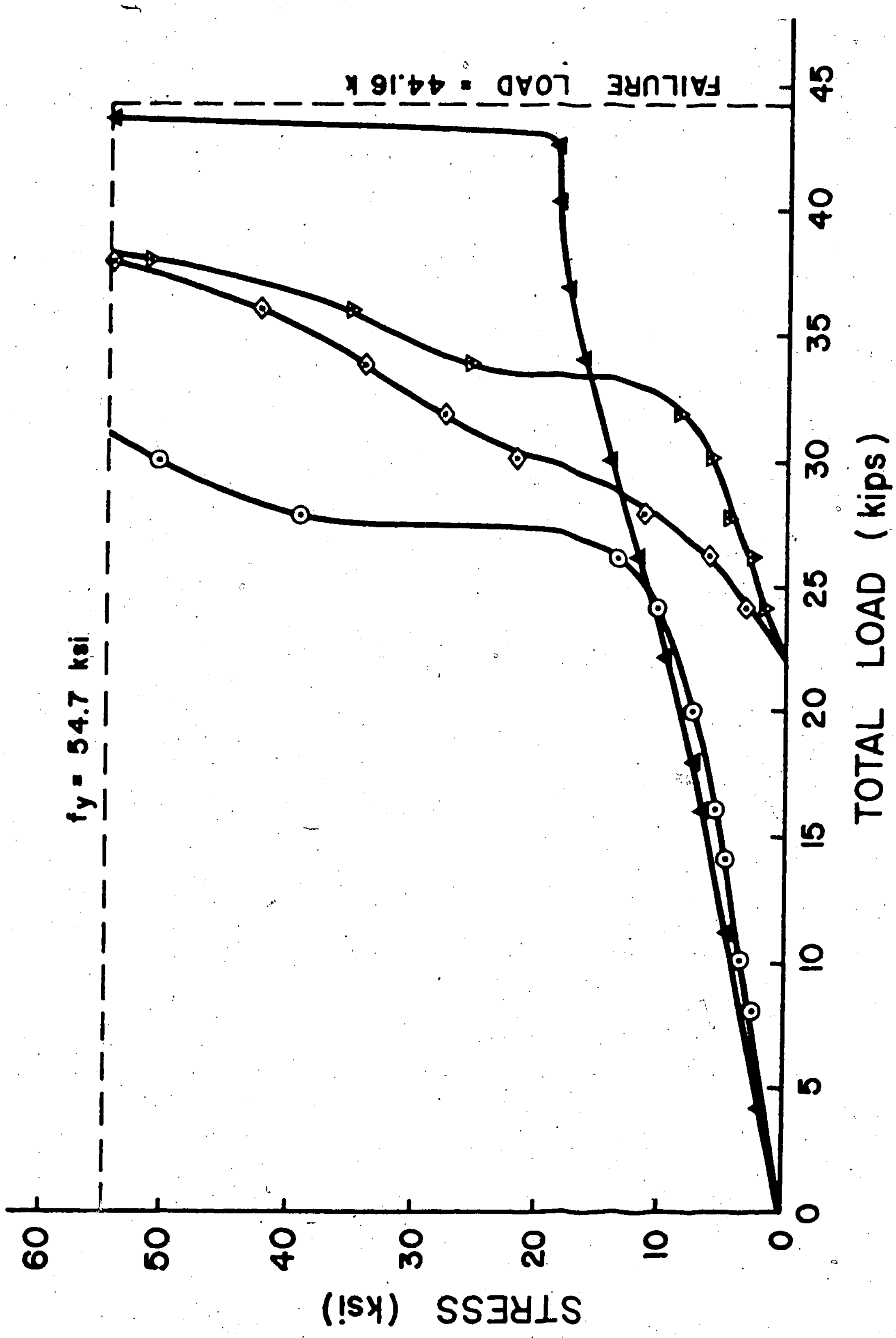


FIG. 5.8 STRESS-LOAD CURVES FOR BEAM R3 REINFORCEMENT

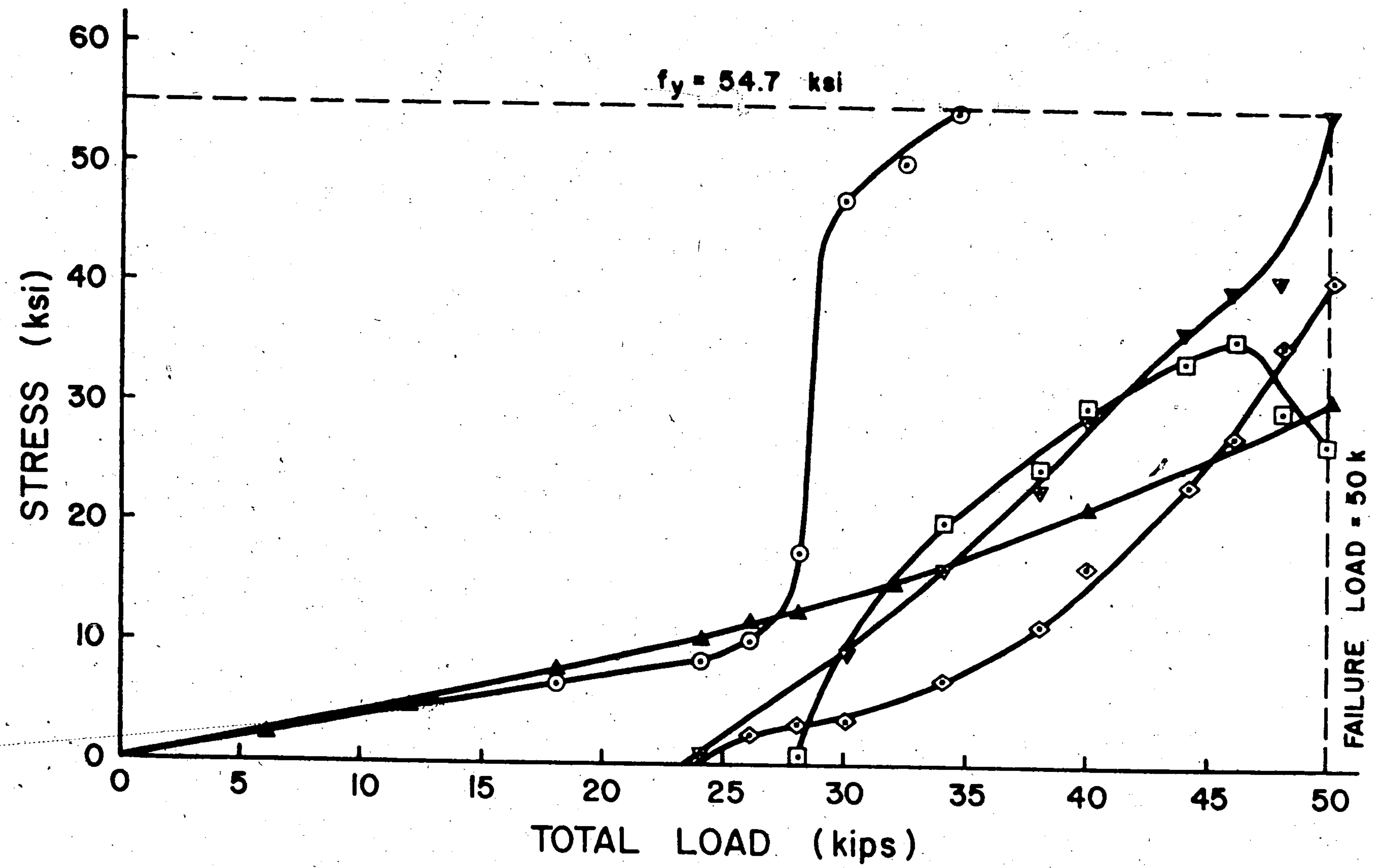


FIG. 5.9 STRESS-LOAD CURVES FOR BEAM R4 REINFORCEMENT

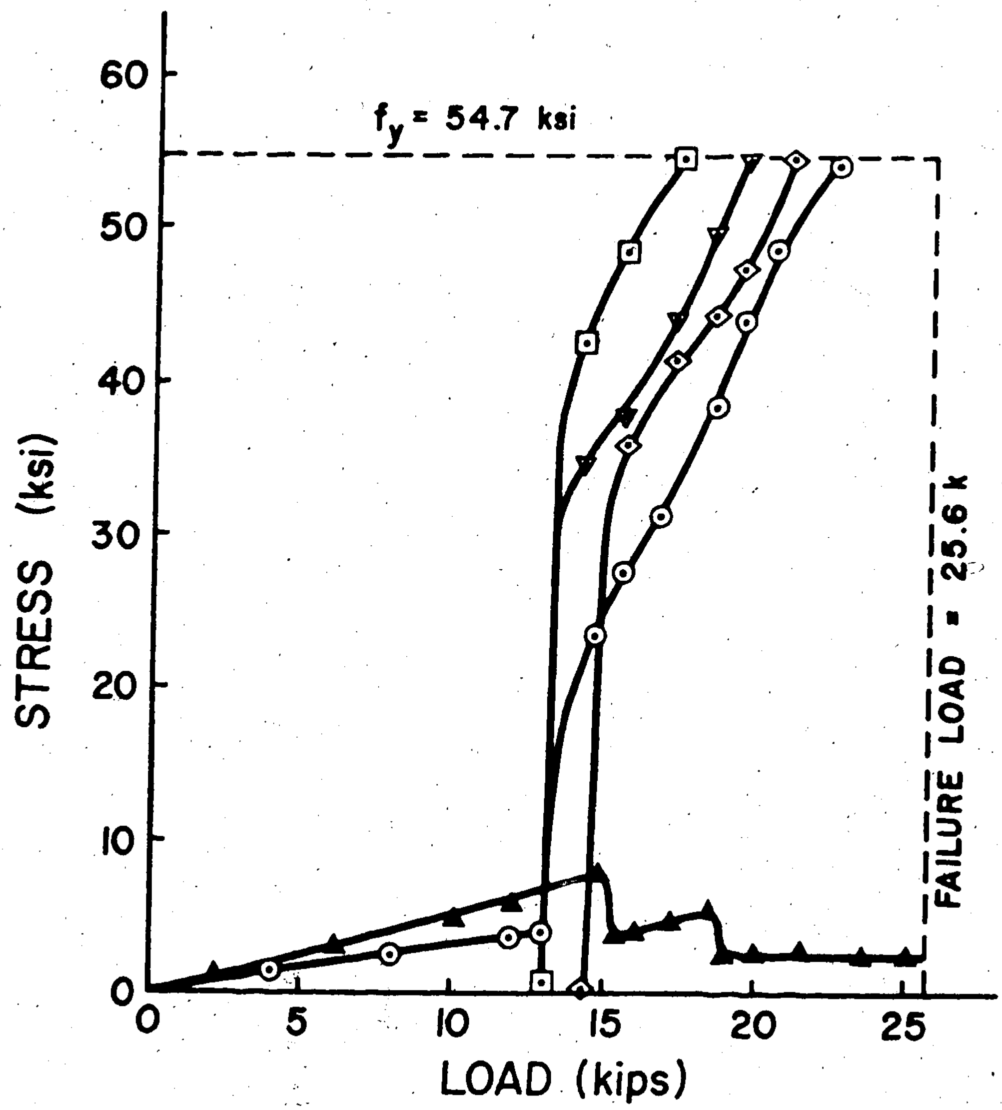


FIG. 5.10 STRESS-LOAD CURVES FOR BEAM R5 REINFORCEMENT

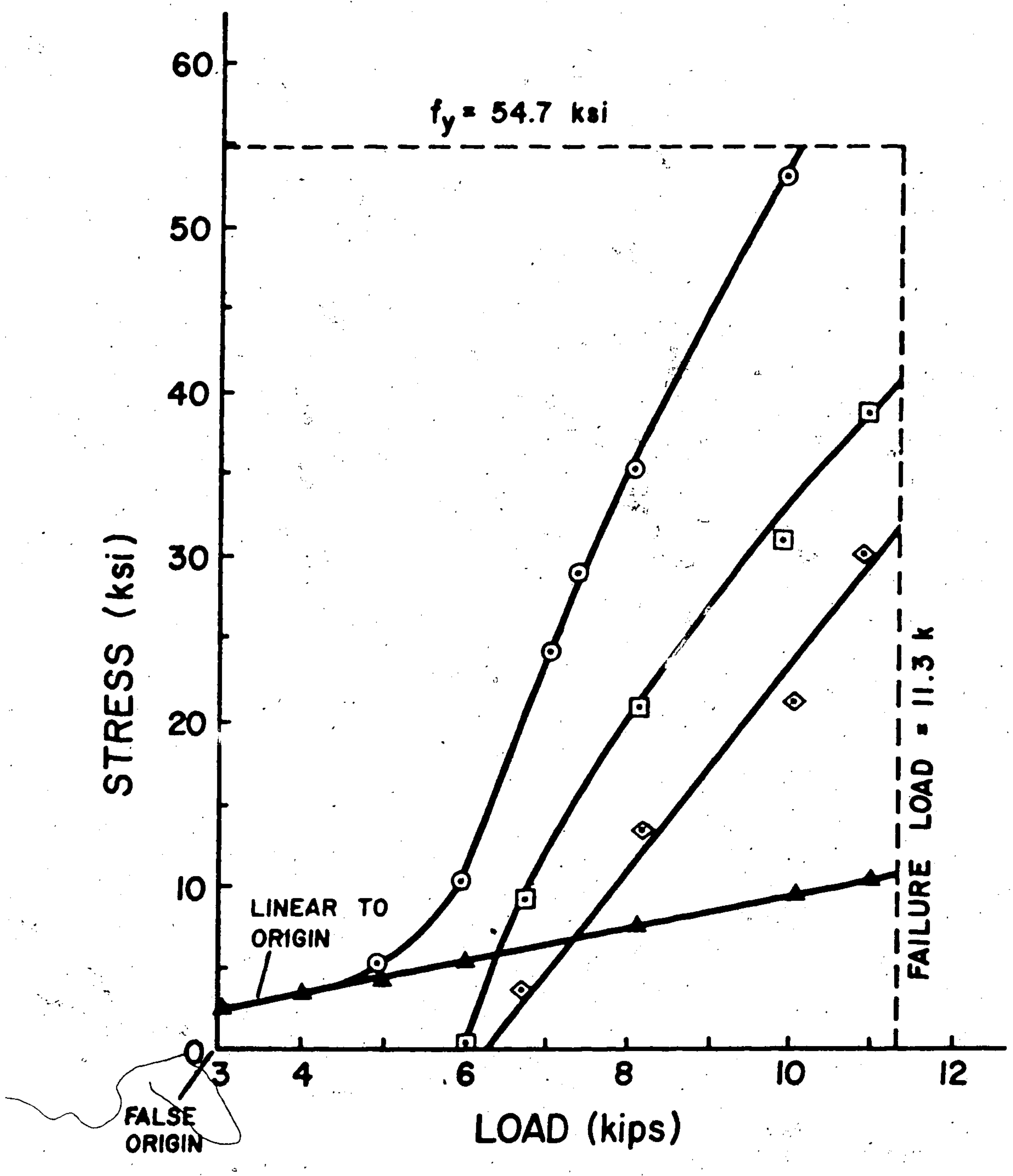


FIG. 5.11 STRESS-LOAD CURVES FOR BEAM T1 REINFORCEMENT

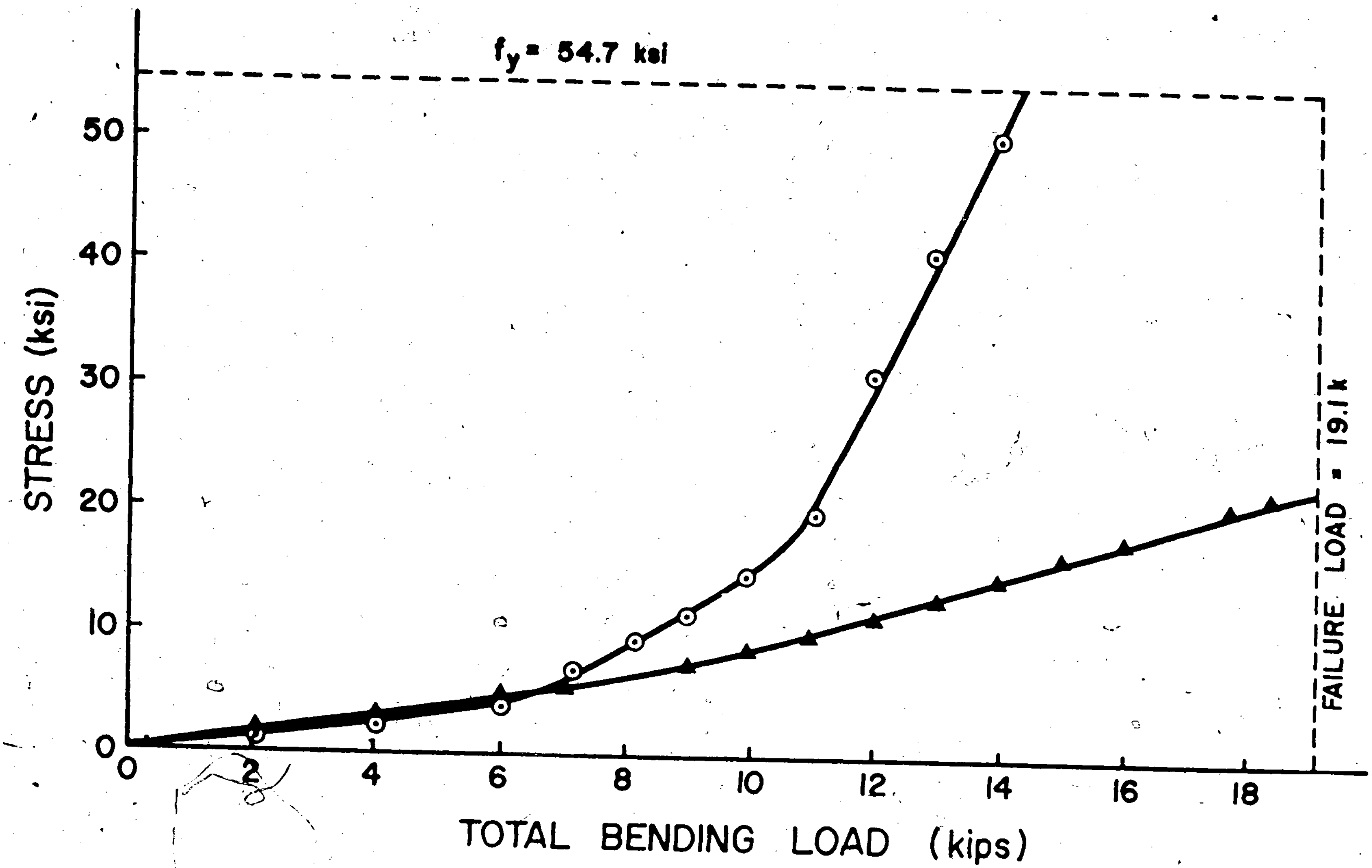


FIG. 5.12 STRESS-LOAD CURVES FOR BEAM T2 REINFORCEMENT

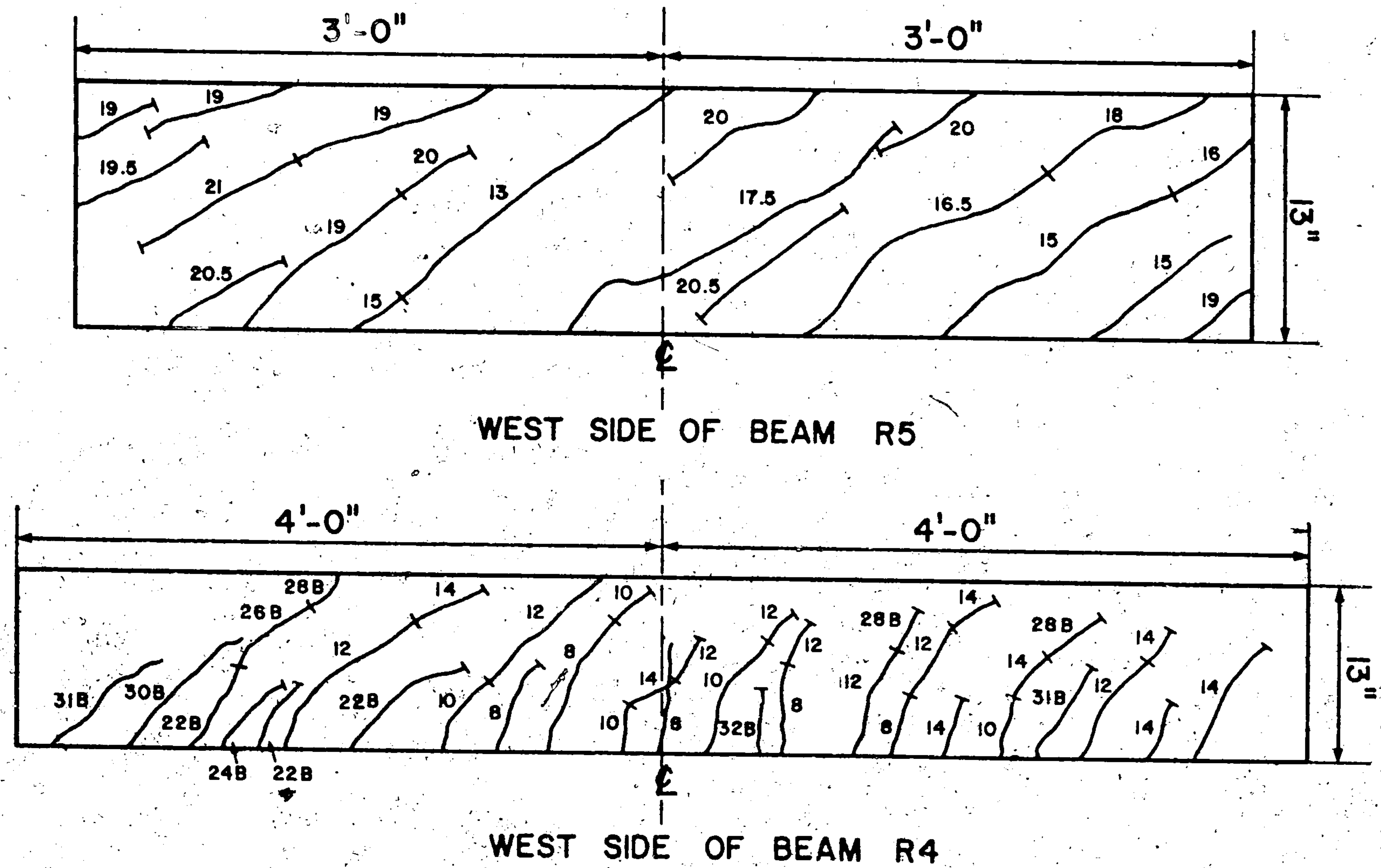


FIG. 5.13 CRACKING PATTERNS FOR BEAMS R4 AND R5

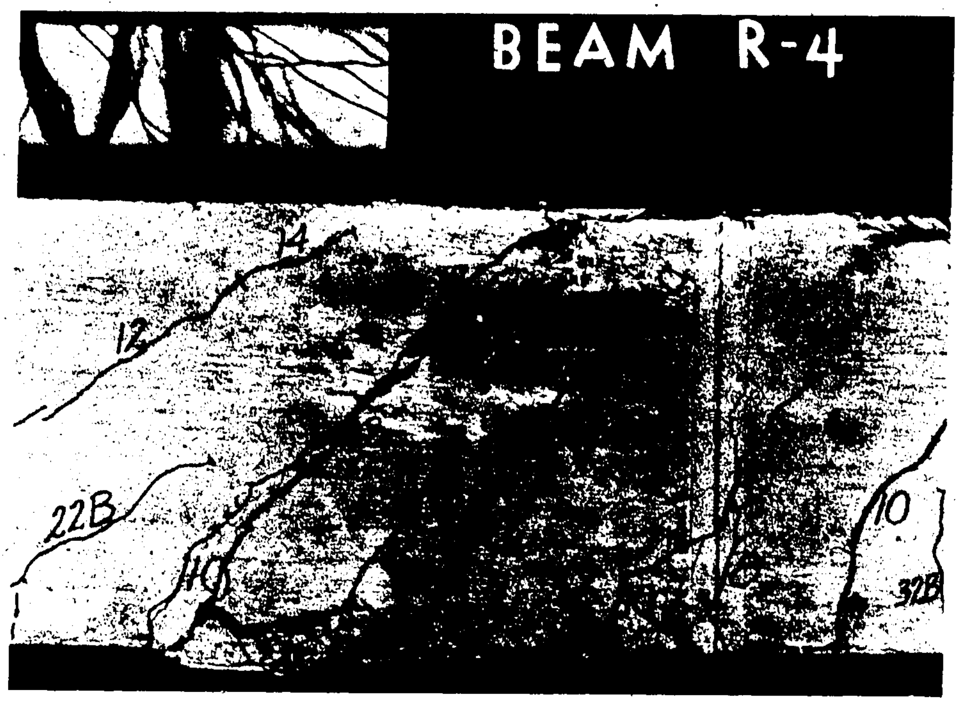


PLATE 5.1 FAILURE MODE OF BEAM R4

PLATE 5.2 LOCAL FAILURE OF BEAM T1

CHAPTER VI

EVALUATION OF COMPUTER MODEL RESULTS

6.1 Introduction

Motivation to develop this finite element computer model arose through the need for a flexible, precise analytical method of analysis of concrete box girders subjected to a general load condition. The principles of the mechanics incorporated in the computer model have been described in detail in Chapter 3 but performance of the assembled model was not addressed. This chapter focuses on the evaluation of the computer model results through comparison with experimental tests and related current theory.

Initially, those aspects that strongly influence the computer model response and do not represent common finite element modelling considerations, are examined to establish an insight into model behavioural characteristics revealed in the subsequent comparisons. The subject of comparison is the spectrum in behaviour of the seven prestressed concrete box girders that were tested in the experimental program described in Chapter 4. Following the presentation of the experimental test results and the corresponding computer model results in Section 6.3, theoretical estimates of ultimate beam strength and interactive response are in turn compared with the analytical model predictions in Section 6.4. Performance of the finite element model is assessed on the basis of the two comparative presentations, and is pursued in Section 6.5.

6.2 Extraordinary Influences on Computer Model Response

In assessing the performance of the computer model, consideration must be given to the following important aspects that both individually and collectively have a strong bearing on the model results. Several aspects are unique in the application of the analytical model to simulate the response of the seven prestressed concrete box beams tested in the experimental program. All aspects are of either a material or structural nature.

6.2.1 Material Behaviour Aspects

In simulating the stiffness of an uncracked reinforced concrete beam, the material property of paramount concern is the initial elastic modulus for the concrete. Since the stiffness contribution of the reinforcement in an underreinforced concrete beam is small compared with that of the concrete, an inaccurate determination of the initial concrete modulus will result in a correspondingly inaccurate prediction of beam deformations. Since concrete cyclinder tests did not yield consistent results and empirical formulae were not sufficiently accurate, the initial deflection measurements of the test beams were used in precisely calculating the respective initial concrete moduli.

A significant discrepancy was observed between the average crack spacing value derived using Eq. 3.29 and the corresponding experimental value determined from the beam cracking patterns at failure. Since aggregate interlock stiffness is a function of crack width and thus of average crack spacing, adoption of the considerably smaller theoretical value would have resulted in an overestimation of post-cracking concrete stiffness.

Reconstitution of the stiffness of a reinforced concrete element beyond cracking is difficult in that the contribution of "dowel action" cannot be determined accurately. Research has not conclusively established the quantitative effect upon dowel stiffness of several layers of web reinforcement or the inclination of the crack to the reinforcement axis other than at ninety degrees. However, consequences of the lack of complete understanding and definition of this phenomenon are not of major significance since dowel action accounts for less than 20 percent[3] of the shear strength of a cracked reinforced concrete beam.

6.2.2. Structural Aspects

Although both conventional and prestress longitudinal reinforcement are represented in the analytical model by one dimensional bar finite elements, the same method cannot be extended to the modelling of stirrup and hoop reinforcement in this computer model. This restriction has arisen through characterizing the concrete finite element behaviour by the stress-strain condition at the centroid of the element. When the principal tensile stress at the concrete element centroid exceeds the specified tensile strength of concrete, the element is designated as a cracked element. However, the element is not fractured by several parallel cracks distributed across the element, but by one crack passing through the centroid. Thus, when the vertical web reinforcement is concentrated into bars located at the vertical boundaries of an element whose aspect ratio exceeds unity, it is highly probable that the sole centroidal crack will not intersect the side boundaries of the element. Consequently, the cracked element cannot utilize its web reinforcement, and a premature shear failure will occur. To resolve this modelling difficulty, the web reinforcement is distributed throughout the element as a uni-directional steel mesh of closely spaced bars. When cracking occurs, the complete mesh stiffness for the concrete element is engaged. Thus, modelling of the mechanics of stirrup and hoop reinforcement in the cracked concrete condition closely reflects real behaviour. Figures 6.1(a) and 6.1(b) illustrate the problem and solution of this vitally important aspect of analytical modelling.

In Section 3.2.1, an expression was derived for the variation in shear stress across a "thick" wall as a function of the uniform shear flow, in the form:

$$\frac{\Delta v}{v} = \frac{A_c}{A'} \qquad (3.12)$$

Clearly, for those tubular members whose height and width dimensions are only moderately large compared with the wall thickness, the maximum shear stress at the wall surface can be considerably larger than that at the wall mid-thickness; ie. the uniform shear flow. Considering that plane stress finite elements do not permit shear stress variation across their thickness, and that such elements are located at the plates' mid-thickness, only uniform shear flows can be adequately represented. The computer model's insensitivity to shear stress variation is of concern since surface cracking in concrete will immediately propagate through the entire thickness of a tubular member's wall. Thus, if the torsional shear stresses are high at the critical cross-section of a concrete box girder, the computer model will yield underconservative estimates of cracking load and to a lesser extent post-cracking member stiffness.

A further possible consequence of the location of plane stress finite elements at wall mid-thickness is the inaccurate prediction of the ultimate bending moment capacity of an underreinforced concrete box girder. In the analytical model, the lever arm length from the force resultant in the concrete compression flange to the tension reinforcement at ultimate load conditions is fixed by geometry. If the compression flange is thick with respect to the girder's depth, the concrete compressive force resultant will be located between the wall mid-thickness and the upper flange surface, the height of the resultant force above the compression flange mid-thickness constituting the difference between the experimental and model lever arms. Also, since the thick concrete compression flange of the computer model is too stiff close to failure, the tension reinforcement will be forced to carry a disproportionately higher load, further reducing the ultimate bending moment capacity of the girder below its actual strength.

Before a reinforced concrete beam is tested, the concrete and reinforcement are already stressed through shrinkage and creep member deformations, the effects of these phenomena being most pronounced in precast prestressed concrete beams. Failure to include the presence of the tensile concrete shrinkage stresses can result in significant overestimation of the cracking load, and a slight distortion of pre-cracked deformations.

Although the longitudinal warping restraint of beam end diaphragms is not normally of real structural significance, the modelling of the seven prestressed box beams tested in the experimental program was complicated by the presence of the 18 inch long solid beam ends extending beyond each beam support. Since the warping resistance of the solid ends was comparable to that of the box beams, accurate representation of their influence on beam behaviour was essential.

Consequently, the approach outlined in Section 3.4.2.7 was adopted, and the warping stiffnesses established in the auxiliary finite element program were read in as input in the principal analytical model.

As shown in Figures 4.7, 4.8, and 4.9, clamping bolts were used to maintain the position of testing equipment at several locations along the length of the test specimens. Verified by observations during testing, the clamping bolts and the radial torsion loading arms effectively acted as very stiff stirrups, greatly restricting the propagation of web cracking in their vicinity. Thus, their presence in the analytical model is essential from both deformation and strength aspects. In a similar manner to that adopted for conventional stirrup and hoop reinforcement, equivalent steel meshes were included in adjacent concrete elements to represent their influence.

In each of the seven test beams, the solid cross-section infringed upon the test span, the distance from the support to the commencement of the styrofoam voids varying from 7.5 to 13.5 inches (Fig. 4.4). Since the stress levels at the beam ends were not sufficient to produce cracking, the primary modelling concern was deformational accuracy, especially with respect to bending deflections. Deformation characteristics were modelled closely through the use of plane stress finite elements of an appropriate, uniform thickness such that the moment of inertia remained unchanged.

In the casting of prestressed concrete beams, small flexural cracks will often appear in the top flange after transfer. Upon the application of positive moment, these cracks will immediately close, and the subsequent beam response will not reflect their initial presence.

Provision is made within the model to reproduce such behaviour. If a model crack closes in the load increments immediately following transfer, the full concrete constitutive matrix is recovered. Should a crack close at a later stage in the loading sequence, the aggregate interlock stiffness does not increase, but is maintained at the level prior to crack closing.

6.3 Comparison of Computer Model and Experimental Results

In this Section, the computer model and experimental results for the seven prestressed box girders are presented in a form that facilitates a thorough comparison. Assessment of the analytical model performance on the basis of this comparison is treated in Section 6.5.1.

Since the complete spectrum of beam response from initial loading to failure is to be examined, plots of torque versus rotation, bending moment versus central beam deflection, and reinforcement stresses versus load have been prepared for every test beam, each plot displaying the corresponding model and experimental coordinates. Monitoring of reinforcement stresses embraces the behaviour of the top and bottom flange conventional reinforcement at the beam span centreline and the four legs of the centrally-located hoop. The legend for identification of the reinforcement stress plots is provided in Table 5.3.

The twenty figures displaying comparative beam plots, Figures 6.2 to 6.21, are not completely comprehensive as two plots have not been presented. Reference to the Figure List indicates that the torque versus rotation curve for beam R4 and the hoop reinforcement stresses versus load graphs for beam T2 are the two absent plots. For beam

R4, its torque-rotation curve closely follows the respective plot for beam R3 since the loading of both beams only differed significantly beyond the scope of the torsion plot of beam R4. During the test of beam T2, the central hoop strain gages exhibited erratic readings which prejudiced their experimental value. In the torque-rotation graph of the experimental results of beam T2, shown in Fig. 6.20, the initial coordinates are not plotted as they displayed considerable inconsistency. The two computer model curves for the bending moment-deflection response of beam R4 represent the two loading extremes of the heavy Amsler jack load applied at the beam centreline. If the beam failure mechanism comprised a centreline hinge in the compression flange, and the loading plate was sufficiently stiff to bridge the hinge, the loading pattern of Fig. 6.22(b) is feasible, in contrast to that illustrated in Fig. 6.22(a) where complete contact is assumed.

Two computer model graph characteristics exhibited in the majority of bending moment-deflection and torque-rotation plots require clarification. For those beams where failure of the finite element model occurred within a load increment, the model strength results are described by a band delineating the lower and upper load bounds. The second point of clarification is the significance of the "translated" computer model curve shown in those figures where there is considerable discrepancy between experimental and model cracking loads. If the computer model behaviour was adjusted such that its cracking load agreed closely with the corresponding experimental load, the projected analytical model results are represented by the "translated" computer model curve. Most importantly, the translation procedure does not entail pre-cracking or post-cracking stiffness modification. Development of the translated curve is detailed in Section 6.5.1.

The cracking and ultimate strength combined loading conditions for the experimental and the corresponding computer model beams are presented in Table 6.1. Invariably, the deformation value at the point of ultimate strength was indeterminable from both the experimental and computer model results. Consequently, there is no objective method of comparing experimental and computer model postcracking stiffnesses, and thus a subjective evaluation of the corresponding curves is the best recourse.

6.4 Comparison of Computer Model Results with Current Theory

Of the vast number of research publications in the field of torsion, bending, and shear in reinforced concrete, reference has been made to Collins and Lampert[25] in estimating pure ultimate torsional strength, Thürliman[50] in the evaluation of bending moment-shear interaction and pure shear strength, and Elfgren[51] concerning torsion-bending and torsion-bending-shear interaction. Since current theory cannot predict post-cracking member deformations under combined loading, only ultimate strength predictions will be considered. Evaluation of cracking loads will not be treated as this area of research has been exhaustively examined in the past, and resolved to a satisfactory degree. Similar to the previous Section, this Section will only present a comparison of theoretical and model predictions, with the assessment and implications of the comparison addressed in Section 6.5.2. Since geometry, concrete strength, and reinforcement levels are almost identical within the two test beam classifications, average ultimate strengths will be presented for the rectangular and trapezoidal beam categories.

6.4.1 Ultimate Strength

In the application of the Space Truss Theory proposed by Collins and Lampert[25] to estimate the ultimate torsional capacity of a prestressed concrete box girder, the form of their equations is not modified to account for initial prestress. The prestress strand is simply considered as reinforcement of strength $f_s^* A_s^*$, where f_s^* is the yield stress of the prestress steel and A_s^* the area of the individual strand. For both the rectangular and trapezoidal beams, the weaker top reinforcement determines the ultimate torsional capacity of the two beam classifications. When considering the trapezoidal beams, the web prestress strand was distributed to the top and bottom flange stringers such that the ultimate bending moment capacity remained unchanged. The ultimate torsional capacity is given by

$$To = 2Ao\sqrt{\frac{\Sigma Z_y}{u} \cdot \frac{S_y}{t}} \qquad (6.1)$$

where Ao = area enclosed by corner longitudinal bars, ΣZ_y = twice the sum of yield forces of longitudinal bars in the weaker flange, S_y = hoop yield force, u = corner longitudinal bar perimeter, and t = hoop spacing. The average ultimate torsional capacity of the five rectangular beams using the Space Truss Theory is 379 inch kips, and the corresponding strength for the two trapezoidal beams is 239 inch kips.

Since the Space Truss Theory cannot accurately estimate the compression zone depth, and Skew Bending Theory cannot accommodate the presence of longitudinal web reinforcement or longitudinal reinforcement of different yield strengths, classical bending the y[52] as currently incorporated in prestressed concrete design has been used to evaluate the ultimate bending moment capacity of the two beam types. For the five rectangular beams, the average ultimate bending moment capacity is 1515 inch kips, and 985 inch kips for the two trapezoidal beams.

Calculation of the pure shear strength of the two box beam types follows the approach advocated by Thürliman[50]. On the basis of a simplified, generalized space truss model, Thürliman devised the following expression for the "plastic shear force" V_{po}:

$$V_{po} = \sqrt{2 F_{yl} \cdot S_y \cdot \frac{h}{t}} \qquad (6.2)$$

where F_{yl} = yield force of bottom flange longitudinal stringers, S_y = yield force of stirrups at cross-section considered, h = ultimate bending moment lever arm, or, in the absence of an accurate estimation of this value, the centre to centre distance between the top and bottom stringers, and t = stirrup spacing.

However, since both the transverse and longitudinal reinforcement must yield simultaneously before failure for the above derivation to be valid, the variable angle of inclination of the compression concrete diagonals must lie within the range:

$$.5 \leq \tan \alpha_s \leq 2 \qquad (6.3)$$

where α_s = angle of inclination of compression struts to horizontal.

For such an inclination range, the maximum shear resistance of a concrete beam is modified to

$$V_{pmax} = \sqrt{\frac{2}{\kappa}} \cdot V_{po} \qquad (6.4)$$

where V_{pmax} = maximum shear resistance, and

$$\kappa = \frac{F_{y1} \cdot t}{S_y \cdot h} \tag{6.5}$$

In the application of the above equations to prestressed concrete (Thürliman's equations were developed for reinforced concrete), the axial force N in the equation below is not zero, and corresponds to the initial prestress force.

$$F_{y1} = \frac{M}{h} + \frac{N}{2} + \frac{V}{2} \cdot \cot\alpha_s \tag{6.6}$$

The form of the developed equations remains unaltered if the term F_{y1} is replaced by the term $(F_{y1} + \frac{Nh'}{h})$ where N is the initial prestress force, and h' is the distance from the prestress strand to the concrete force resultant. Mostly, h' equals h. Therefore, Eq. 6.6 is valid in its original form if F_{y1} is defined as the total bottom stringer yield strength including the initial prestress force.

For the rectangular box beams, the plastic shear force is 70.7 kips and the maximum shear resistance is 49.5 kips. Similarly, for the trapezoidal box beams, the plastic shear force is 57.4 kips and the maximum shear resistance is 40.5 kips.

Since a computer model loading combination could not be devised that produced a pure shear loading similar to that considered in the preceding theory, an analytical estimate of the ultimate shear capacity could not be determined.

The experimental and corresponding computer model ultimate strengths ar summarized in Table 6.3.

6.4.2 Torsion - Bending Interaction

Of the seven box beams tested in the experimental program, five were subjected to torque and bending moment only, those beams being R1, R2, R5, T1, and T2.

The interaction equations of Elfgren[51], in an identical form to those developed by Collins and Lampert[25], are utilized to describe beam response under combined torque-bending moment load conditions. Under such combined loading, beam failure under the action of a positive bending moment can be either of two modes, mode t when failure is initiated by yielding of the bottom flange longitudinal reinforcement, or mode c when yielding of the top flange stringers precipitates beam fail

Mode t

$$\frac{M}{M_o} + \left(\frac{T}{T_o}\right)^2 r = 1 \qquad (6.7)$$

Mode c

$$\frac{M}{M_o}\left(\frac{-1}{r}\right) + \left(\frac{T}{T_o}\right)^2 = 1 \qquad (6.8)$$

where r = ratio of yield forces of top and bottom flange longitudinal reinforcement.

In determining the value of term r in the above two equations, the full yield strength of the prestress strand is used. For the two trapezoidal beams that contain web prestress strand reinforcement, distribution of the longitudinal web steel to the top and bottom flange stringers is such that the ultimate bending moment capacity of the beams remains unchanged. The yield force ratio "r" is .2377 and .448 for the rectangular and trapezoidal beam categories respectively.

As a basis of comparison, the interaction Equations 6.7 and 6.8 are plotted in Fig 6.23 for both the rectangular and trapezoidal beam types, together with the corresponding computer model results for the five beams that failed under torque and bending moment loading only. The computer model results are displayed in their dimensional and non-dimensional form in Table 6.3, all results being average values for the load increment in which failure of the analytical model occurred.

6.4.3 Torsion - Bending - Shear Interaction

Only two of the seven test beams were subjected to shear in addition to torque and bending moment at their critical cross-sections, the two beams being rectangular box beams R3 and R4.

The interaction equations adopted to define beam response under the combined loading of torque, bending moment, and shear are those proposed by Elfgren[52]. In conjunction with the two previously defined modes of failure the presence of shear introduces an additional mode of failure, mode s, where the compression zone is formed on the side of the beam. The corresponding interaction equations are as follows:

Mode t

$$\frac{M}{M_o} + \left(\frac{T}{T_o}\right)^2 r + \left(\frac{V}{V_o}\right)^2 r = 1 \qquad (6.9)$$

Mode c

$$\frac{M}{M_o}\left(\frac{-1}{r}\right) + \left(\frac{T}{T_o}\right)^2 + \left(\frac{V}{V_o}\right)^2 = 1 \qquad (6.10)$$

Mode s

$$\left(\frac{T}{T_o}\right)^2 \frac{2r}{r+1} + \left(\frac{V}{V_o}\right)^2 \frac{2r}{r+1} + \frac{TV}{T_oV_o} \cdot \frac{2r}{r+1} \cdot \frac{2}{\sqrt{1+b'/h'}} = 1 \qquad (6.11)$$

where b' = horizontal centre to centre distance of corner stringers in top or bottom flanges, and h' = vertical centre to centre distance of corner stringers in top and bottom flanges.

In conjunction with the above equations, Thürliman's interaction equations for bending moment and shear is also considered as an additional interactive constraint that must not be violated:

$$\frac{M_p}{M_{po}} + \left(\frac{V_p}{V_{po}}\right)^2 = 1 \qquad (6.12)$$

where M_p = applied moment, and M_{po} = "plastic moment" or ultimate bending moment capacity. From earlier discussion, the applied shear V_p must not exceed the maximum shear capacity V_{pmax} established by Eq. 6.4. Similarly, as a consequence of the inclination of the compression struts being restricted to $.5 \leq \tan \alpha_s \leq 2$, the maximum applied bending moment must satisfy the condition:

$$\frac{M_{pmax}}{M_{po}} + \frac{V_p}{V_{po}} \cdot \frac{1}{4}\sqrt{\frac{2}{K}} = 1 \qquad (6.13)$$

The interaction Eq. 6.12, together with the imposed limits of Equations 6.4 and 6.13, is shown in Fig. 6.24.

Using Eq. 6.2 to evaluate the plastic shear force V_o appearing in Equations 6.9, 6.10 and 6.11, the interaction equations for the three modes of failure of beams R3 and R4 are illustrated in Fig. 6.25. Only one set of interaction equations is shown as the respective equations for the two beams are very similar. All three interaction equations have been simplified through the evaluation of the $(\frac{V}{V_o})$ terms which are then transferred to the numerical right hand sides of their respective equations. Thus, three dimensional interaction is reduced to a two dimensional torque-bending moment interaction.

In Table 6.3 that displays the computer model results for beams R3 and R4, the bending moment failure loads have been adjusted to allow for the presence of the central downward vertical concentrated load illustrated in Fig. I.1(c) in Appendix I. Under such a loading system, Thürliman[50] has established that the principal design cross-section for shear is not located beneath the central load, but at a distance h either side of the concentrated load. Elfgren's[51] examination of the effect of a concentrated load yields a similar result. Consequently, the failure bending moments for beams R3 and R4 have been altered accordingly. The adjusted computer model results for the latter two beams are plotted in Fig. 6.25.

6.5 Assessment of Computer Model Results

6.5.1 Computer Model Assessment in Light of Experimentation

6.5.1.1 Prominent Aspects of Model's Performance: In the uncracked state, the computer model yielded an accurate assessment of beam stiffness since the initial modulus of elasticity for concrete was evaluated directly from beam deformations at low load levels. This procedure in determining the concrete modulus was adopted as cylinder tests conducted for the sole purpose of modulus measurement produced a wide scatter of results. In this study the need for estimating the modulus of concrete as accurately as possible is paramount as both deformation and strength characteristics are examined. The few instances of disparity in agreement between experimental and model behaviour in the pre-cracked condition appear in the torque-rotation relationship since the experimental deflection monitoring equipment was incapable of consistently accurate measurements of very small differential deflections of adjacent beam locations.

Although corresponding experimental and model elastic stiffnesses are in close agreement, a very discernible discrepancy is evident at the onset of cracking. For all beams where cracking occurred under a combined torque-bending moment load condition: i.e. all beams with the exception of R3 and R4; the model beam cracked at a considerably higher load than the experimental beam. The three potential sources of this deviation in behaviour are: concrete tensile strength, concrete shrinkage stresses, and variation in shear stresses across the wall thickness. Of the three potential sources, the effect of the variation in wall shear stresses is the most dominating influence on behavioural discrepancy. In Section 3.2.1, the variation in the St. Venant torsional shear stress from the wall surface to wall mid-thickness was shown to be equal to the ratio of the cross-sectional area to the area enclosed by the corner longitudinal reinforcement stringers. For the rectangular and trapezoidal beams, the ratio equals .61 and .87 respectively. Since the analytical model evaluates the torsional shear stresses at the wall mid-thickness, the maximum torsional shear stresses at the surface of the experimental beams are 61% and 87% higher than the corresponding model stresses for the rectangular and trapezoidal beams. This substantial discrepancy between mid-thickness and surface torsional shear stresses is critical as cracks in the outer surface fibres immediately propagate through the entire wall thickness. If the shear stress variation were taken into account in the calculation of the model concrete principal tensile stresses, the discrepancy in cracking loads would be substantially reduced. However, difficulty in accurately measuring the concrete tensile strength and uncertainty in establishing concrete shrinkage stresses can collectively contribute to inaccurate cracking estimates. Thus, although the variation in torsional shear stresses can be taken into account, accurate prediction of cracking loads may not be achieved consistently.

As a result of the model's higher cracking load, a response delay is exhibited in the bending moment-deflection, torque-rotation, and reinforcement stresses-load graphs. In those moment-deformation figures where the response delay is considerable, a "translated computer model curve" has been plotted in addition to the actual model curve to illustrate the projected computer model response if the cracking load discrepancy did not occur. To define the translated curve, the post-cracking portion of the actual model curve is moved horizontally to the right until the tangent projection of the inelastic curve below its local origin intersects the elastic slope at the experimental cracking load. The prophetic significance of the translation procedure illustrated in Fig. 6.1(b) is as follows. Should a concrete box beam be modelled where no account is taken of the variation of torsional shear stresses and the presence of shrinkage stresses, the analytical load-deformation results will be of the form of curve OBC in the latter figure, where a considerable discrepancy is apparent between experimental and model cracking loads. However, if both the two previously specified sources of error in cracking prediction are taken into account, the modified analytical results are defined by the curve OAB'C'. The stiffnesses at corresponding intermediary points along BC and B'C', points D and D' for example, correspond closely as the effects of shrinkage and torsional shear stress variation are minimal in comparison to the dramatic redistribution of stresses that follows cracking. In beam tests that exhibited considerable ductility beyond cracking, the inelastic slope up to moderate load levels exhibited little decay, and thus construction of the segment AB' is a reasonable approximation of the initial modified post-cracking response. The principal motivation in introducing the translated curves is that a closer subjective comparison can be made of experimental and model post-cracking stiffnesses, and projected member deformations are predicted more accurately.

With few exceptions model post-cracking stiffness closely follows experimental behaviour, and deformation predictions of the translated model curves are consistently accurate before entering the highly inelastic deformation regions close to ultimate failure. The less pronounced effect of the torsional shear stress variation after beam cracking arises since redistribution of forces through concrete cracking and reinforcement yielding produces large stress variations that greatly exceed the influence of surface to mid-thickness shear stress variation.

At ultimate failure load conditions, the computer model bending moments are consistently 8% to 10% below the corresponding experimental moments for those beams that were subjected to a high ratio of bending moment to torque. This modelling inaccuracy is largely due to the method of delineation of the beam cross-section in the finite element model.

In representing a concrete wall by a two-dimensional plane stress finite element located at the wall mid-thickness, the moment of inertia of the uncracked beam is accurately represented and actual reinforcement locations can usually be accommodated. However, the thickness of the concrete compression zone and the location of the compressive force resultant are fixed, effectively predefining the model bending moment lever arm independently of reinforcement levels and critical beam cross-sectional parameters. For both the rectangular and trapezoidal finite element meshes, the bending moment lever arms are 4.5% shorter than theoretical estimates at ultimate load conditions, resulting in almost a 5% reduction in the ultimate bending moment capacity. The higher stress levels in the longitudinal tension reinforcement of the computer model have little effect on post-cracking stiffness except in the region close to failure where model stiffness is invariably less than experimentation indicated.

Several less significant sources of error are introduced through selection of the geometry of the finite element mesh. Defining cross-sectional geometry by mid-thickness dimensions dictates the location of all reinforcement. However, imposed reinforcement bar movements do not exceed the magnitude of their respective diameters, thus limiting error to a minimal degree. Of a more difficult nature to evaluate objectively, the fineness of a finite element mesh can influence convergence and modelling accuracy.

The mesh size was selected in this instance as a compromise between realistic accuracy expectation and computer execution costs.

6.5.1.2 <u>Review of Individual Beam Results</u>: Two areas of discrepancy in the comparison of model and experimental behaviour are consistent for all beams. Model bending moments at failure underestimate experimental values, thus affecting the corresponding torques in a similar manner. At low load levels when beam deformations are small, the experimental differential rotation measurements are not reliable. In the following examination of Figures 6.2 to 6.21, the legend of graph coordinates for the reinforcement stresses-load plots is given in Table 5.3.

<u>Beam R1</u>: The relevant plotted relationships are illustrated in Figures 6.2, 6.3 and 6.4.

Since the ratio of torque to bending moment is moderate (.5) throughout the loading sequence, the model cracking load is significantly higher than the corresponding experimental value as anticipated in the discussion of Section 6.5.1.1. The "translated" computer model curves illustrated in Figures 6.2 and 6.3 show a reasonably close post-cracking stiffness correspondence, the exception being in the highly inelastic region of the torque-rotation relationship. During the experimental testing of beam R1, the central beam length over which the differential rotation was calculated, was further removed from the influence of the substantial stirrup-like torsion load arms than in the computer model. Consequently, average model differential rotation measurements are significantly smaller than their experimental counterparts at high load levels when the restraint of the torsion arms on crack widening is most pronounced. Difference in cracking load predictions is also exhibited in the reinforcement stress plots of Figures 6.4(a) and 6.4(b).

<u>Beam R2</u>: The relevant plotted relationships are illustrated in Figures 6.5, 6.6, and 6.7.

As the bending moment-deflection ratio at failure is higher for beam R2 than R1, the degree to which the model ultimate strength predictions underestimate beam capacity is more pronounced. Correspondence of the elastic and post-cracking stiffnesses is accurate in both the bending moment and torque graphs, with modest divergence occurring close to failure. The only significant difference in the reinforcement stress plots is that the eastern leg of the central hoop did not yield in the model beam, but the discrepancy is not significant.

<u>Beam R3</u>: The relevant plotted relationships are illustrated in Figures 6.8, 6.9, and 6.10.

For beam R3, the disproportionate underestimation of the ultimate torsional capacity compared with that of the ultimate bending moment capacity is a direct result of the loading sequence. In the load increments preceding failure, the ratio of bending moment to torque was equal to 1.06. Consequently, premature failure of the test specimen had an exaggerated influence on the accuracy of the ultimate torsional estimate. Accurate prediction of the model cracking load is expected as the beam was subjected to bending moment only at the formation of initial cracks. The reinforcement stress plots also reflect the close estimate of the cracking load.

<u>Beam R4</u>: The relevant plotted relationships are illustrated in Figures 6.11 and 6.12.

During testing, an unexpected increase in beam stiffness occurred beyond the applied bending moment of 1100 inch kips. In retrospect, it seems that the nature of application of the central load altered, reducing the central bending moment as illustrated in Fig. 6.22. In Fig. 6.11, the "adjusted" experimental curve represents the bending moment-deflection curve based on the assumption that the central loading plate ceased to maintain uniform contact with the concrete beam at centre span, effectively maintaining only edge contact as shown in Fig. 6.22(b). The smooth stiffness change, absence of a marked post-cracking stiffness increase, and a more realistic ultimate bending moment capacity suggest that the adjusted curve is a more reasonable representation of the actual experimental behaviour. Correspondence in post-cracking stiffness between the computer model and adjusted experimental curve is close. The disagreement between the corresponding east hoop stress curves in Fig. 6.12(b) is the result of the hoop strain gage being located above its assumed mid-depth position.

Beam R5: The relevant plotted relationships are illustrated in Figures 6.13, 6.14, and 6.15.

As illustrated in Table 4.2, the initial concrete modulus for beam R5 was considerably below the average modulus of the other six beams, despite the comparable concrete strengths obtained from cylinder tests. The low concrete modulus was undoubtedly due to inadequate vibration in the narrow web walls and bottom flange, whereas the top flange, being completely exposed, was compacted sufficiently. Thus, the modulus value in Table 4.2 slightly over-estimates the web and bottom flange stiffness, and substantially underestimates the top flange stiffness. Consequently, the post-cracking stiffness of the analytical model is less than the corresponding experimental stiffness in the bending moment-deflection relationship where the top flange stiffness is critical, and greater in the torque-rotation relationship where web and bottom flange stiffness is influential. The top longitudinal steel strain gage was malfunctioning during the experimental test.

<u>Beam T1</u>: The relevant plotted relationships are illustrated in Figures 6.16, 6.17, and 6.18.

In addition to the expected prediction of a higher cracking load, the only region of significant deviation is the torque-rotation curve segment close to failure. During the latter stages of the experimental test, localized crushing commenced on a top flange corner directly beneath a torsion loading arm. The crushing was confined to the corner only, but the trapezoidal beam's torsional stiffness was undoubedly diminished, thus contributing to the increasingly inelastic response of the experimental beam.

<u>Beam T2</u>: The relevant plotted relationships are illustrated in Figures 6.19, 6.20, and 6.21.

Beyond 550 inch kips in the bending moment-deflection curve, considerable discrepancy is apparent that defies explanation. Deviation of model and experimental behaviour displayed in the torque-rotation graph is exaggerated since both curves are highly inelastic immediately prior to failure and the ultimate torque at failure is small in comparison to the corresponding bending moment. Experimental coordinates were not plotted in the latter figure below a torque of 109 inch kips as the results were inconsistent.

6.5.1.3 Summary: The computer model simulation of the seven prestressed box girders tested in the experimental program satisfactorily described member deformations and evaluated ultimate strengths to within an acceptable degree of accuracy. To assess the analytical model's performance in its proper perspective, two important sources of model behavioural inadequacy must be recognized. Firstly, the computer model consistently overestimates the cracking strength of "thick" walled concrete box girders that are subjected to any loading combination that includes torque. However, this shortcoming can be overcome as the actual cracking load can be calculated accurately, and a "translated computer model curve" can be drawn to illustrate model behaviour if model and experimental cracking loads coincide. Secondly, upon development of a finite element mesh, the ultimate bending moment lever arm is fixed by geometry, resulting in an inaccurate estimate of the ultimate bending moment capacity. am strength under combined loading that includes bending moment is influenced in a similar manner. No corrective measures can be made in the computer model to negate the influence of the latter anomaly.

6.5.2 Computer Model Assessment in Light of Current Theory

6.5.2.1 Ultimate Strength

Ultimate Torsional Strength: Calculation of the theoretical ultimate torsional capacity of both rectangular and trapezoidal beams has been made on the basis of the "compressive stress field theory" or space truss theory. The theory is based on a kinematic approach in the theory of plasticity where an external loading is sought at which a mechanism of deformations is formed. However, the formation of the mechanism implicitly assumes that both stirrups and longitudinal steel yield before failure. Although the space truss model has several other inherent limitations in its application, the requirement that reinforcement proportions be such that stirrups and stringers yield prior to failure is crucial in establishing the theory's validity in this instance.

Elfgren[51] reports that several researchers have confirmed that the yield prerequisite is met if the ratio of longitudinal to transverse reinforcement is such that

$$0.5 < \cot \alpha_T < 2.0 \tag{6.14}$$

where

$$\cot \alpha_T = \sqrt{\frac{2A_l \sigma_l^y}{b' + h'} \cdot \frac{t}{A_w \sigma_w^y}} \tag{6.15}$$

in which A_l = area of longitudinal reinforcement, σ_l^y = yield stress of longitudinal reinforcement, b' = width of beam measured between corner stringers, h' = beam depth measured from top to bottom stringer, t = stirrup spacing, A_w = stirrup area, and σ_w^y = stirrup yield stress.

For the five rectangular beams, cot α_T equals 1.96, and 2.05 for the two trapezoidal beams. The value of cot α_T for the trapezoidal beams is of questionable significance as Eq. 6.15 was developed for rectangular beams only.

On the basis of the theoretical verification, a valid comparison can be made of the computer model and theoretical ultimate torsional capacity estimates for the rectangular beams, and the respective values in Table 6.2 illustrate good agreement. However, Equations 6.14 and 6.15 indicate that a comparison of trapezoidal results is highly questionable and furthermore, experimental and computer model results verify that the stirrup reinforcement in both the trapezoidal beams was not yielding at failure.

The slightly higher computer model estimate for the rectangular beams is as anticipated, since only the yield force of the reinforcement is used in the theoretical calculation. Unlike the computer model, space truss theory does not take into account reinforcement strain hardening or dowel action. The additional reserve of reinforcement strength beyond yielding does not increase the ultimate torsional capacity substantially, however, as the transfer of load from concrete to steel increases reinforcement stresses dramatically as failure is approached.

Ultimate Bending Moment Strength: Both the space truss and computer models cannot accurately estimate ultimate bending moment capacities as their respective ultimate bending moment lever arms are fixed by geometry. Although not afflicted by the latter shortcoming, the skew bending model cannot accommodate longitudinal web bars and tension reinforcement of different yield strengths. Thus, the equivalent stress block theory incorporated in prestressed concrete design is the most accurate method of evaluating the ultimate bending moment capacity. The shortcoming of the computer model in this respect is treated in detail in Section 6.5.1.1.

Ultimate Shear Strength: The approach adopted by Thürliman in formulating the theoretical shear capacity of a reinforced or prestressed concrete beam is only applicable to beams underreinforced for shear. Definition of an underreinforced beam in the context of pure shear is identical to that for pure torsion, in that both the longitudinal stringers and the transverse reinforcement must yield at failure. Should either reinforcement type not yield at failure, the yield criteria are violated and the theory becomes invalid.

Unfortunately, a loading pattern could not be devised to enable the computer model to predict the maximum shear resistance of rectangular beams R3 and R4. The choice of loading pattern was restricted since the beam span could not be reduced without modifying behaviour, and concentrated loads could not be located close to the critical cross-section where shear failure was anticipated as their presence altered the local shear stress distribution[50,51]. Recognizing these two restrictions, the only feasible load combination produced a design region of maximum shear accompanied by a large bending moment.

6.5.2.3 Combined Loading Interaction: Under combined loading conditions, the correspondence between computer model and theoretical results is difficult to analyze objectively, expecially since the number of beam specimens is small and no noticeable trends of deviation are apparent. Strength comparisons for each individual load type have been made, and qualifications that arose through those comparisons are applicable to the combined loading evaluation.

Correspondence in the torque-bending moment interaction, illustrated in Fig. 6.23, is good, although it should be recognized that non-dimensional presentation of results can disguise fundamental differences in performance. For example, beams T1 and T2 are both overreinforced for torsion, but their interaction coordinates lie close to the corresponding theoretical underreinforced interaction curve.

Displayed in Fig. 6.25, the interaction equations for beams R3 and R4 do not significantly reflect the influence of the modest amount of shear present in both beams. Since the degree of error that is inevitably encountered in experimentation and computer modelling is of the same order of magnitude as the adjustment of the two interaction equations for the presence of shear, little can be deduced from the proximity of the computer model coordinates to the theoretical torque-bending moment-shear interaction curve illustrated in the latter figure. In the testing of beam R3, the ratio of torque to bending moment was close to unity in the load increments preceding failure. Thus, experimental and computer modelling error would have resulted in disproportionately large changes in the torque loading at failure.

The most serious reservation concerning interactive behaviour is the obscure definition of an underreinforced beam, especially when shear is present. Current provisions proposed to prevent premature failure by crushing of concrete are conservative in the absence of a more thorough understanding of load interaction. In contrast, the computer model is a generalized analytical tool whose applicability is not restricted by loading or reinforcement limitations.

6.5.2.3 <u>Limitations in Application of Theory</u>: The following limitations apply to the calculation of the ultimate torsional capacity of a reinforced concrete beam using space truss theory:

1. The beam must be underreinforced: ie. longitudinal and transverse reinforcement must yield prior to failure.
2. St. Venant torsion must be dominant.
3. Along the beam length, the cross-section must be uniform.
4. Dowel action of reinforcement is neglected.

The theory adopted to calculate the ultimate bending moment strength, as described in Section 6.4.1 and 6.5.2.1, is not subjected to limitation in its applicability.

Thürliman's approach in estimating the ultimate shear capacity of a reinforced or prestressed beam is only subject to two limitations:

1. The beam must be underreinforced for shear: ie. longitudinal and transverse reinforcement yield simultaneously at failure.

2. The top uncracked flange does not carry shear.

The interaction equations presented by Elfgren[5.1] are subject to the following qualifications in their application:

1. The beam cross-section must be consistent along its length.

2. Interaction equations for polygonal cross-sections are not verified.

3. Criteria for preventing premature failure through concrete crushing are not explicit.

The only limitation or qualification equally applicable to the computer model is that the uncracked top flange does not carry shear. One serious qualification not attributed to theory is the inaccurate length of the bending moment lever arm at failure. In conclusion, the flexibility of the computer model is characterized by several important areas of application that are beyond theoretical capabilities:

1. The member cross-section can change along its length through the use of a general quadrilateral concrete finite element.

2. Any reinforcement arrangement can be accommodated for both underreinforced and overreinforced conditions.

3. Member deformations are described comprehensively.

4. All stress-deformation information is available at any loading stage between cracking and failure.

5. Indeterminate structural analysis under any loading combination is possible.

6.5.2.4 Summary: In pure torsion, agreement between the computer model and theoretical results was satisfactory for the under-reinforced rectangular beams. However, since the two trapezoidal beams were overreinforced for torsion, a meaningful comparison could not be made. Analytical model estimates of the ultimate bending moment capacities of all beams were inaccurate since the bending moment lever arms, fixed by finite element mesh geometry, were incorrect.

Although difficult to assess objectively, torque-bending moment interaction corresponded closely. Such close correspondence was not apparent in the torque-bending moment-shear interactive behaviour of beams R3 and R4.

Objective assessment of the computer model results through comparison with the corresponding theoretical predictions was not conclusive as the theory is limited in its range of application, and complex interactive behaviour is presented theoretically in a general form that does not permit explicit comparison.

Beams	Bending Moment						Torque			Shear		
	Cracking			Ultimate			Ultimate			Ultimate		
	E	M	M/E	E	M	M/E	E	M	M/E	E	M	M/E
R1	460	570	1.24	1080	1015	.94	520	492	.95	0	0	-
R2	660	820	1.24	1291	1197	.93	372	336	.9	0	0	-
R3	760	760	1.0	1357**	1267 1307	.93 .96	532	432 470	.81 .88	11.0	11.0	1.0
R4*	820	820	1.0	1452**	1239 1312	.853 .9	336	336	1.0	12 13	18	.67 .72
R5	330	330	1.0	640	600 620	.94 .97	614	562 583	.92 .95	0	0	-
T1	300	350	1.17	598	570 595	.95 .99	290	276 288	.95 .99	0	0	-
T2	300	420	1.4	801	742 767	.93 .96	196.5	168 180	.85 .92	0	0	-

Notes: (1) E = Experimental result M = Computer model result
(2) All units are in inches and kips.
* The adjusted experimental values are given.
** These are centreline moments. The design moments at "h" from centreline are plotted in interaction diagrams.

TABLE 6.1 EXPERIMENTAL AND COMPUTER MODEL CRACKING AND ULTIMATE LOADING

Beams	Ultimate Bending Moment Capacity			Ultimate Torque Capacity			Ultimate Shear Capacity		
	Space Truss Estimate	Model Estimate	Model / Theory	Bending Theory Est.	Model Est.	Model / Theory	Shear Theory Est.	Model Est.	Model / Theory
R1, R2, R3 R4, R5	1515	1370	.905	379	390	1.029	49.5	-	-
T1, T2	985	915	.929	239	300	1.256	-	-	-

Note: All units are in inches and kips.

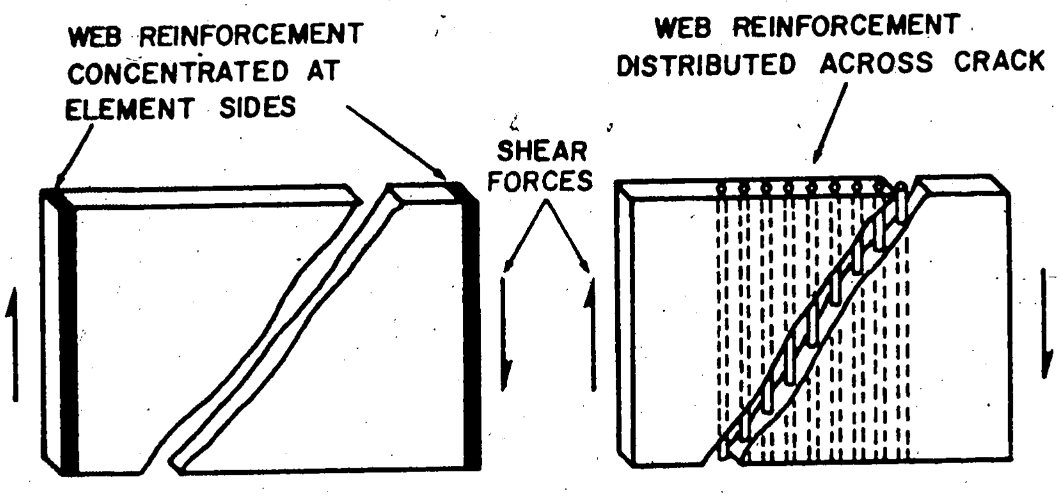
TABLE 6.2 MODEL AND THEORETICAL ULTIMATE STRENGTHS

		Beams						
		R1	R2	R3	R4	R5	T1	T2
Failure* Loads	M_u	1015	1197	1166**	1085**	610	582.5	754.5
	T_u	492	336	451	336	572.5	282	177
	V_u	0	0	11	12.5	0	0	0
Beam* Capacity	M_{uo}	1370	1370	1370	1370	1370	915	915
	T_{uo}	390	390	390	390	390	300	300
	V_{uo}**	70.7	70.7	70.7	70.7	70.7	57.44	57.44
Dimensionless Failure Loads	$\frac{M_u}{M_{uo}}$	.741	.874	.851	.792	.446	.637	.825
	$\frac{T_u}{T_{uo}}$	1.262	.862	1.157	.86	1.468	.94	.59
	$\frac{V_u}{V_{uo}}$	0	0	.155	.177	0	0	0

* All values are average values for the failure load increment
** Bending moment values distance h (beam depth) from centre concentrated load.
*** Theoretical Value

Note: All units are in inches and kips

TABLE 6.3 COMPUTER MODEL RESULTS



(a) Unstable cracked reinforced concrete element

(b) Stable cracked reinforced concrete element

* Note: Web reinforcement is uniformly distributed across element, but when crack forms through centroid, the reinforcement is effectively distributed across the crack only.

FIG. 6.1(A) METHOD OF MODELLING WEB REINFORCEMENT

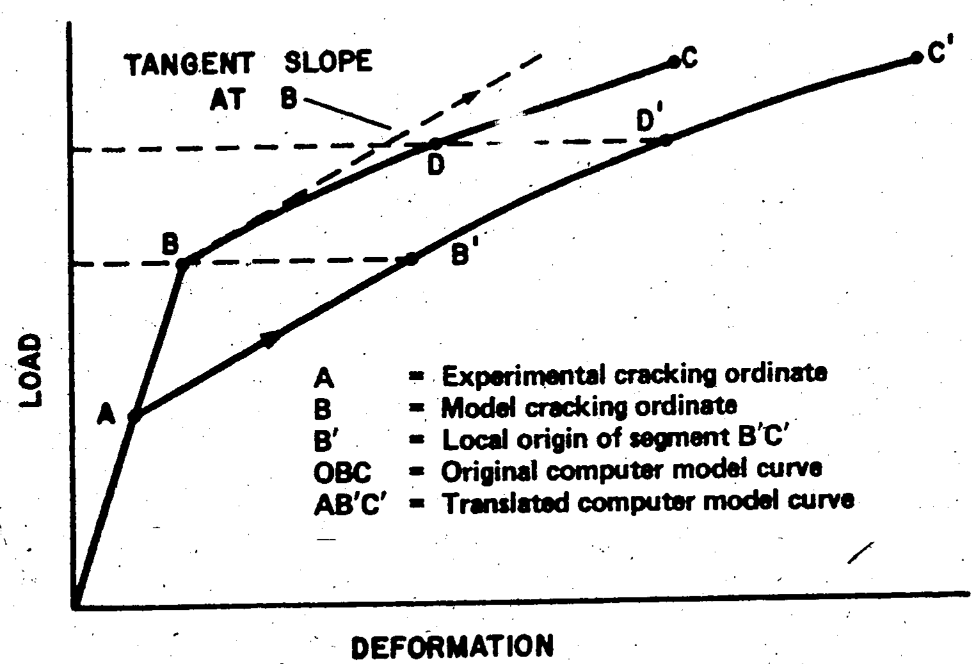


FIG. 6.1(B) TRANSLATION PROCEDURE

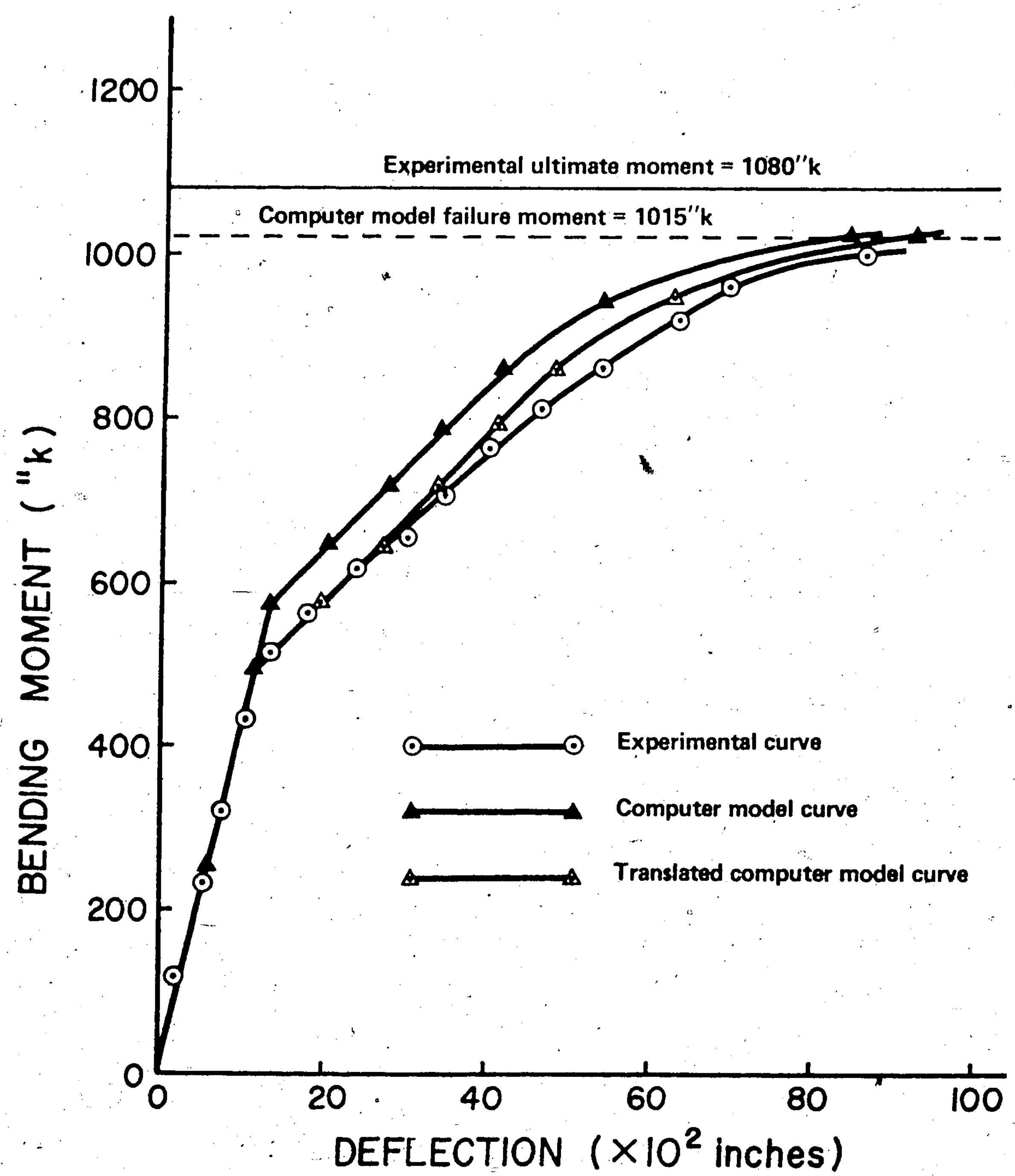


FIG. 6.2 MODEL AND TEST BENDING MOMENT-DEFLECTION CURVES FOR BEAM R1

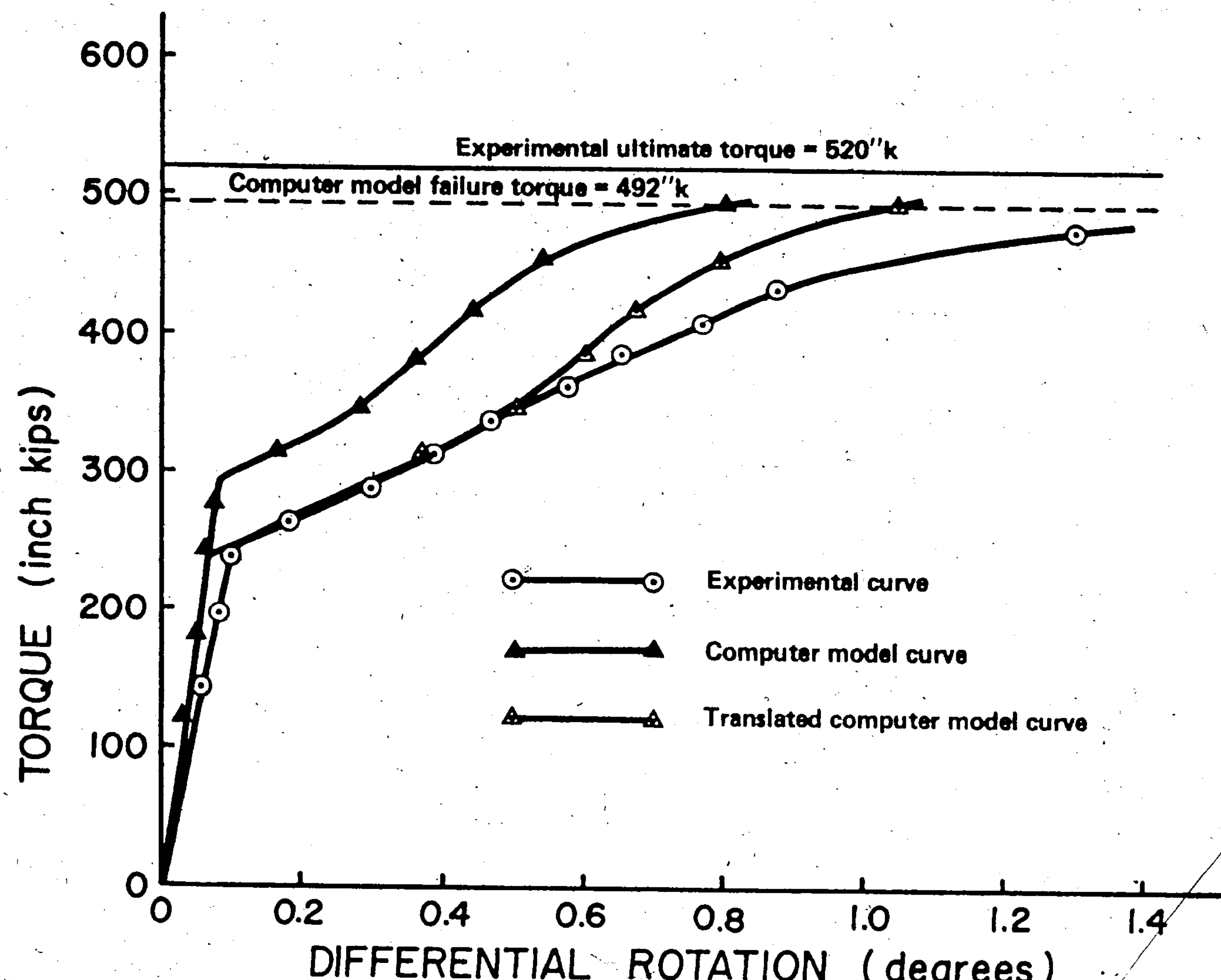


FIG. 6.3 MODEL AND TEST TORQUE-ROTATION CURVES FOR BEAM R1

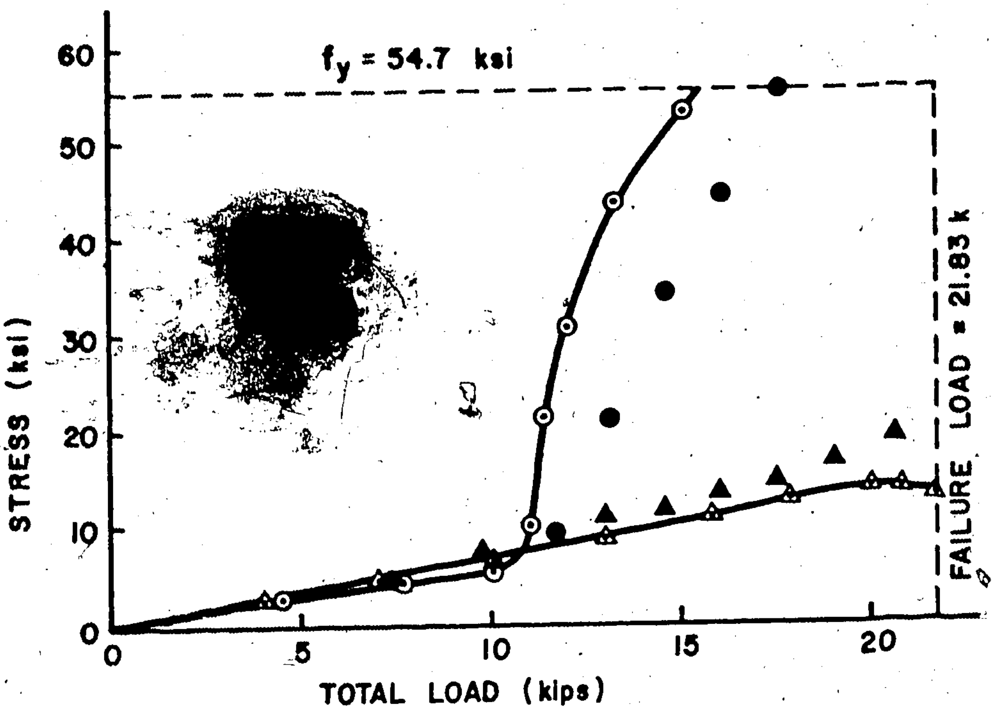


FIG. 6.4(A) TEST AND MODEL LONGITUDINAL CONVENTIONAL REINFORCEMENT STRESSES FOR BEAM R1

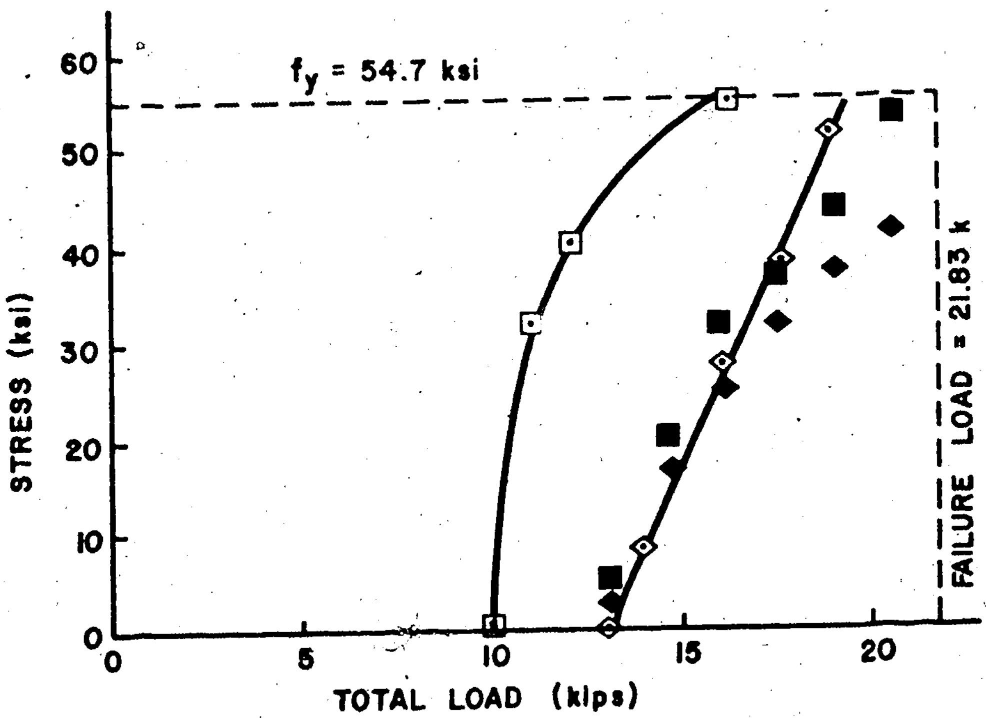


FIG. 6.4(B) TEST AND MODEL HOOP REINFORCEMENT STRESSES FOR BEAM R1

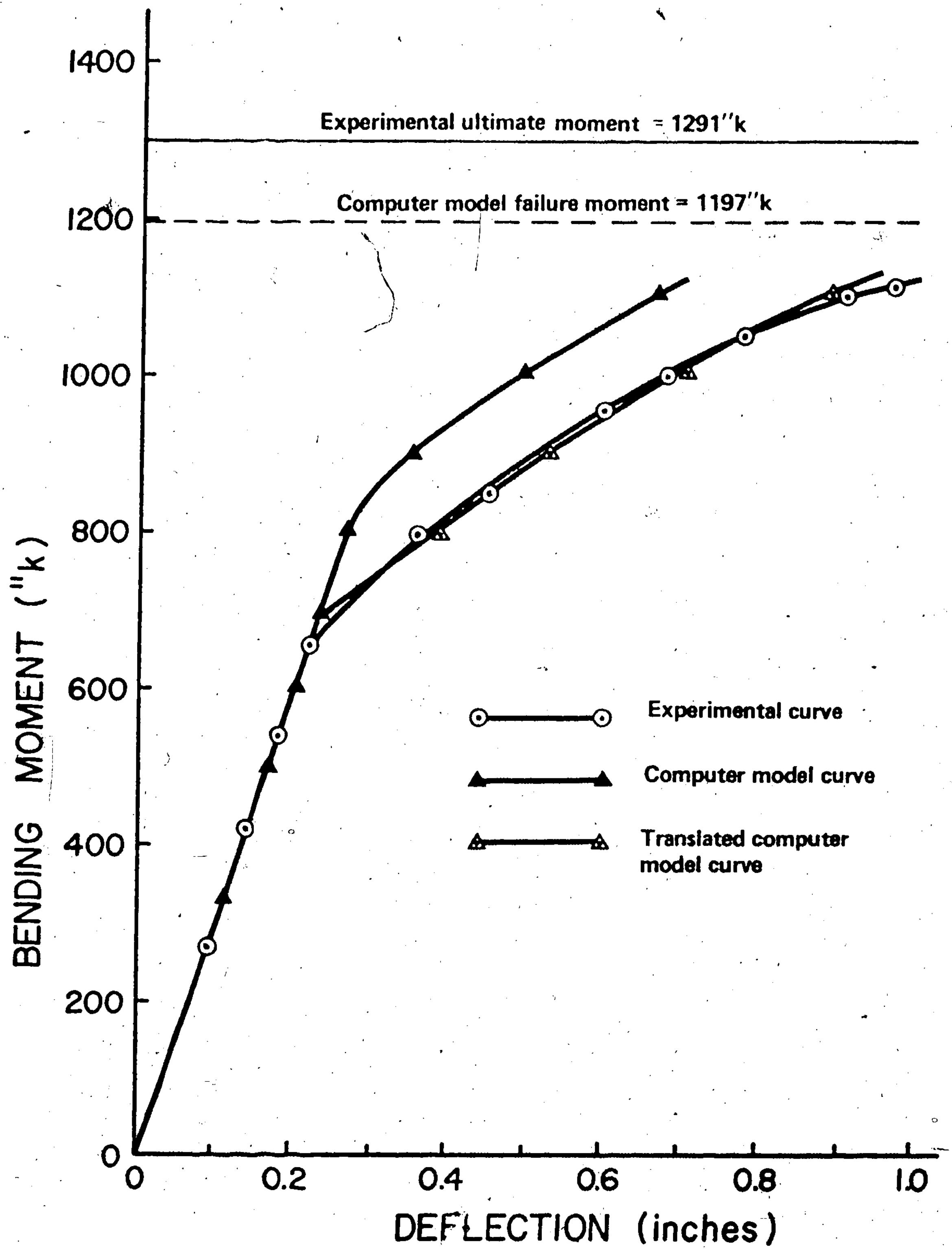


FIG. 6.5 MODEL AND TEST BENDING MOMENT-DEFLECTION CURVES FOR BEAM R2

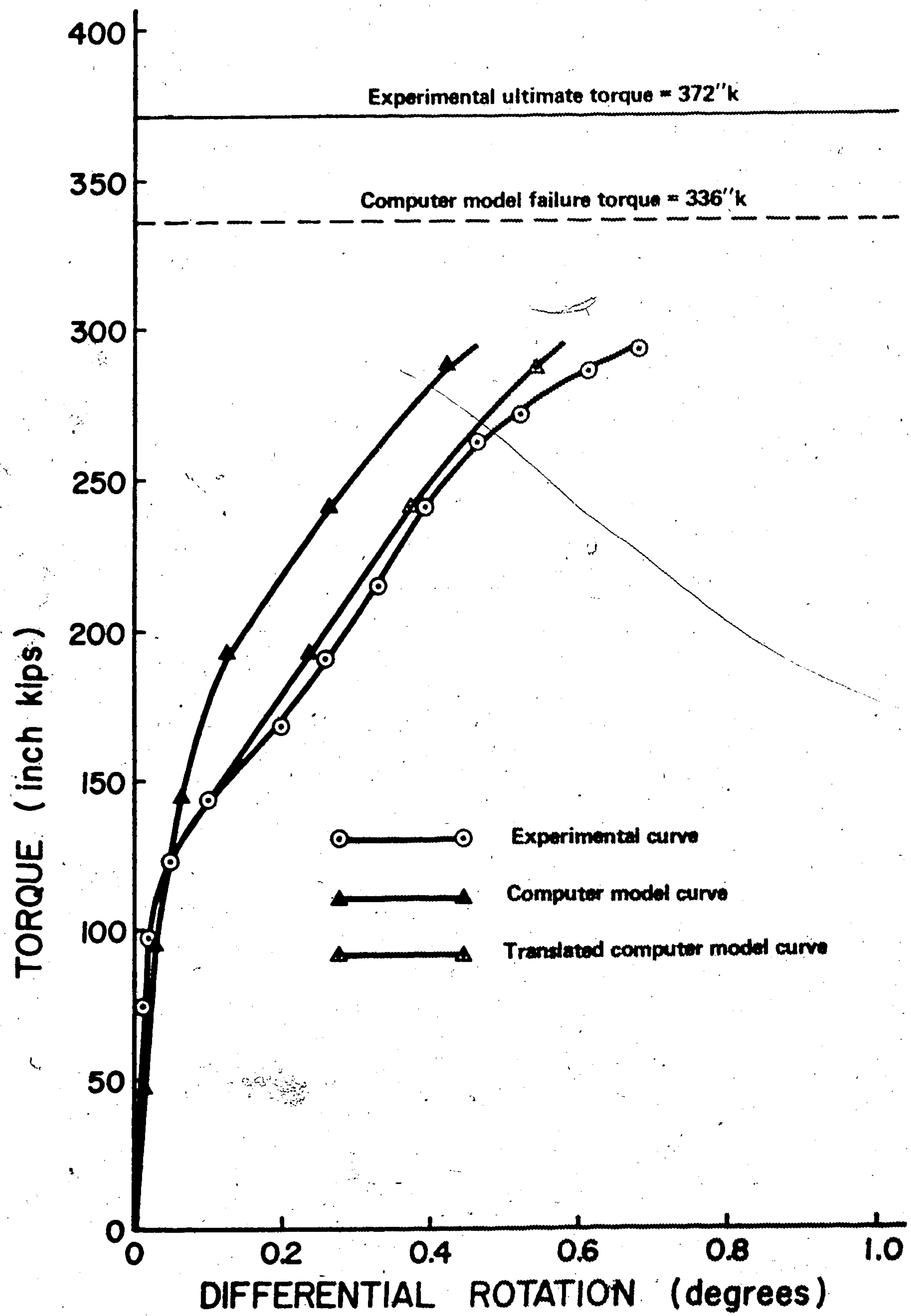


FIG. 6.6 MODEL AND TEST TORQUE-ROTATION CURVES FOR BEAM R2

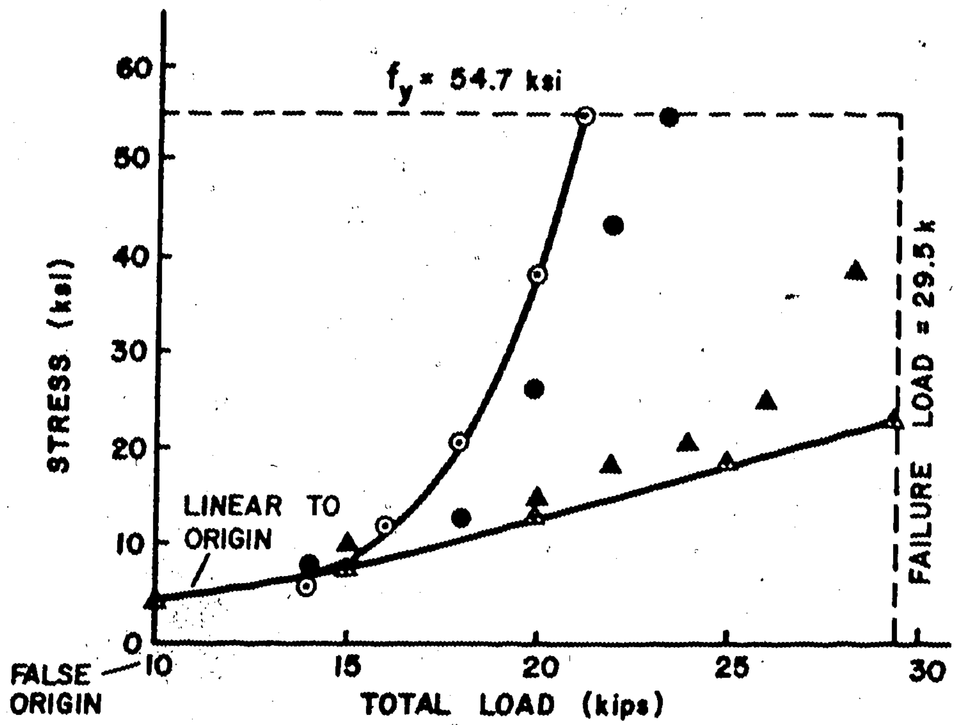


FIG. 6.7(A) TEST AND MODEL LONGITUDINAL CONVENTIONAL REINFORCEMENT STRESSES FOR BEAM R2

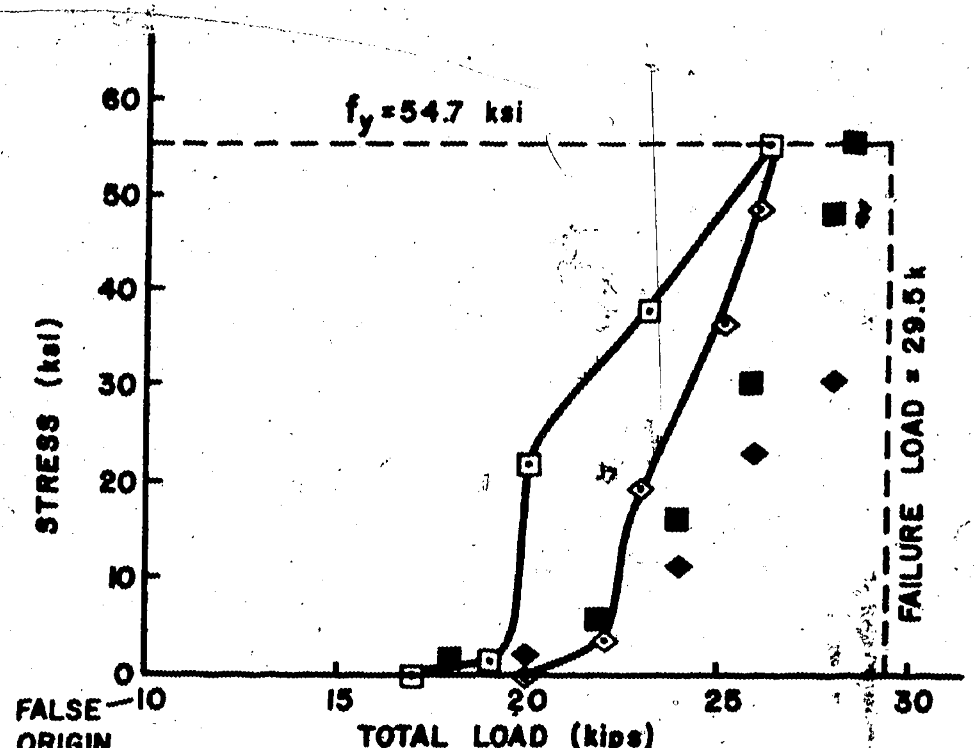


FIG. 6.7(B) TEST AND MODEL HOOP REINFORCEMENT STRESSES FOR BEAM R2

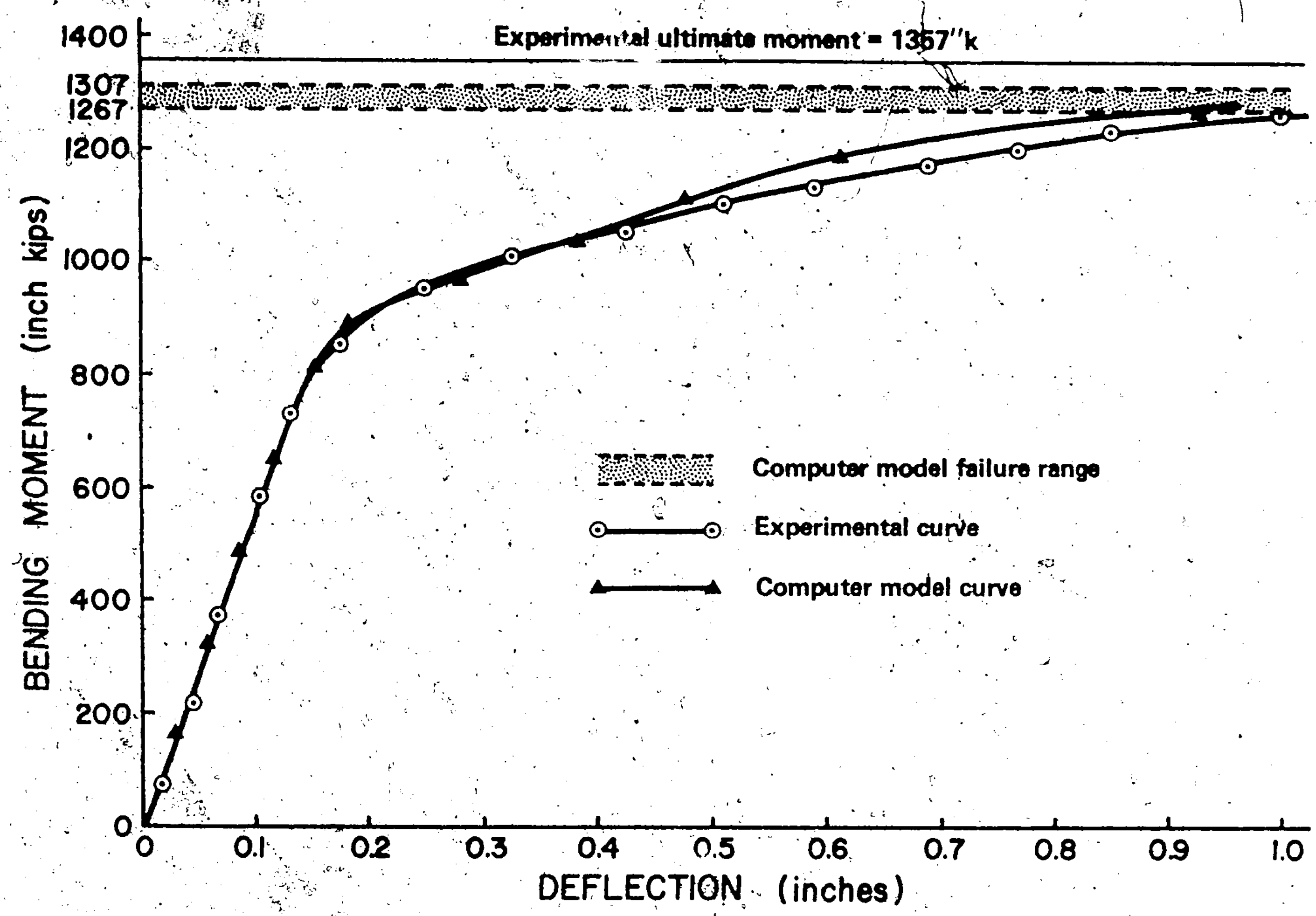


FIG. 6.8 MODEL AND TEST BENDING MOMENT-DEFLECTION CURVES FOR BEAM R3

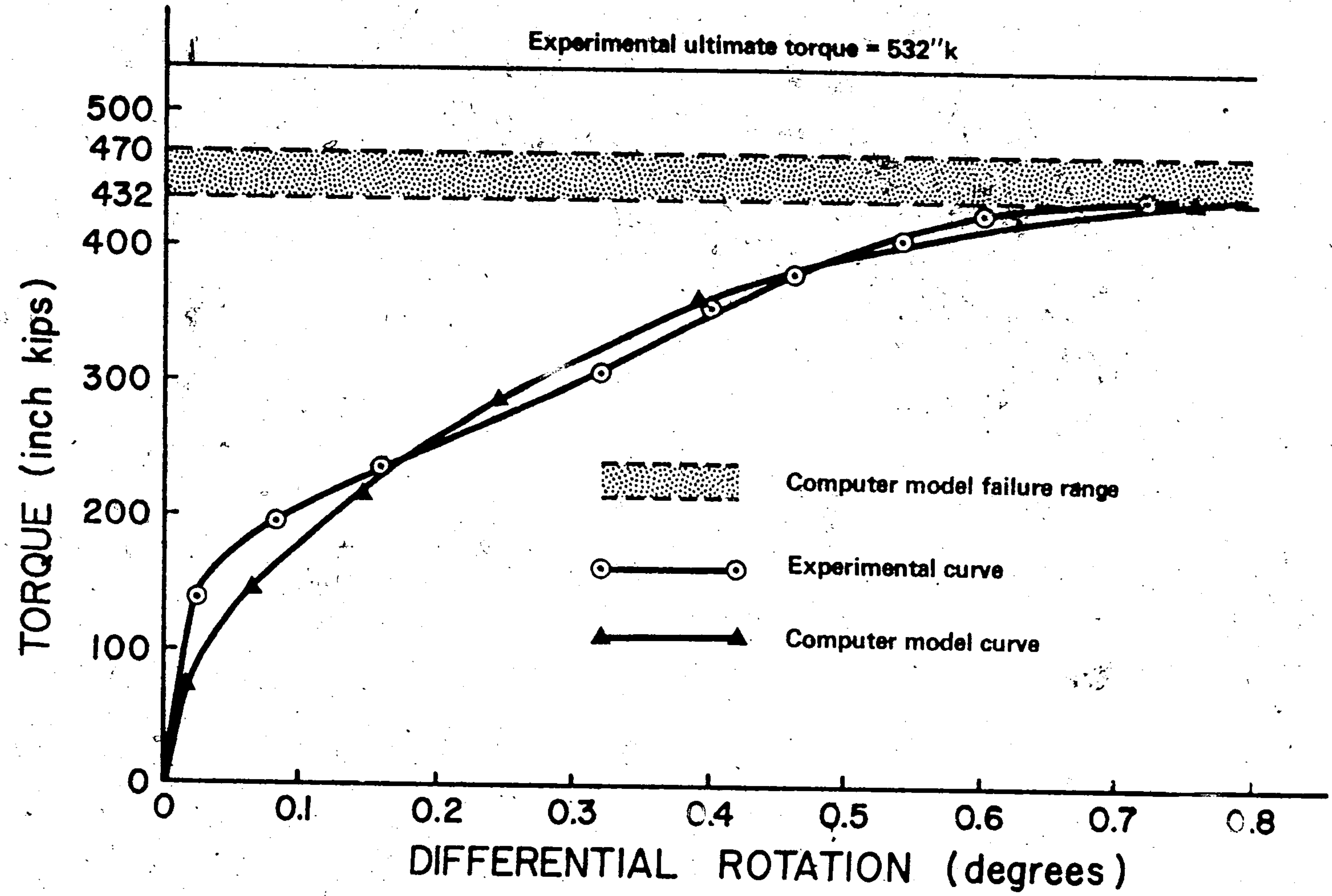


FIG. 6.9 MODEL AND TEST TORQUE-ROTATION CURVES FOR BEAM R3

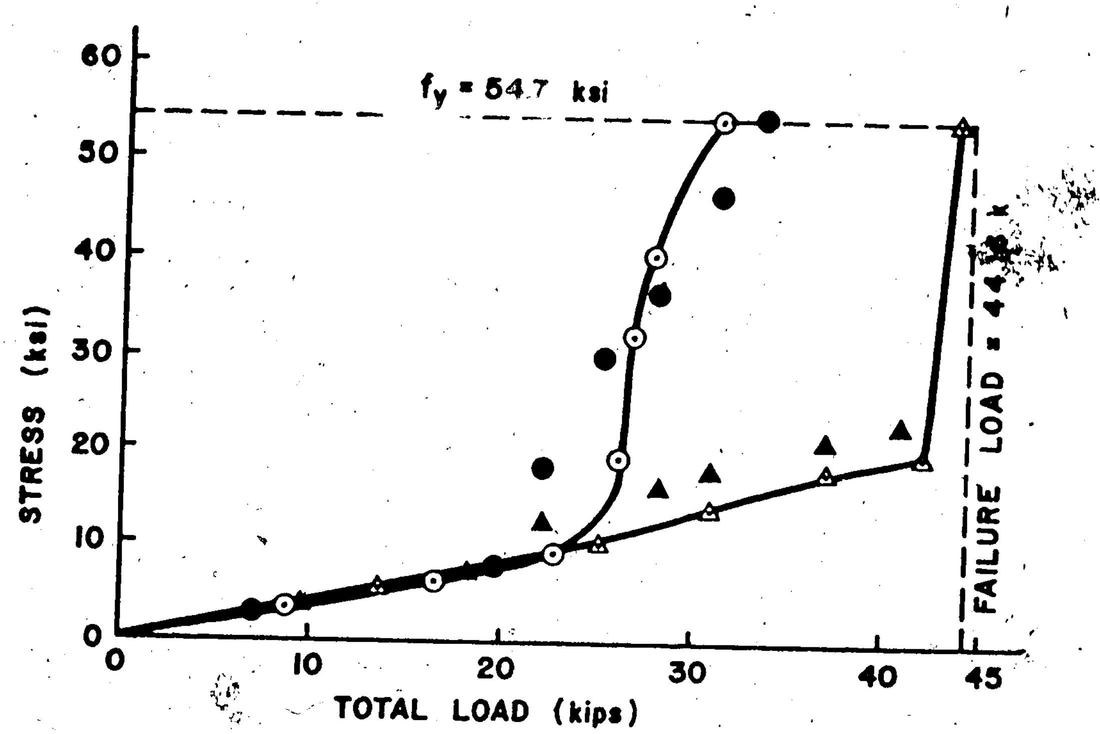


FIG. 6.10(A) TEST AND MODEL LONGITUDINAL CONVENTIONAL REINFORCEMENT STRESSES FOR BEAM R3

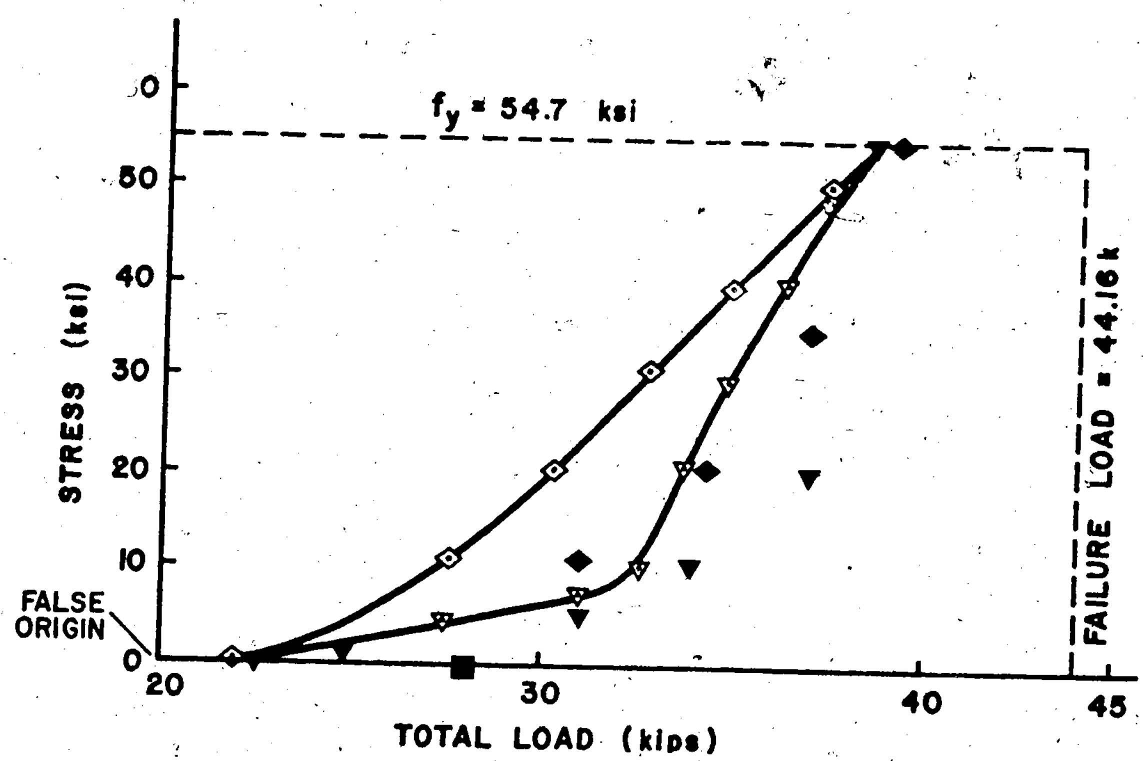


FIG. 6.10(B) TEST AND MODEL HOOP REINFORCEMENT STRESSES FOR BEAM R3

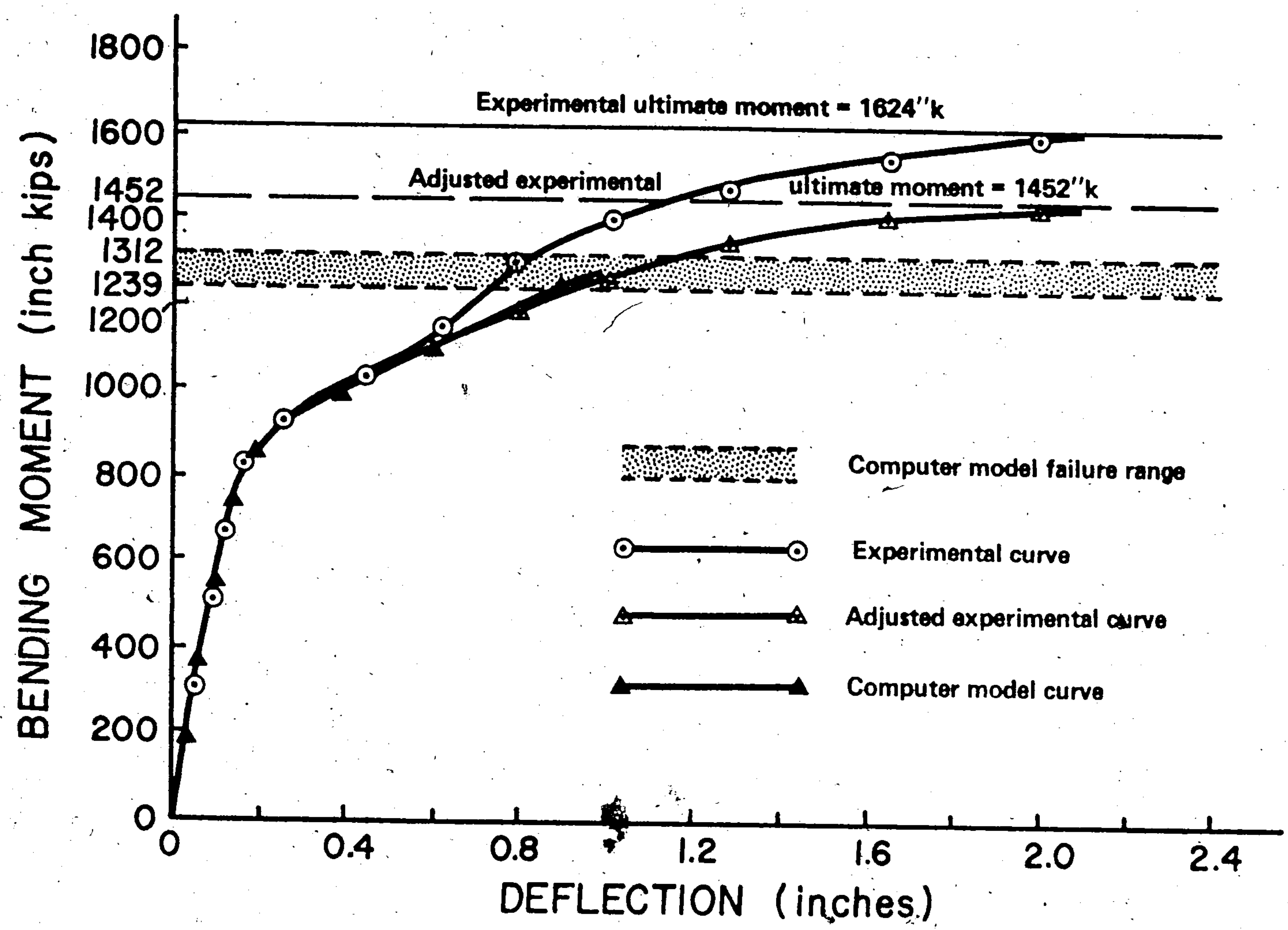


FIG. 6.11 MODEL AND TEST BENDING MOMENT-DEFLECTION CURVES FOR BEAM R4

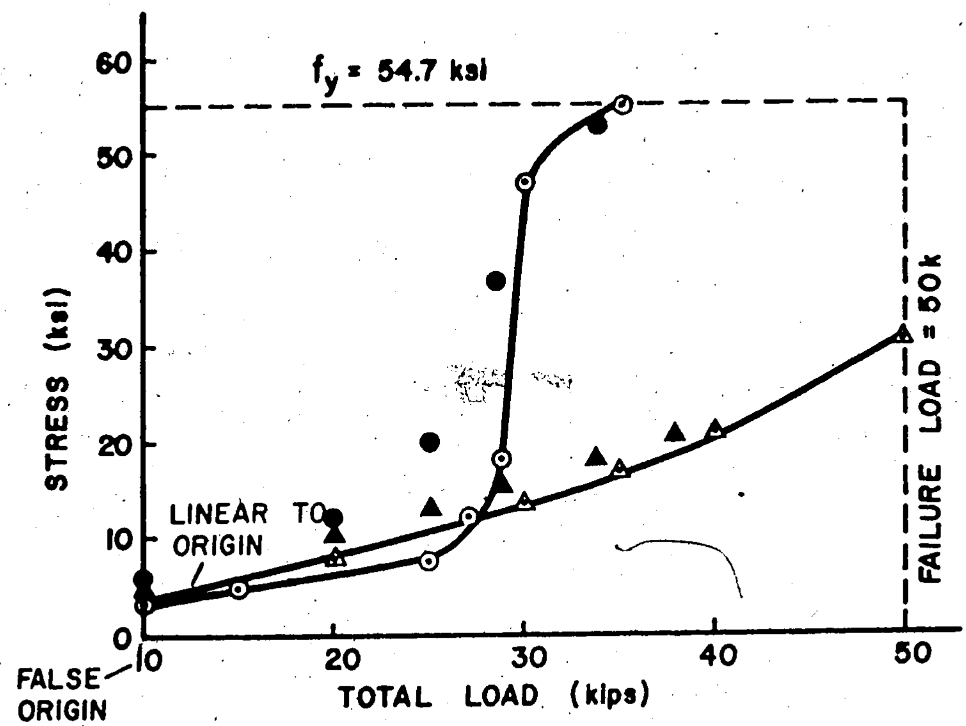


FIG. 6.12(A) TEST AND MODEL LONGITUDINAL CONVENTIONAL REINFORCEMENT STRESSES FOR BEAM R4

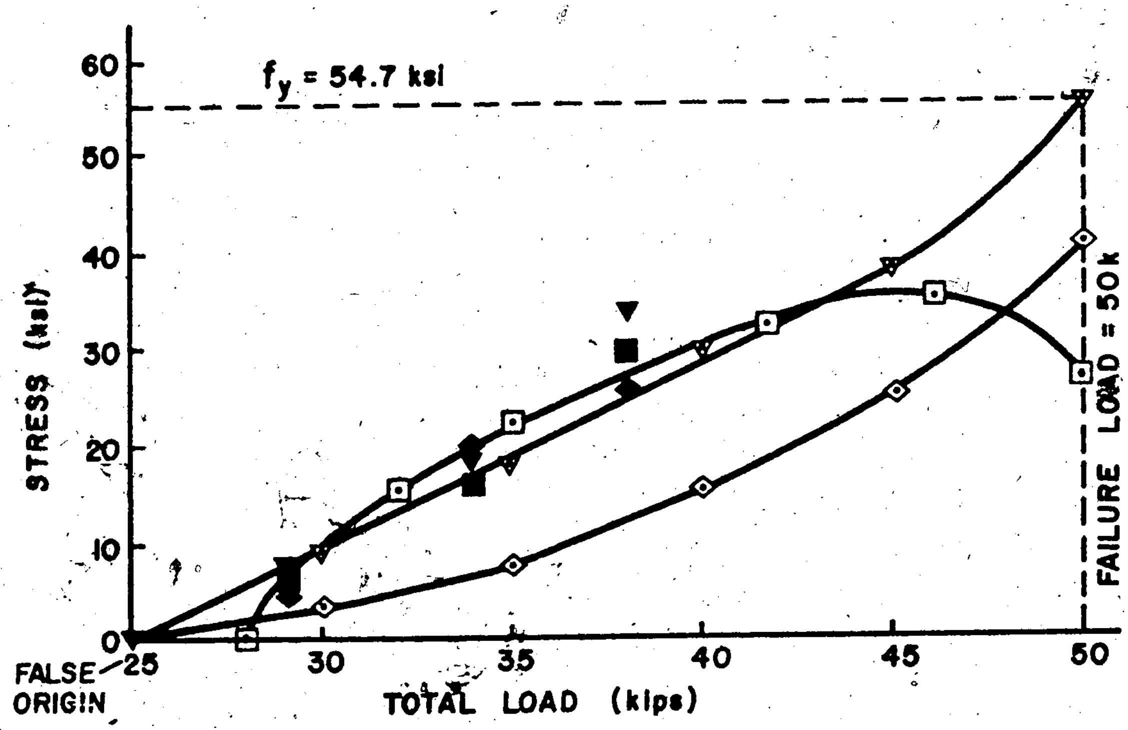


FIG. 6.12(B) TEST AND MODEL HOOP REINFORCEMENT STRESSES FOR BEAM R4

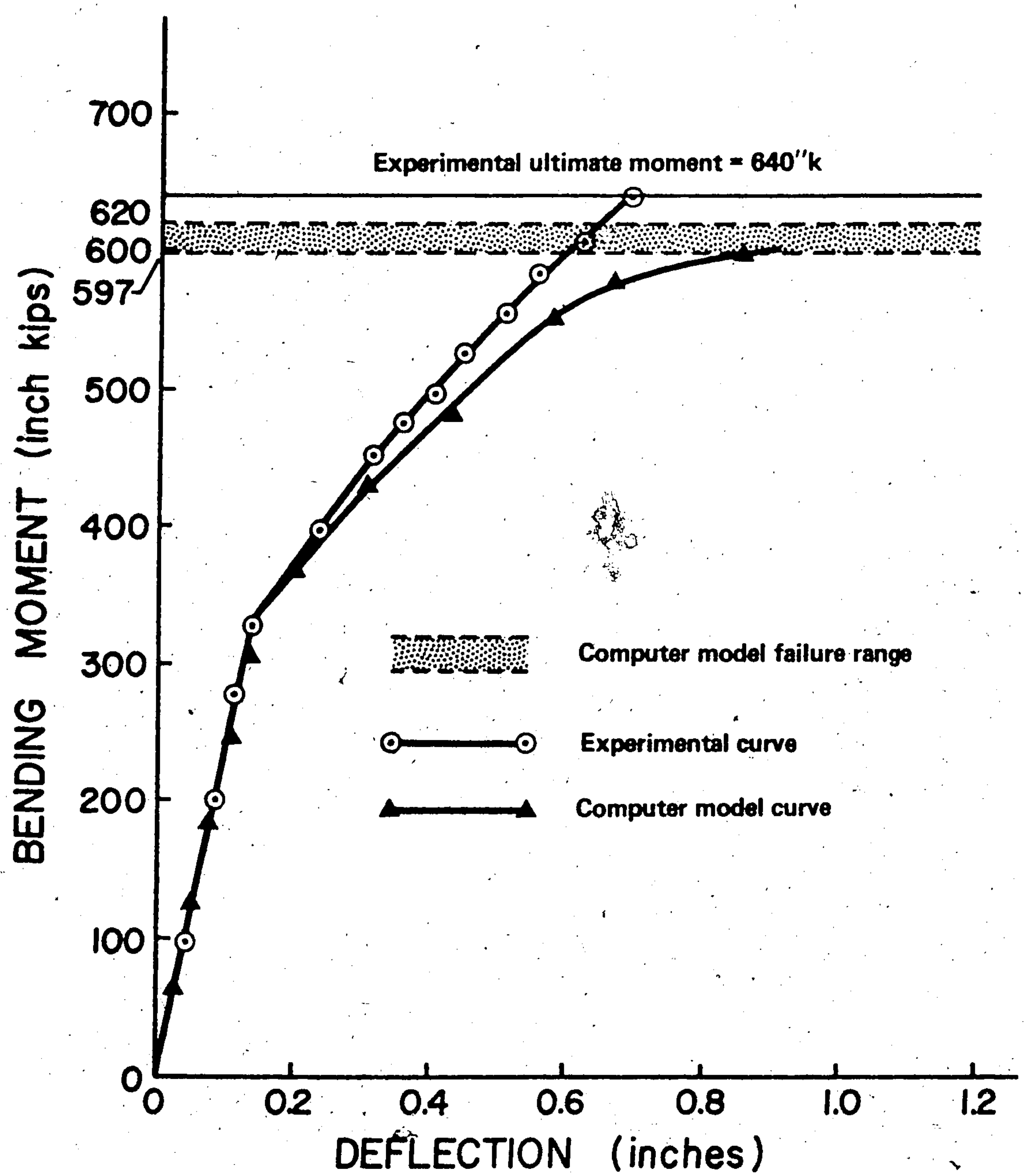


FIG. 6.13 MODEL AND TEST BENDING MOMENT-DEFLECTION CURVES FOR BEAM R5

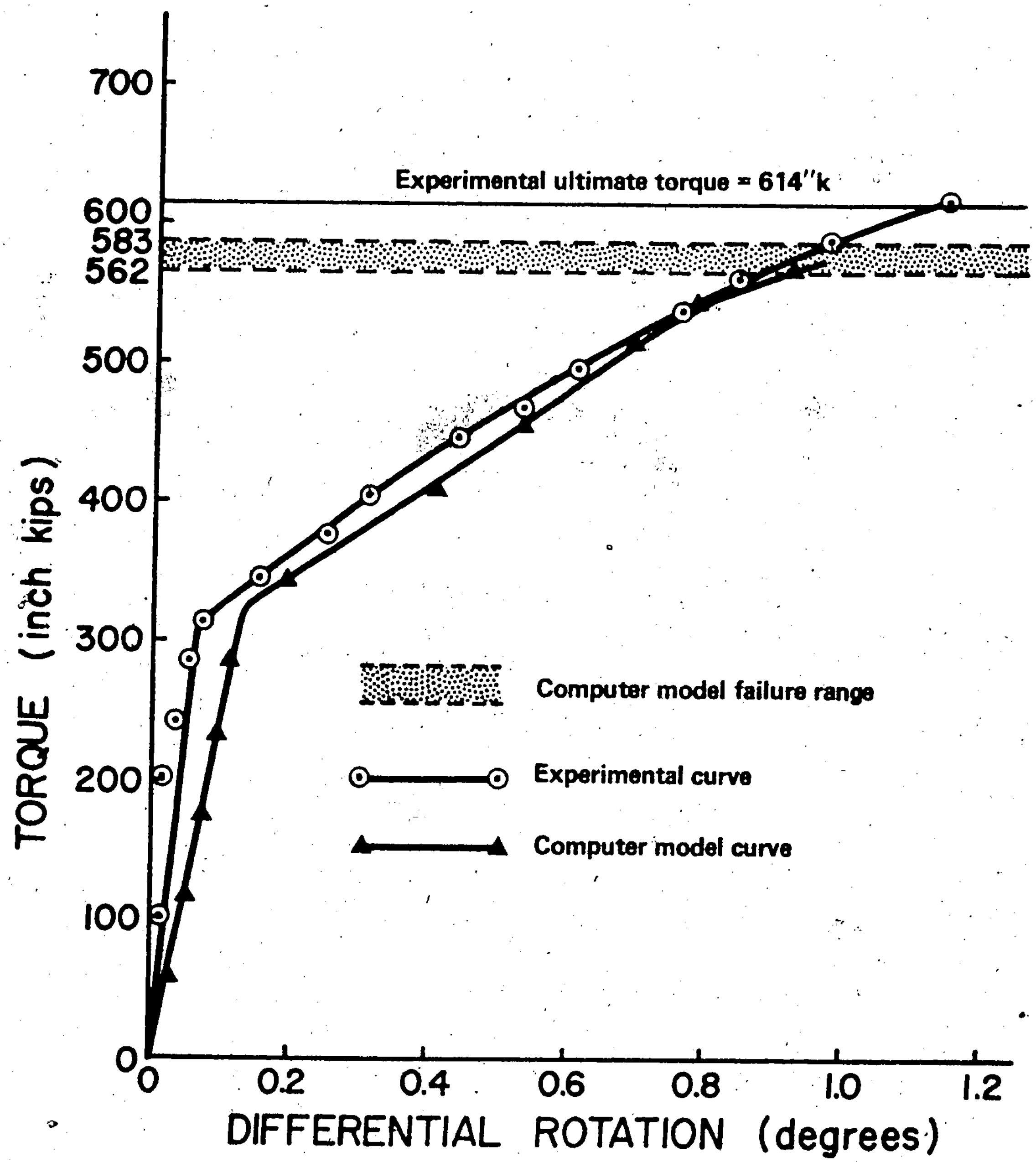


FIG. 6.14 MODEL AND TEST TORQUE-ROTATION CURVES FOR BEAM R5

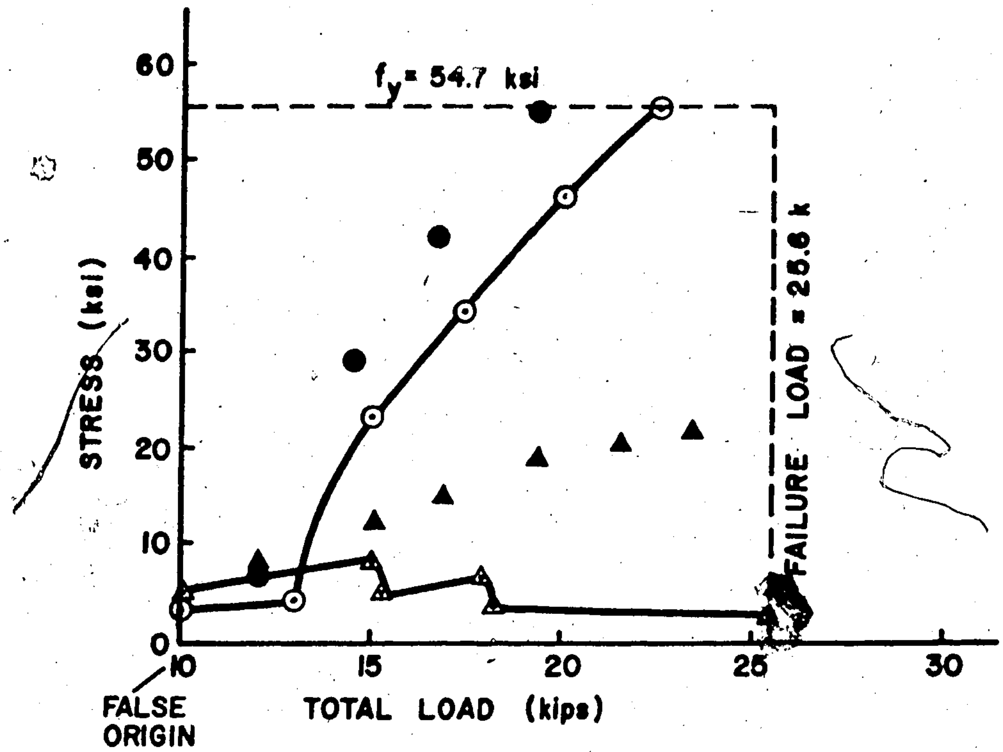


FIG. 6.15(A) TEST AND MODEL LONGITUDINAL CONVENTIONAL REINFORCEMENT STRESSES FOR BEAM R5

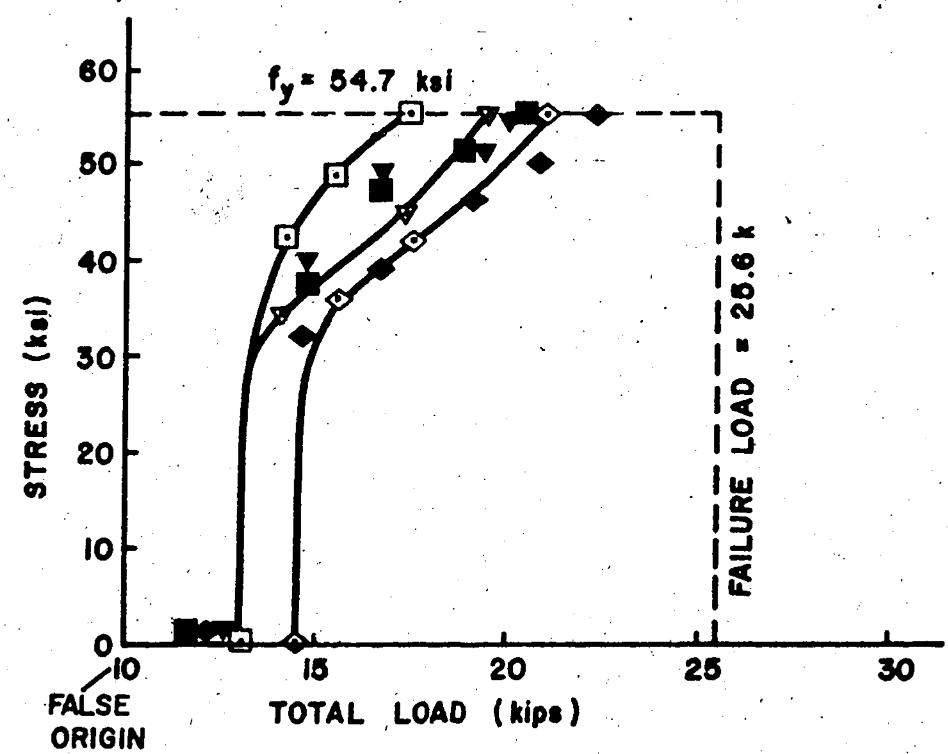


FIG. 6.15(B) TEST AND MODEL HOOP REINFORCEMENT STRESSES FOR BEAM R5

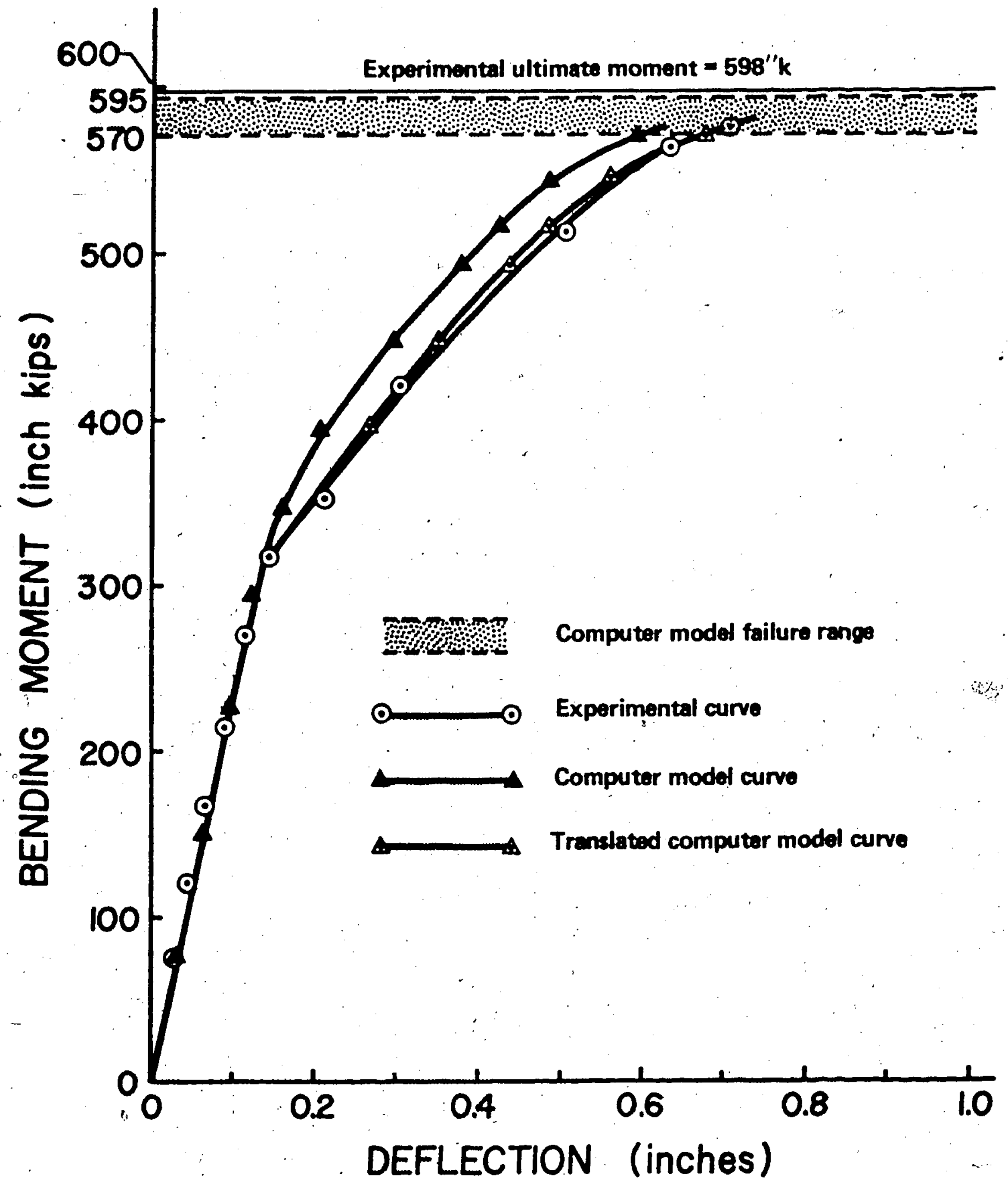


FIG. 6.16 MODEL AND TEST BENDING MOMENT DEFLECTION CURVES FOR BEAM T1

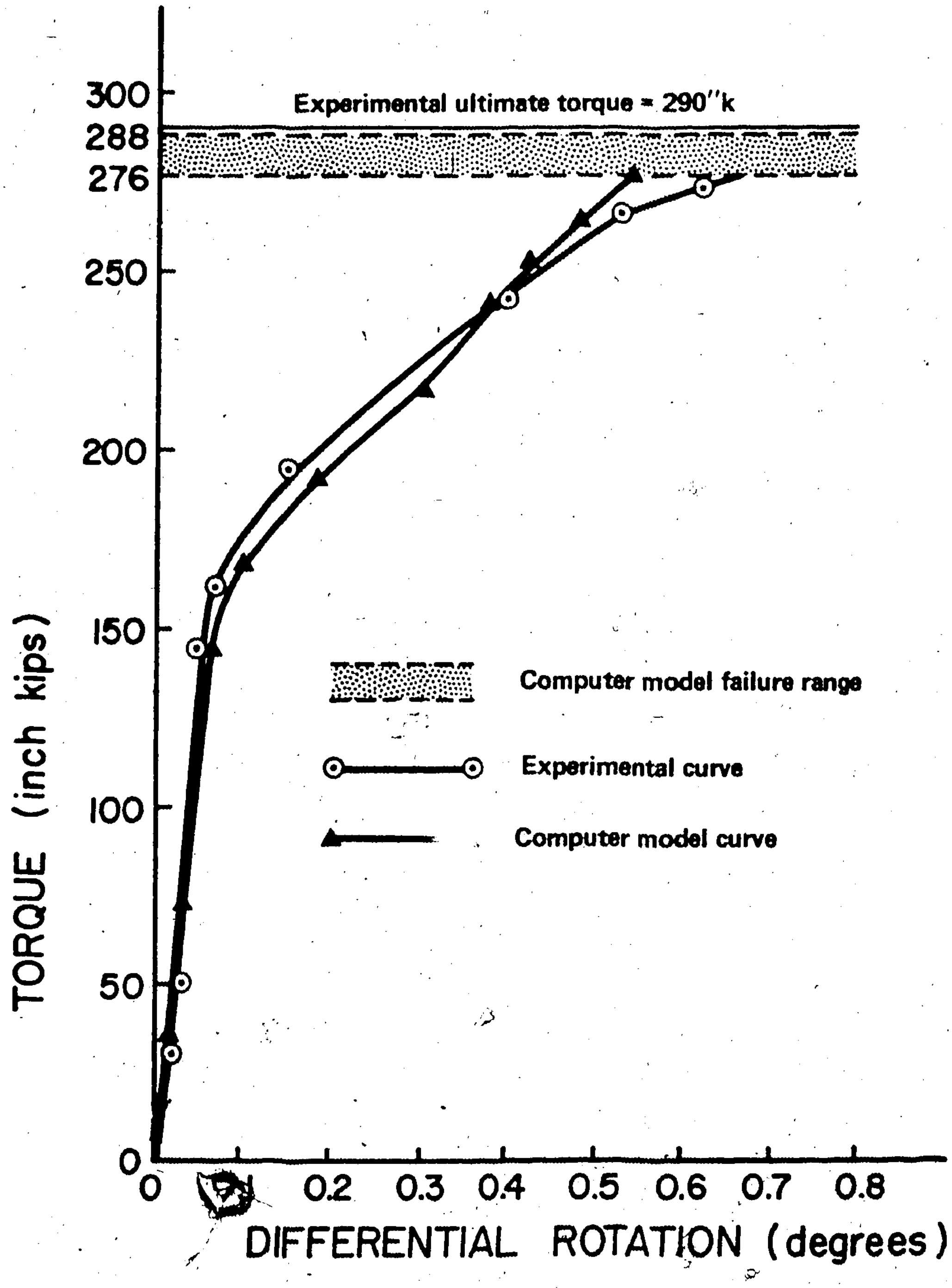


FIG. 6.17 MODEL AND TEST TORQUE-ROTATION CURVES FOR BEAM T1

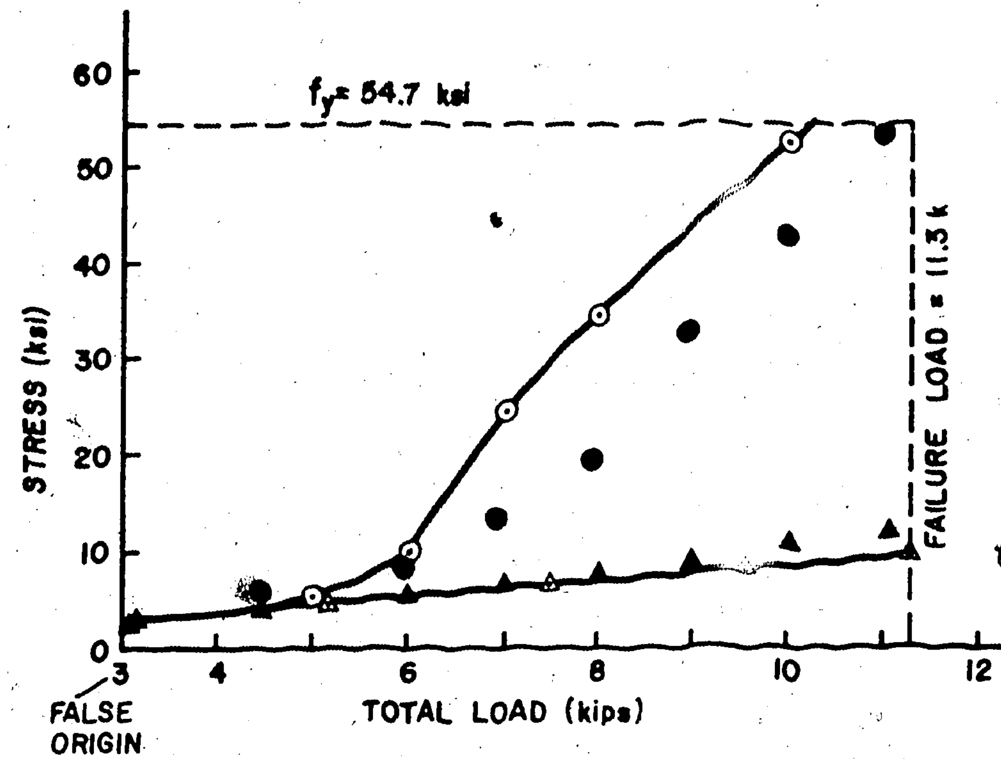


FIG. 6.18(A) TEST AND MODEL LONGITUDINAL CONVENTIONAL REINFORCEMENT STRESSES FOR BEAM T1

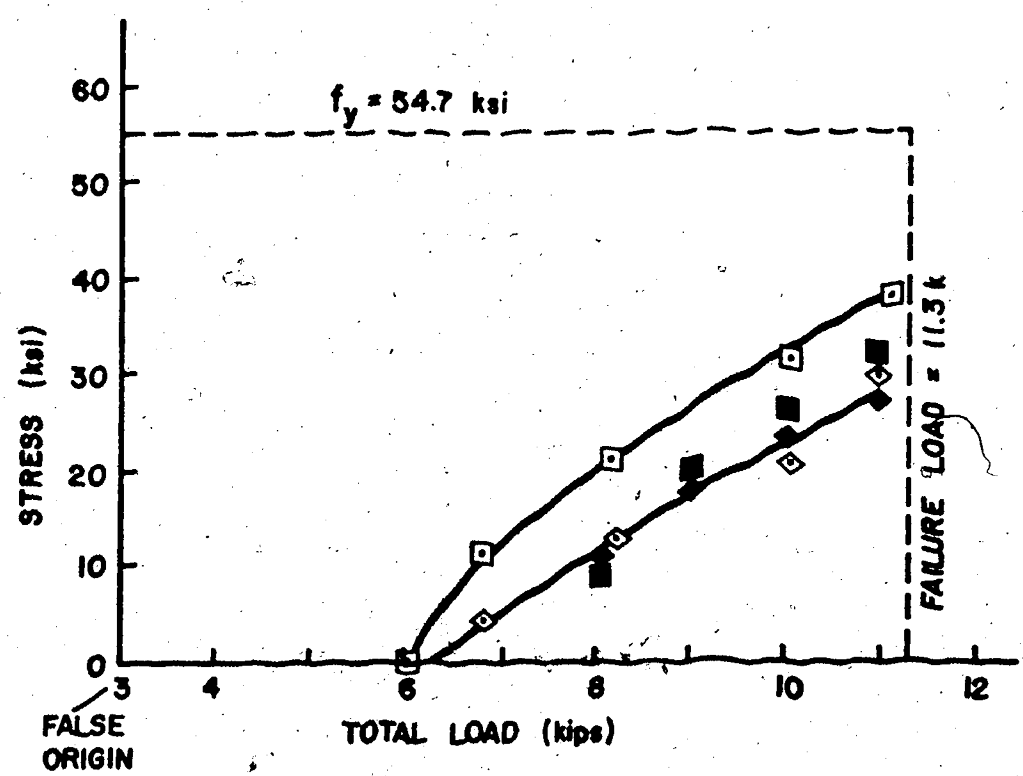


FIG. 6.18(B) TEST AND MODEL HOOP REINFORCEMENT STRESSES FOR BEAM T1

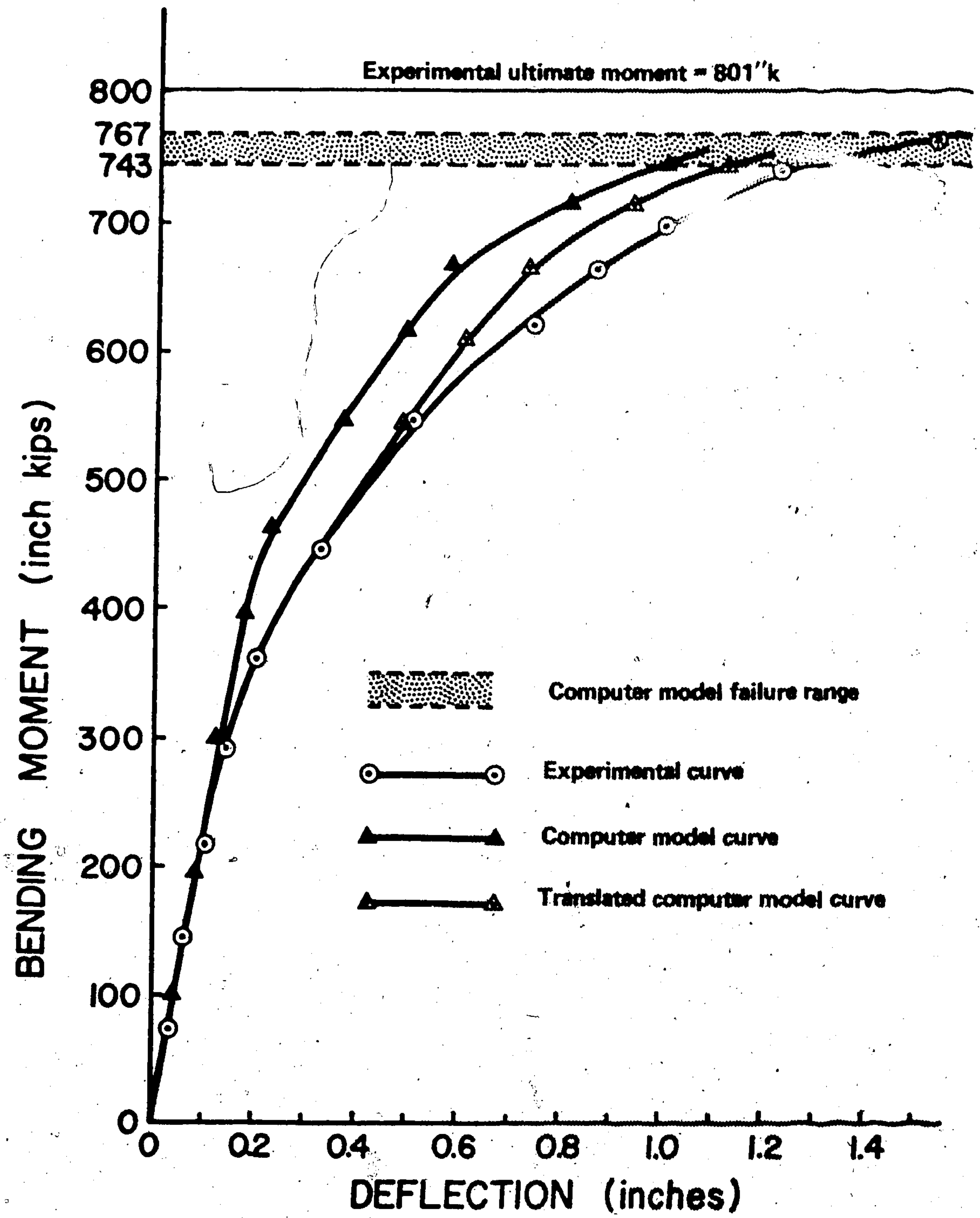


FIG. 6.19 MODEL AND TEST BENDING MOMENT-DEFLECTION CURVES FOR BEAM T2

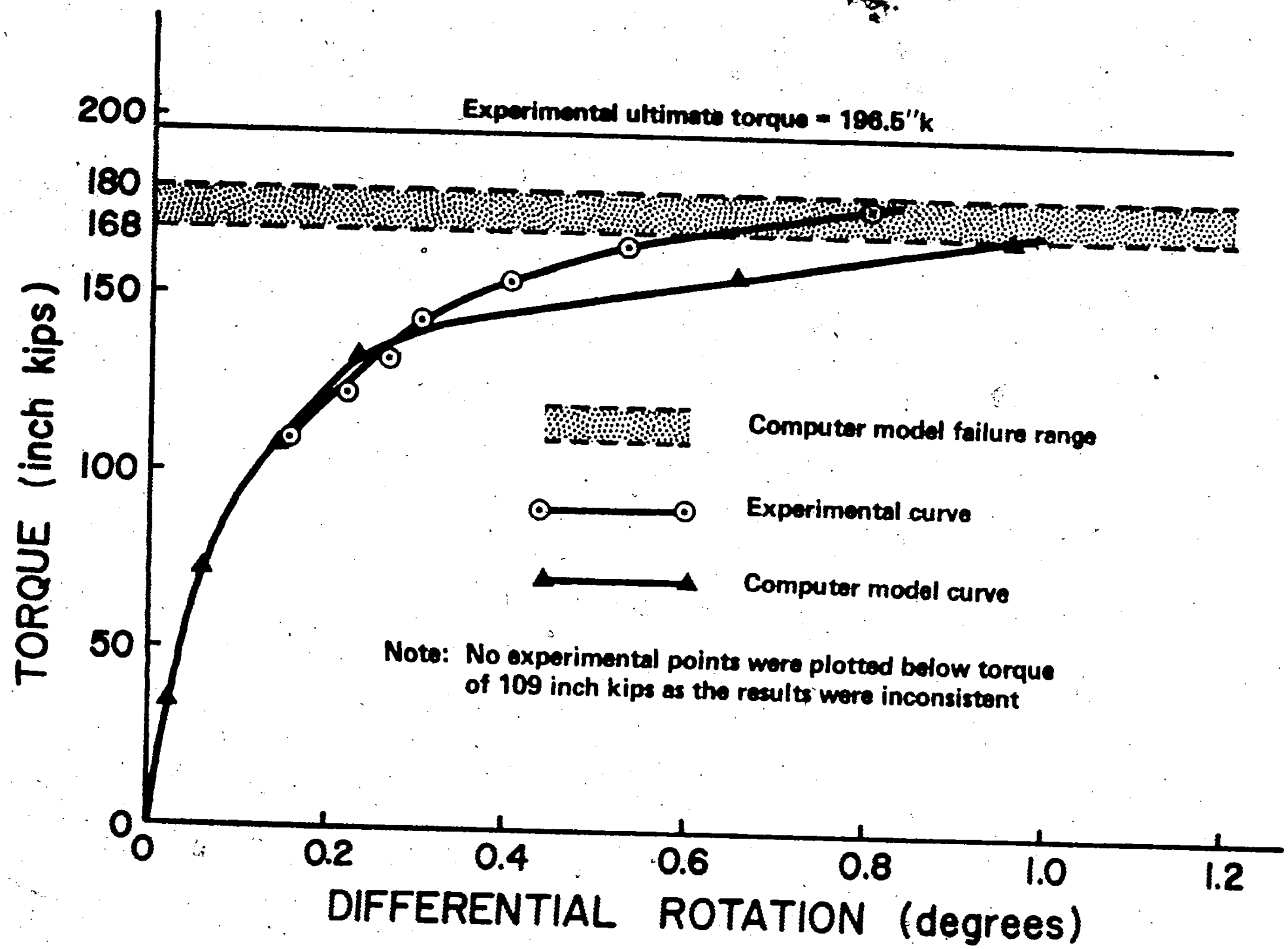


FIG. 9.20 MODEL AND TEST TORQUE-ROTATION CURVES FOR BEAM T2

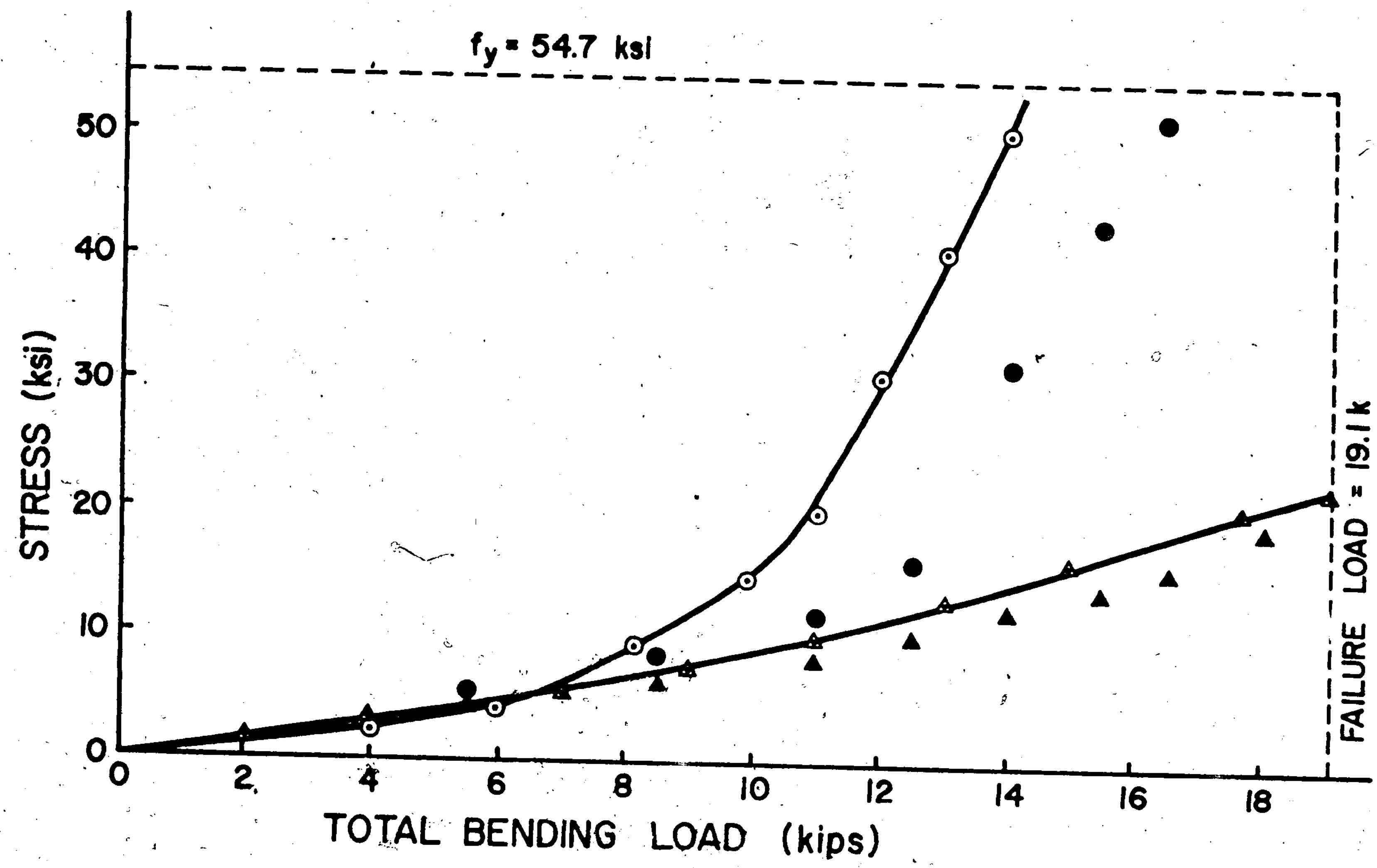
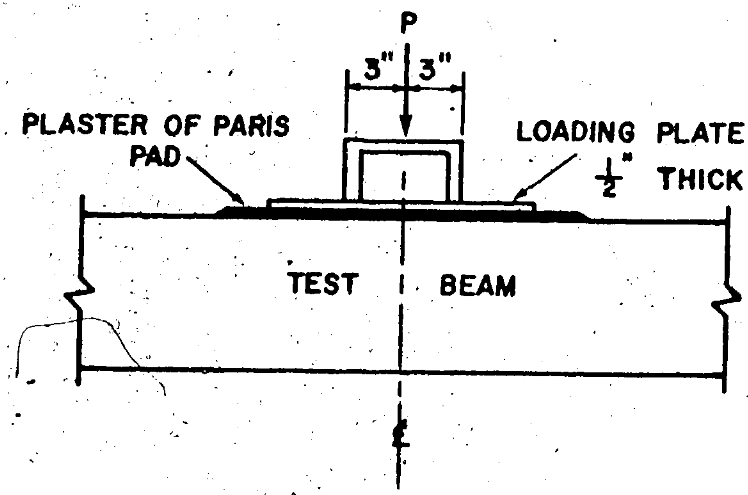
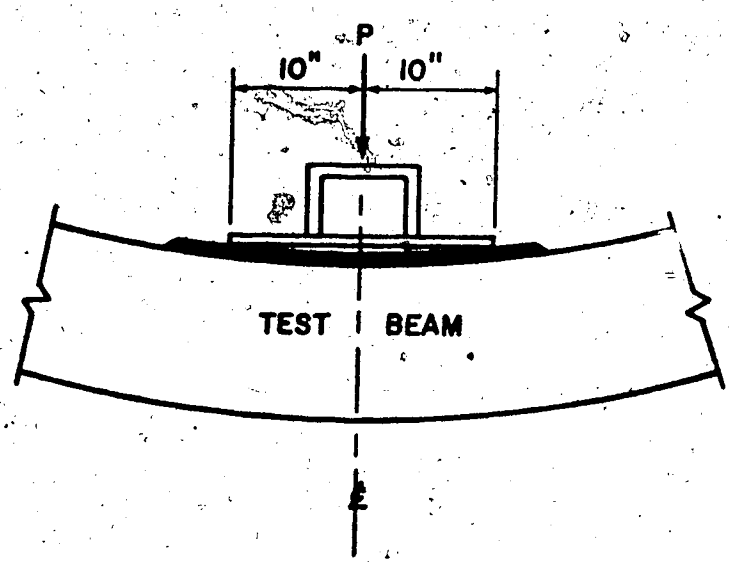


FIG. 6.21 TEST AND MODEL LONGITUDINAL CONVENTIONAL REINFORCEMENT STRESSES FOR BEAM T2



Beam span = 147"
Maximum moment due to P = 35.25P

(a) Complete contact of loading plate and plaster of paris pad



Maximum moment due to P = 31.75P

(b) Edge contact of loading plate and plaster of paris pad

FIG. 6.22 CENTRAL LOAD VARIATION FOR BEAM R4

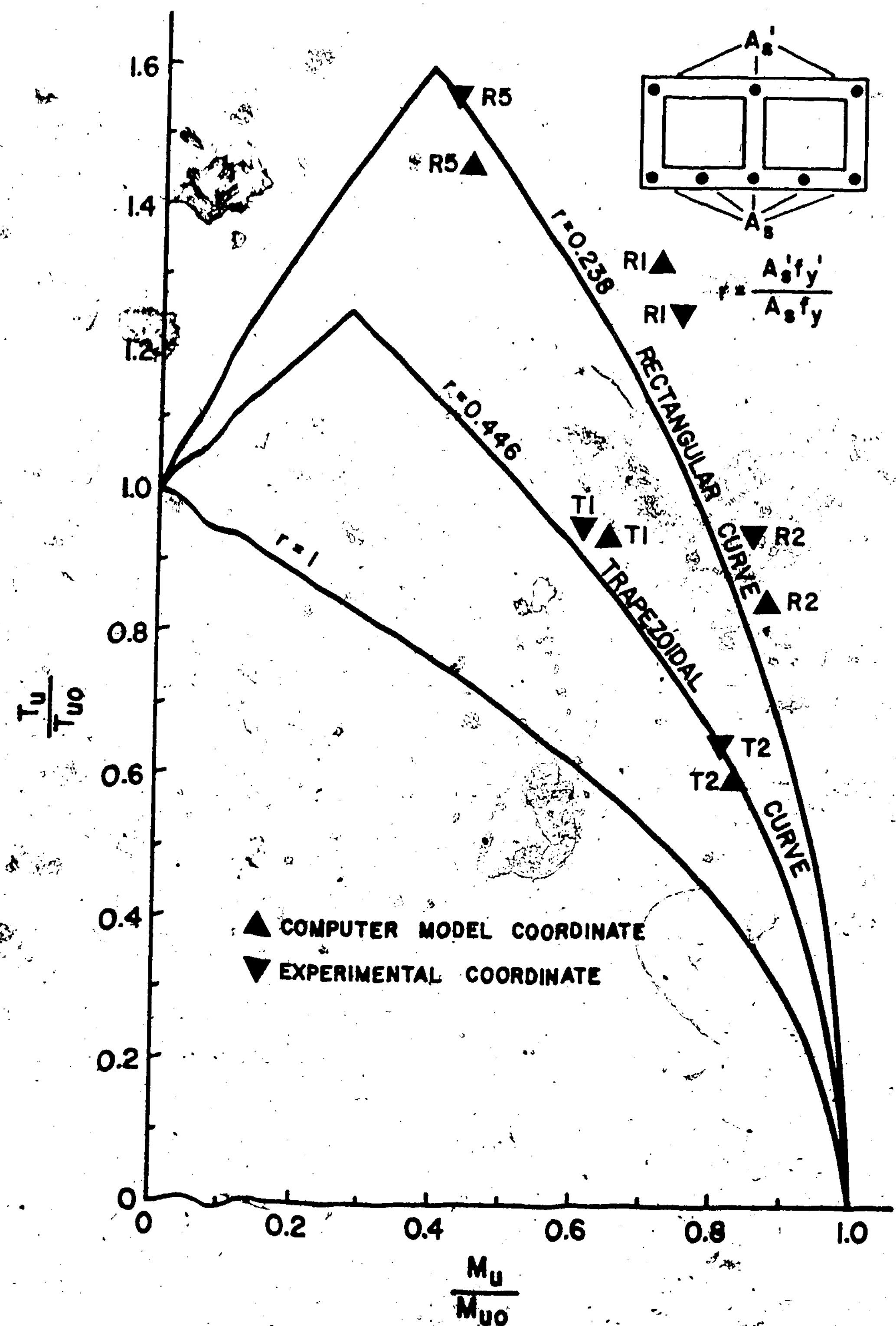


FIG. 6.23 TORQUE-BENDING MOMENT INTERACTION DIAGRAM FOR MODEL EVALUATION

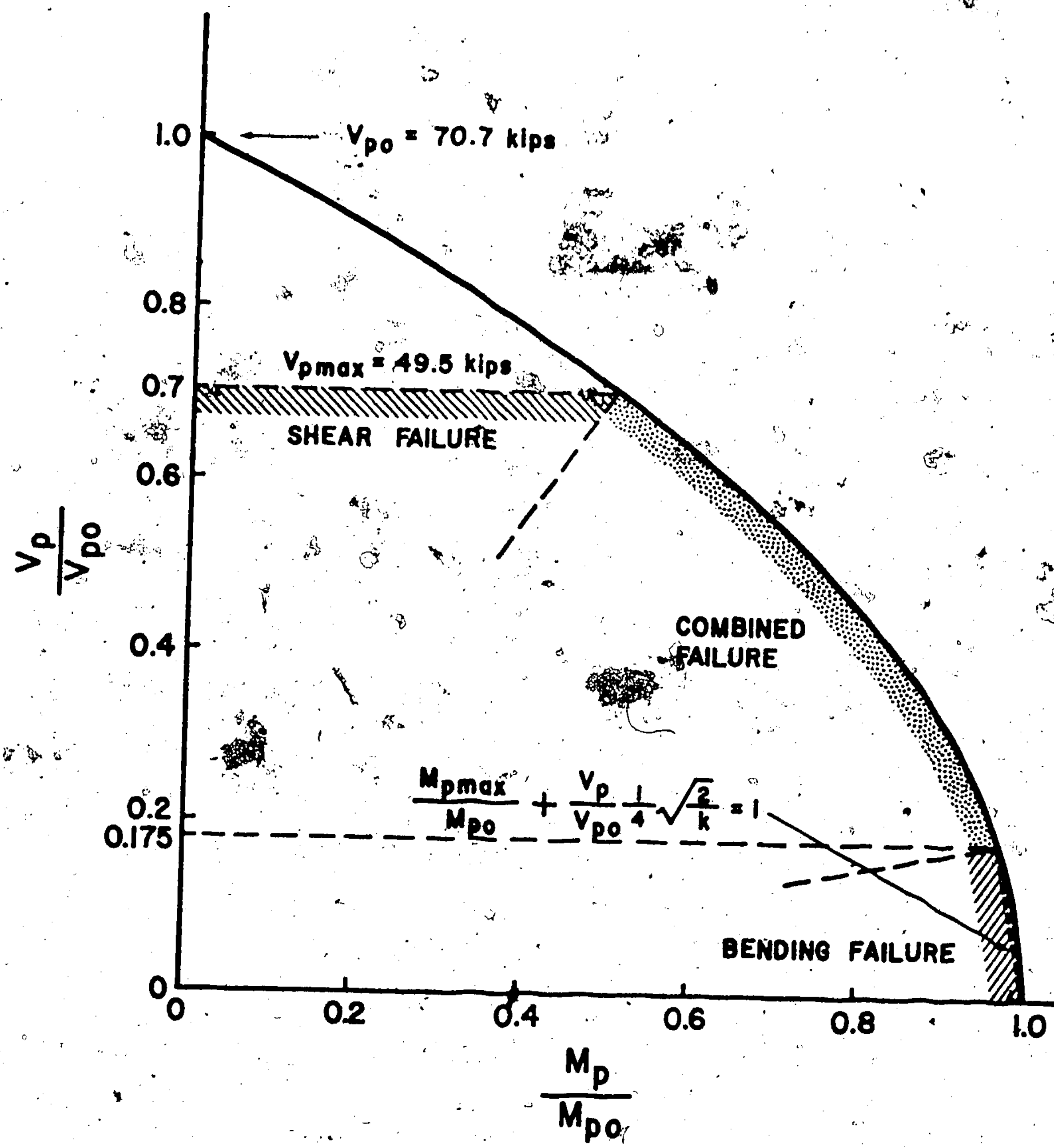
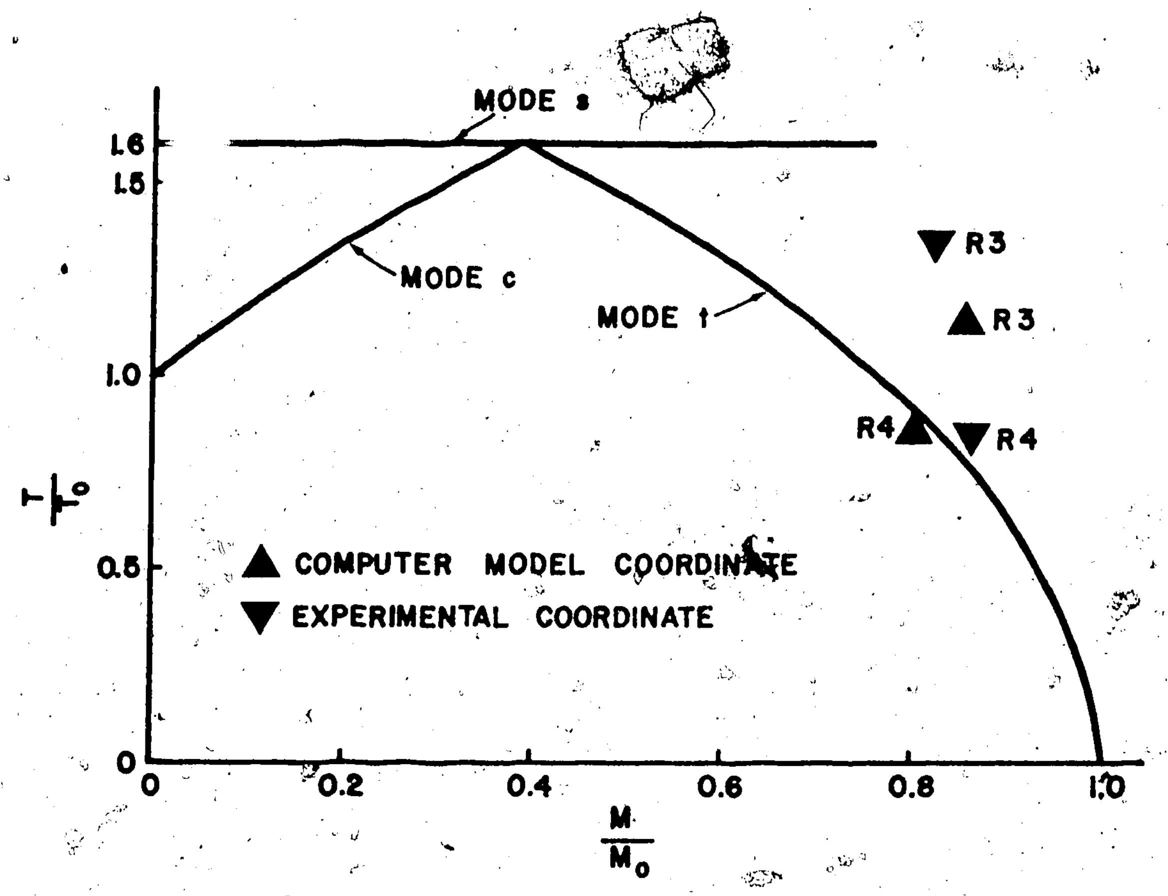


FIG. 6.24 INTERACTION DIAGRAM FOR BENDING MOMENT AND SHEAR



INTERACTION EQUATIONS FOR BEAM R3:

MODE t: $\frac{M}{M_o} + 0.2377\left(\frac{T}{T_o}\right)^2 = 0.994$

MODE c: $-4.207\frac{M}{M_o} + \left(\frac{T}{T_o}\right)^2 = 0.976$

MODE s: $\frac{T}{T_o} = 1.6$

INTERACTION EQUATIONS FOR BEAM R4:

MODE t: $\frac{M}{M_o} + 0.2377\left(\frac{T}{T_o}\right)^2 = 0.985$

MODE c: $-4.207\frac{M}{M_o} + \left(\frac{T}{T_o}\right)^2 = 0.936$

MODE s: $\frac{T}{T_o} = 1.6$

FIG. 6.25 ADJUSTED TORQUE-BENDING MOMENT-SHEAR INTERAC- DIAGRAM FOR BEAMS R3 AND R4

CHAPTER VII

CONCLUSION AND SUMMARY

7.1 Principal Implications of Comparison

Of the two sources of comparison that were utilized in the evaluation of the analytical model, the results of the small, comprehensive experimental program provided the better basis for a conclusive assessment.

From experimental comparisons, two potential sources of model inaccuracy were isolated; underconservative estimation of the cracking load under torsion loading conditions, and inaccurate representation of the bending moment lever arm at ultimate failure. The former qualification of behaviour can be overcome if the variation of torsional shear stress across the wall thickness is taken into account. To reflect the more accurate prediction of cracking load, the computer model results are transformed accordingly through the translation of the post-cracking sections of the bending moment-deflection and torque-rotation curves. Inaccurate prediction of the ultimate bending moment capacity cannot be resolved as the maximum lever arm length is fixed by geometry, but this deficiency is only of significance in underreinforced beams where the ratio of compression flange thickness to beam depth is unusually high. Since the modest degree of discrepancy between experimental and model results is principally derived from the latter qualification, accurate simulation of beam behaviour can be attained for concrete box girders of lower wall thickness to depth ratios.

The principal conclusion that arose from the model assessment in light of current theory was the general applicability of the analytical model achieved through freedom from restrictive assumptions. Where valid comparisons can be made, agreement between analytical and theoretical results is good. Theoretical interactive behaviour, presented in its dimensionless equation form, does not permit an explicit evaluation of model results, and can indeed be misleading because of the dimensionless form of presentation.

Consideration of the significant aspects of the assessment procedure affirm the value of the computer model as a capable, versatile, analytical tool in its context of the analysis of concrete box girders acted upon by torsion, bending, and shear. Qualification of its accuracy is isolated to one potential source of error, and the range of application is almost without restriction. Characteristic of all finite element modelling, the reliability of output information is reflected in the quality of input specifications.

7.2 Application of the Analytical Model

Within their respective confines of validity determined by explicit assumptions, theoretical strength predictions discussed in Chapter 6 are not in conflict with analytical model results. The important qualification concerning the analytical value of current theory is the restrictive nature of assumptions that limit the regions of theoretical application. As specified in Section 6.5.2.3, the following computer model capabilities are beyond the range of theory application:

1. Beams may contain any level of reinforcement, to the extent of being overreinforced.
2. Cross-sectional geometry may vary along the member's length.
3. St. Venant torsion need not be dominant.
4. Member deformations are described comprehensively at all load levels from commencement of loading to failure.
5. The stress levels of all component materials are evaluated throughout the loading sequence.
6. Indeterminate analysis under any loading condition is possible.

[illegible] present form, the computer model cannot permit variation in [illegible]s-sectional geometry along its length as a rectangular concrete finite element has been used. Replacement of the rectangular element with a plane stress general quadrilateral element[38] that has the same degrees of freedom will overcome this shortcoming.

The flexibility and fully comprehensive nature of the analytical model can be a valuable asset in the design process. Although the model is not suited to direct incorporation in preliminary design, final design proposals can be checked thoroughly through the use of the analytical model after material and geometrical parameters have been chosen. Since current practice is increasingly oriented toward ultimate strength design, a certain degree of concrete cracking is often tolerated at extreme service load conditions. Evaluation of reinforcement and concrete stress levels under such conditions, though beyond theoretical capabilities, is readily achieved by the computer model which, in addition, can provide accurate estimation of structural deformations. In statically indeterminate structures, the analytical model may well represent the only reliable method of analysis.

Enthusiastic adoption of the analytical model approach must be tempered through recognition of the inherent dependence of model accuracy on quality of input specifications. Evaluation of failure load conditions is most strongly influenced by beam geometry and reinforcement strength, the effect of concrete compressive strength being more pronounced as the overreinforced condition is approached. In the accurate determination of deformations, the initial concrete modulus is the most crucial parameter, assuming that the cracking load is estimated closely through accurate monitoring of concrete tensile strength and shrinkage stresses. Invariably, concrete strength parameters can be notoriously inconsistent if care is not exercised in their evaluation. Therefore, suitability of the computer model method must be viewed within the context of input parameter accuracy, as the highly sophisticated and comprehensive nature of finite element analysis can be completely negated by erroneous input.

7.3 Summary

The principal objective of this thesis topic is the development of an analytical computer model that can analyze reinforced or prestressed concrete box girders of arbitrary cross-section for any loading combination of bending moment, torque, and shear. Of the many refined capabilities of the finite element model, prediction of deformations in the post-cracking region has been chosen as the main premise on which performance of the model is evaluated.

Linear concrete segments of the box girder are represented in the model by higher-order plane stress rectangular finite elements that are assigned twelve degrees of freedom, two translational and one rotational degree of freedom at each element node. Reinforcement is represented by one-dimensional bar elements. To enable the complete load-deformation path to be described from onset of loading through to failure, the loading sequence is an incremental, iterative one where the probability of material behavioural deviation in successive load increments is reduced by the Runge-Kutta method. Should significant material deviation be detected, the modified Newton-Rapson method restores equilibrium in an iterative procedure.

Concrete is modelled as a non-linear, anisotropic material, and following cracking, its effective shear modulus in the reformulated constitutive relationship is redefined on aggregate interlock and dowel action considerations. The important aspects of diaphragm action are treated comprehensively in the analytical model in the simulation of cross-sectional distortional stiffness, longitudinal warping restraint, and intrinsic transverse shear rigidity of box girders without diaphragms. In the absence of theoretical or experimental information, an auxiliary finite element model has been employed to investigate the longitudinal warping resistance of "thick" diaphragms.

To test the capabilities of the computer model rigorously, a literature survey was conducted to find experimental results of multi-celled box girders subjected to torsion, bending, and shear where testing was pursued to failure. No suitable reference was found, and consequently, a small experimental program was undertaken in which seven double-celled, prestressed concrete box girders, five rectangular and two trapezoidal in cross-section, were tested to failure under varying combinations of the latter three load types.

To evaluate the accuracy of the analytical model predictions, corresponding computer model and experimental results were compared in detail through examination of the respective bending moment-deflection, torque-rotation, and reinforcement stresses-load relationships. Where applicable, the model results were also assessed in light of current theoretical predictions. From an overall perspective, performance of the analytical model was very satisfactory.

7.4 Conclusion

The object of this thesis has been achieved in the development of a finite element analytical model that can analyze concrete box girders of arbitrary cross-section subjected to torsion, bending, and shear. Being afflicted by only one shortcoming that is not significant in members of common cross-sectional geometry, the computer model is a flexible, comprehensive mode of analysis whose range of application is limited by few constraints. Of the numerous material parameters that govern the model's behaviour, only the phenomenon of dowel action is ill-defined as certain aspects of its contribution to shear strength have not as yet been clarified. In the research of concrete box girder behaviour, exhausive time-consuming experimental test programs are now superseded in almost every respect by the analytical model. Considering the restrictive assumptions in current theory, the computer model greatly increases the analytical scope and design capability in this field of study.

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APPENDICES

APPENDIX A

SIGN CONVENTIONS AND SYSTEMS

APPENDIX A

SIGN CONVENTIONS AND SYSTEMS

(1) Global Axes Directions

The global x axis must be in the beam's longitudinal direction, and the global y axis in the upward vertical direction. Having chosen the x and y directions, the global z axis is defined by the right hand axis convention.

(2) Node Numbering

(a) Rectangular concrete elements

Refer to Figure A-1.

(b) Reinforcement elements

The node numbering sequence should be in the positive global axis direction. A vertical inclined bar should be numbered in the upward direction.

(c) Bond spring linkages

No predefined node numbering convention

(3) Nodal Forces and Displacements

Positive in positive global axis direction.

(4) Nodal Moments and Rotations

Positive when acting clockwise looking along global axis in positive direction.

(5) Stresses and Strains

(a) Direct stresses and strains

Tension positive, compression negative

(b) Shear stresses and strains

Positive when acting on a positive face in a positive global axis direction, or a negative face in a negative global axis direction.

(6) Mohr's Circle

Same sign convention for direct stresses and strains as above is used. However, shear stresses and strains are positive when the couple acts in the clockwise direction.

(7) Dimensional Parameter Units

Length - inches

Force - pounds wt.

Rotation - degrees (CANGLE(NEL))

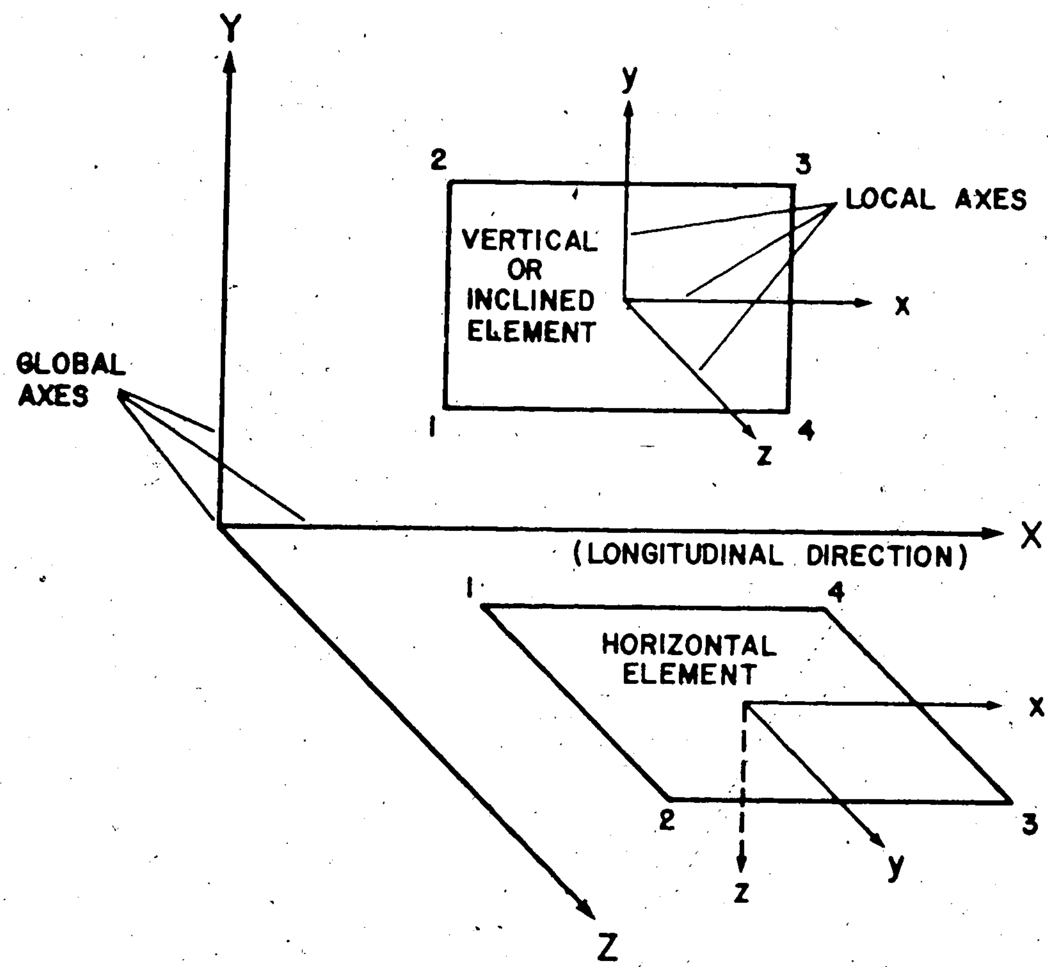

FIG. A-1 NODE NUMBERING SEQUENCE FOR RECTANGULAR CONCRETE ELEMENTS

APPENDIX B

SYMBOLIC NAMES

SYMBOLIC NAMES DESCRIPTION

THE CODES REFERRED TO IN THE DESCRIPTION COLUMN BELOW ARE DEFINED AT THE BOTTOM OF THE TABLE.

SYMBOLIC NAME	DESCRIPTION
AGMOD	AGGREGATE INTERLOCK RIGIDITY MODULUS
AL(NS3)	TOTAL STIFFNESS MATRIX ARRAY
ANGLE(NEL,I)	ANGLE IN DEGREES OF STEEL MESH LAYER I TO LONGITUDINAL DIRECTIONAL
AR(NS3)	TOTAL STIFFNESS MATRIX ARRAY
AVCSP	AVERAGE CONCRETE SPACING IN INCHES
BS(3,3)	COMPONENT ELEMENT STIFFNESS BLOCK OF CONCRETE ESTP(12,12)
CALTER(NEL)	ALTERATION INDICATOR FOR CONCRETE ELEMENT(CODE1)
CANGLE(NEL)	ANGLE OF PRINCIPAL TENSILE STRESS OR CRACK TO LONGITUDINAL DIRECTION
CAREA(NR)	CIRCUMFERENCIAL BOND AREA FOR BOND SLIP LINKAGE
CONDEV	MAXIMUM ALLOWABLE PERCENTAGE DEVIATION OF PRINCIPAL CONCRETE COMPRESSIVE STRESS FROM ITS STRESS-STRAIN CURVE
CONMOD	INITIAL MODULUS FOR CONCRETE
D(3,3)	COMBINED CONSTITUTIVE MATRIX FOR CONCRETE AND STEEL MESH
D1(NEQNS)	GENERAL DISPLACEMENT VECTOR
D2(2,NEQNS)	DISPLACEMENT ARRAY FOR CURRENT LOAD INCREMENT AND FOLLOWING HALF LOAD INCREMENT
DAGG(NEL)	COMBINED AGGREGATE INTERLOCK AND DOWEL MODULUS FOR A CRACKED CONCRETE ELEMENT
DCONC(NEL,3,3)	CONCRETE CONSTITUTIVE MATRIX
DEVCON	MAXIMUM ALLOWABLE CHANGE IN CONCRETE CONSTITUTIVE MATRIX BEFORE ALTERATION IS MADE
DEVREO	SIMILARLY FOR REINFORCEMENT
DF	DOWEL LINKAGE FAILURE DISPLACEMENT
DPHMOD(NEL)	DIAPHRAGM ELEMENT SHEAR MODULUS
DSM(3,3)	STEEL MESH CONSTITUTIVE MATRIX
DTOT(NEQNS)	TOTAL DISPLACEMENT VECTOR
ECON(NEL,3)	INCREMENTAL CENTROIDAL CONCRETE STRAINS (EX,EY,EXY)
ECULT	ULTIMATE CONCRETE STRAIN
EEPRE	PRESTRESS REINFORCEMENT YIELD STRAIN
EEREO	CONVENTIONAL REINFORCEMENT YIELD STRAIN
ELTHK(NEL)	CONCRETE ELEMENT THICKNESS
EMS(NEL,NDIRNS)	INCREMENTAL CENTROIDAL STEEL MESH STRAINS FOR ALL MESH DIRECTIONS
EMSULT	ULTIMATE STEEL MESH STRAIN
EPULT	" PRESTRESS STEEL STRAIN
EREO(NR)	INCREMENTAL REINFORCEMENT ELEMENT STRAIN
ERULT	ULTIMATE CONVENTIONAL STEEL STRAIN
ESLIP	" BOND LINKAGE STRAIN
ESTP(12,12)	CONCRETE ELEMENT STIFFNESS MATRIX
EXB1	LONGITUDINAL SHRINKAGE STRAIN IN BOTTOM FLANGE
EXS1	" " " " WEBS
EXT1	" " " " TOP "
FAGG	ULTIMATE AGGREGATE INTERLOCK STRENGTH
FC	ULTIMATE UNIAXIAL CONCRETE COMPRESSIVE STRENGTH
FDUB1	EXTERNAL DISC FILE POINTER
FDUB2	" " " "
FT	CONCRETE TENSILE STRENGTH
FUMS	STEEL MESH TENSILE STRENGTH
FUP	PRESTRESS REINFORCEMENT TENSILE STRENGTH
FUP	CONVENTIONAL REINFORCEMENT TENSILE STRENGTH
ICNODE(I)	NODE TYPE INDICATOR(CODE7)
ICON3	PRINT INDICATOR FOR CONCRETE CENTROIDAL STRESSES (CODE1)
ICONPR	" " " " PRINCIPAL CENTROIDAL STRESSES(CODE1)
ICOUNT	LOADING STATE INDICATOR 1 - INCREMENTAL LOADING STATE 2 - TOTAL LOADING STATE
ID	DEVIATION COUNTER
IDRPLN	PRINTOUT INDICATOR FOR NODAL DISPLACEMENTS(CODE1)
IDEV	MAXIMUM ALLOWABLE MATERIAL DEVIATION BEFORE THE ITERATIVE PROCESS IS INVOKED.
ILOAD	PRINTOUT INDICATOR FOR LOAD INCREMENT(CODE1)
IMESH	" " " MESH STRESSES AND STRAINS (CODE 1)
INDCEL(NEL)	CONCRETE ELEMENT INCLINATION INDICATOR(CODE2)
INDOWL(NEL)	" " DOWEL PRESENCE INDICATOR(CODE1)
INPLSZ(NEL)	" " SIZE INDICATOR(SAME INTEGER FOR SAME SIZE).
INELTY(NEL)	" " TYPE INDICATOR(CODE3)
INMESH(NEL)	" " STEEL MESH PRESENCE INDICATOR-CODE1
INRDR(NR)	REINFORCEMENT ELEMENT DIRECTION INDICATOR(CODE4)
INRTY(NR)	" " TYPE INDICATOR(CODE5)
INSZMS(NEL)	STEEL MESH SIZE INDICATOR(IDENTICAL FOR SAME SIZES)
IREO	PRINTOUT INDICATOR FOR REINFORCEMENT STRESS-STRAIN INFORMATION(CODE1)
ISOLVE	SUBROUTINE SOLVE COUNTER
ISTIF	PRINTOUT INDICATOR FOR TOTAL STIFFNESS FORMULATION INFORMATION(CODE1)
ITLOAD	PRINTOUT INDICATOR FOR TOTAL LOAD(CODE1)
KODED(NDISPL)	DIRECTION CODE FOR BOUNDARY CONDITION(CODE6)
KODER(NLOADS)	" " " EXTERNAL LOADS(CODE6)
LEN	NO. OF BYTES TRANSFERRED BETWEEN CORE AND DISC FILE
LPOINT(NBLK,2)	POINTER FOR STIFFNESS BLOCK NBLK STORED ON DISC
MAXINC	MAXIMUM NUMBER OF LOAD INCREMENTS
MAXIT	MAXIMUM NUMBER OF CONSECUTIVE ITERATIONS BEFORE PROGRAM EXECUTION IS STOPPED.
NBAND	HALF BAND WIDTH
NBLK	TOTAL STIFFNESS BLOCK NUMBER IN CORE
NCRACK(NEL)	CRACKED ELEMENT INDICATOR(CODE8)
ND(3,3)	ADJUSTED TRANSFORMED CONCRETE CONSTITUTIVE MATRIX (REAL*4)
NDIRNS	NUMBER OF STEEL MESH LAYERS
NDISPL	NO. OF BOUNDARY CONDITIONS
NDOWEL(NEL)	DOWEL LINKAGE FAILURE INDICATOR FOR CONCRETE ELEMENT(CODE1)

Variable	Description
NDREF(NEL)	DIAPHRAGM ELEMENT SIZE INDICATOR(SAME INTEGER FOR SAME TYPE AND SIZE)
NEL	CONCRETE ELEMENT NUMBER
NELCHK	NUMBER OF CONCRETE ELEMENTS TO BE CHECKED FOR CRACKING
NELD	NUMBER OF DIAPHRAGM ELEMENTS
NELSM	" " ELEMENTS WITH STEEL MESH
NELT	TOTAL NUMBER OF CONCRETE ELEMENTS
NELTOP	NUMBER OF CONCRETE ELEMENTS IN TOP FLANGE
NEQNS	TOTAL NUMBER OF EQUILIBRIUM EQUATIONS
NINCRT(I)	NUMBER OF LOAD INCREMENTS IN I TH. LOAD TYPE
NLDTY	NUMBER OF LOAD TYPES(DIFFERENT LOADING PATTERNS)
NLBB(I)	CONCRETE ELEMENT LOCATED IN BOTTOM FLANGE
NLOADS(I)	NUMBER OF INDIVIDUAL LOADS FOR LOAD TYPE I
NLTP(NEL)	CONCRETE ELEMENT LOCATED IN TOP FLANGE
NNODES	TOTAL NUMBER OF FINITE ELEMENT NODES
NODBD(NDISPL)	NODE LOCATION OF BOUNDARY CONDITION
NODEEL(NEL,4)	FOUR NODE NUMBERS OF CONCRETE ELEMENT
NODER(I,J)	NODE LOCATION I OF EXTERNAL LOAD OF LOAD TYPE J
NODES(NR,2)	REINFORCEMENT ELEMENT NR NODE NUMBERS
NR	REINFORCEMENT ELEMENT NUMBER
NREO	TOTAL NUMBER OF REINFORCEMENT ELEMENTS
NS1	INTEGER(=NBAND)
NS2	INTEGER(=NBAND*2)
NS3	INTEGER(=NBAND**2)
NUMBLK	TOTAL NUMBER OF STIFFNESS BLOCKS IN TOTAL STIFFNESS MATRIX
NUMINC	LOAD INCREMENT NUMBER
NUMREC	NUMBER OF RECORDS(LINES) IN EACH STIFFNESS BLOCK
NWTCON	DEVIATION WEIGHTING FOR A CONCRETE ELEMENT
NWTPRE	DEVIATION WEIGHTING FOR A PRESTRESS REINFORCEMENT ELEMENT
NWTREO	DEVIATION WEIGHTING FOR A CONVENTIONAL REINFORCEMENT ELEMENT
NYAGG(NEL)	FAILURE INDICATOR FOR AGGREGATE INTERLOCK MECHANISM IN A CONCRETE ELEMENT(CODE1)
NYCREO(NR)	FAILURE INDICATOR FOR CONVENTIONAL REINFORCEMENT (CODE1)
NYMESH(NEL,I)	" " " STEEL MESH DIRECTION I(CODE1)
NYPREO(NR)	" " " REINFORCEMENT ELEMENTS OTHER THAN FOR CONVENTIONAL REO.(CODE1)
P1	POISSON RATIO FOR CONCRETE IN BIAXIAL COMPRESSION
P2	" " " " " " TENSION
P3	" " " " " " TENSION AND COMPRESSION
PDTOT	NUMBER OF DISPLACEMENTS TO BE DISPLAYED ON SCREEN
PDT(I)	ROW LOCATION OF I TH. DISPLACEMENT TO BE DISPLAYED (INTEGER*4)
PERSTL(NEL,I)	PERCENTAGE STEEL MESH REINFORCEMENT IN I TH. DIRN.
PNEL(I)	I TH. CONCRETE ELEMENT NUMBER WHOSE CENTROIDAL STRESSES WILL BE DISPLAYED ON THE SCREEN.
PNELSM	NUMBER OF STEEL MESH STRESSES TO BE DISPLAYED
PNELT	NUMBER OF CONCRETE ELEMENTS WHOSE CENTROIDAL STRESSES WILL BE DISPLAYED ON THE SCREEN
PNR(I)	REINFORCEMENT ELEMENT NUMBER WHOSE STRESS-STRAIN STATE WILL BE DISPLAYED ON SCREEN
PNREO	NUMBER OF REINFORCEMENT ELEMENTS WHOSE STRESS-STRAIN STATES WILL BE DISPLAYED ON SCREEN
PREDEV	MAXIMUM ALLOWABLE PERCENTAGE DEVIATION OF PRESTRESS STRESS FROM ITS STRESS-STRAIN CURVE
PREMOD	PRESTRESS REINFORCEMENT MODULUS
PSM(I)	CONCRETE ELEMENT NUMBER WHOSE STEEL MESH STRESSES WILL BE DISPLAYED ON SCREEN
PU	UNIAXIAL POISSON RATIO
R2(2,I)	THE I TH. COL OF EXTERNAL LOAD VECTOR ROW 1 - CURRENT FULL LOAD INCREMENT ROW 2 - HALF FOLLOWING LOAD INCREMENT
RALTER(NR)	REINFORCEMENT ALTERATION INDICATOR(CODE1)
RAREA(NR)	REINFORCEMENT ELEMENT CROSS-SECTIONAL AREA
REODEV	MAXIMUM ALLOWABLE PERCENTAGE DEVIATION OF REINFORCEMENT STRESS FROM ITS STRESS-STRAIN CURVE
REOMOD	REINFORCEMENT MODULUS
REOTOT(NEQNS)	REINFORCEMENT YIELD ADJUSTMENT LOAD VECTOR
RL(2,NS1)	LOAD VECTOR USED IN SOLUTION PROCESS
RR(2,NS1)	LOAD VECTOR USED IN SOLUTION PROCESS
RSTF(NR)	CURRENT REINFORCEMENT ELEMENT STIFFNESS
RT(2,NS2)	LOAD VECTOR USED IN SOLUTION PROCESS
RTOT(NEQNS)	TOTAL LOAD VECTOR
SIGCON(NEL,3)	INCREMENTAL CENTROIDAL CONCRETE STRESSES
SIGMS(NEL,I)	INCREMENTAL STEEL MESH STRESS OF I TH. LAYER IN CONCRETE ELEMENT NUMBER NEL
SIGREO(NR)	INCREMENTAL REINFORCEMENT ELEMENT STRESS
SIGXB1	SHRINKAGE STRESS IN BOTTOM FLANGE CONCRETE
SIGXS1	" " " WEB CONCRETE
SIGXT1	" " " TOP FLANGE CONCRETE
SLPDEV	MAXIMUM ALLOWABLE PERCENTAGE DEVIATION OF BOND LINKAGE STRESS FROM ITS STRESS-DISPLACEMENT CURVE
SMSTF(NEL,3,3)	CURRENT STEEL MESH CONSTITUTIVE MATRIX
TECC(NEL)	STRAIN IN CONCRETE PRINCIPAL COMPRESSIVE STRESS DIRN
TECON(NEL,3)	TOTAL CONCRETE CENTROIDAL STRAINS
TECT(NEL)	CONCRETE STRAIN IN PRINCIPAL TENSILE DIRECTION OR ORTHOGONAL TO CRACK DIRECTION
TEMS(NEL,I)	TOTAL STEEL MESH STRAIN IN I TH. DIRN
TEREO(NR)	TOTAL REINFORCEMENT ELEMENT STRAIN
TS(NS1,NS2)	TOTAL STIFFNESS MATRIX IN CORE AT ANY ONE TIME(TWO BLOCKS)
TSGAGG(NEL)	TOTAL AGGREGATE INTERLOCK STRESS IN CONCRETE ELEMENT NUMBER NEL
TSGCC(NEL)	PRINCIPAL COMPRESSIVE CONCRETE STRESS
TSGCON(NEL,3)	TOTAL CENTROIDAL CONCRETE STRESSES
TSGCT(NEL)	PRINCIPAL TENSILE STRESS IN CONCRETE ELEMENT NEL
TSGMS(NEL,I)	TOTAL STEEL MESH STRESS IN I TH. DIRN.
TSGPRE(NR)	INITIAL PRESTRESS IN STRAND
TEPRE(NR)	INITIAL PRESTRAIN IN STRAND
TSGREO(NR)	TOTAL REINFORCEMENT ELEMENT STRESS
VALUED(I)	I TH. BOUNDARY CONDITION DISPLACEMENT
VALUER(I,J)	I TH. EXTERNAL LOAD IN LOAD TYPE J
WC	CRACK WIDTH
WIDTHC(NEL)	DOWEL SPLITTING WIDTH FOR ELEMENT
X(I)	X COORDINATE OF NODE I
Y(I)	Y " " " "
Z(I)	Z " " " "

CODE DESCRIPTION

Code	Description
CODE 1	0 - NEGATIVE 1 - POSITIVE

CODE 2	-1 - INCLINED CONCRETE ELEMENT 0 - HORIZONAL CONCRETE ELEMENT 1 - VERTICAL " "
CODE 3	1,2 - ECCLROD CONCRETE ELEMENT TYPES 3 - ACTUAL DIAPHRAGM ELEMENT 4 - EQUIVALENT DIAPHRAGM ELEMENT 5 - WARPING RESISTANCE ELEMENT
CODE 4	-1 - X GLOBAL AXIS 0 - Y " " 1 - Z " " 2 - INCLINED VERTICAL DIRECTION
CODE 5	-1 - CONVENTIONAL REINFORCEMENT 0 - PRESTRESSED " 1 - BOND LINKAGE
CODE 6	1 - X GLOBAL AXIS 2 - Y " " 3 - Z " " 4 - THETAY GLOBAL AXIS(AXIS OF ROTATION ABOUT Y AXIS) 5 - THETAZ " "
CODE 7	-1 - BOND SPRING LINKAGE NODE WITH 1 DEGREE OF FREEDOM 0 - INTERNAL NODE WITH 3 DEGREES OF FREEDOM 1 - CORNER " " 5 " " " 2 - SPECIAL DIAPHRAGM NODE WITH 4 DEGREES OF FREEDOM(ALL DEGREES OF FREEDOM EXCEPT THETAY) 3 - INTERNAL DIAPHRAGM NODE - 2 TRANSLATIONAL DEGREES OF FREEDOM. THIS NODE DOES NOT ADJOIN CONCRETE BOX GIRDER WALL ELEMENTS.
CODE 8	-1 - JUST CRACKED IN CURRENT LOAD INCREMENT 0 - UNCRACKED 1 - CRACKED

FILE

APPENDIX C

MAIN PROGRAM LISTING

```
C*************************************************************************
C*************************************************************************
C                         MAIN PROGRAM
C         THE FOLLOWING ROUTINE IS THE MAIN PROGRAM OF THE COMPUTER MODEL
C       THAT ANALYSES THE BEHAVIOUR OF A PRECAST,PRESTRESSED CONCRETE BOX
C       GIRDER SUBJECTED TO TORSION,BENDING, AND SHEAR. COMPLETE (STRESS,
C       DEFORMATION) DATA IS OUTPUTTED FOR EACH PROGRESSIVE INCREMENT OF
C       LOAD, THUS TRACING THE BEAM'S BEHAVIOUR THROUGH THE PRE-CRACKING
C       AND POST-CRACKING STAGES TO ULTIMATE FAILURE.
C*************************************************************************
      COMMON/BLOCK1/NEQNS,NBAND,NBLK,NUMINC,ICOUNT
      COMMON/BLOCK2/NEL,NELT,NELCHK,NELD,INDCEL(185),INELSZ(185),
     1INDOUL(185),NIDTHC(185),CANGLE(185),CALTER(185),
     2INELTY(215),ELTHN(215),NODEEL(215,4),NDPEP(215),DPENOD(215)
      COMMON/BLOCK3/NELSM,INMESH(185),INSEMS(185),NDIRNS,PERSIL(185,4),
     1ANGLE(185,4),SMSTP(185,3,3),SMRSH(185,1)
      COMMON/BLOCK4/NN,NREO,NODES(190,2),INRDN(190),INRTY(190),
     1NALTER(190),EAREA(190),CAREA(190),RSTF(190),PSGPRE(190),PEPRE(190)
      COMMON/BLOCK6/CONNOD,REONOD,PRENOD,SLPNOD,SMSNOD,FC,FY,
     1FAGG,FUR,FUP,FUNS,DP,ESLIP,ECULT,ERULT,EPULT,EMSULT,P1,P2,P3,P4,
     2PU,ERREO,EPPRE,CONDEV,DEVCON,DEVREO,PREDEV,SLPDEV,IDEV,AVCSP,RELAX
      COMMON/BLOCK8/NDISPL,NOORD(30),KODED(30),VALUED(30),D1(750)
      COMMON/BLOCK9/LPOINT(20,2),FDUB1,FDUB2,NUMREC,
     1NUMBLK,ISOLVE,LEN
      COMMON/BLCK10/ESTP(12,12),BS(3,3),B1(3,12,9),B2(3,12,9),D(3,3),
     1DCONC(185,3,3),DCRACK(185),DAGG(185)
      COMMON/BLCK11/RTOT(750),RTOTOT(750),DTOT(750),R2(2,753),D2(2,750)
      COMMON/BLCK12/SIGCON(185,3),ECON(185,3),SIGMS(185,4),EMS(185,4),
     1TSGCON(185,3),TECON(185,3),TSGCC(185),TSGCT(185),TSGA3G(185),
     2TECC(185),TECT(185),TSGMS(185,4),TEMS(185,4),SIGREO(190),
     3EREO(190),TSGREO(190),TEREO(190)
      COMMON/BLCK13/NYMESH(185,4),NDOUEL(185),NCRACK(185),NFAGG(185),
     1NYREO(190),NYCREO(190)
      COMMON/BLCK14/IDEFLN,ICON3,ICONPR,INMSH,IREO,ILOAD,ITLOAD,ISTIP,
     1MAXIT,NYTCON,NYTREO,NYTPRE
      DIMENSION TS(62,124),AL(3844),AR(3844),A(185,14),B(190,2),
     1RT(2,166),RL(2,62),RR(2,62)
      EQUIVALENCE (TS(1,1),AL(1)),(TS(1,63),AR(1)),(SIGCON(1,1),A(1,1)),
     1(SIGREO(1),B(1,1)),(RT(1,1),RL(1,1)),(RT(1,63),RR(1,1))
      INTEGER*4 INFO(4),FDUB1,FDUB2,CALTER,NALTER,ITIME
      INTEGER*2 LEN
C     TWO DISC FILES TO BE USED IN STIFFNESS BLOCK MANIPULATION ARE
C     OBTAINED.
      EXTERNAL GETFD
      CALL SETDSN(1,'-FILE1 ',FDUB1,&200)
      REWIND 1
      EXTERNAL GETFD
      CALL SETDSN(2,'-FILE2 ',FDUB2,&210)
      REWIND 2
C     READING IN OF ALL INPUT
      CALL TIME(1,0,ITIME)
      COSTMY=COST(0)
      WRITE (6,260) ITIME,COSTMY
      CALL READIN
      CALL TIME(1,0,ITIME)
      COSTMY=COST(0)
      WRITE (6,270) ITIME,COSTMY
C     INITIALISING OF ALL APPROPRIATE VECTORS AND ARRAYS
      CALL PRESET
C     DEFINITION OF PARAMETERS FOR IN-CORE STIFFNESS MATRICES
      NUMINC=0.
      NS1=NBAND
      NS2=NBAND*2
      NS3=NBAND**2
      NS4=NS1+1
C     CALCULATION OF INITIAL STRUCTURAL STIFFNESS MATRIX OF BEAM
      CALL TOTSTF(NS1,NS2,NS3,TS,AL,AR)
      CALL TIME(1,0,ITIME)
      COSTMY=COST(0)
      WRITE (6,290) ITIME,COSTMY
C     PROGRESSIVE APPLICATION OF LOAD INCREMENTS
      NUMINC=0
      MAXINC=1
   20 ICOUNT=1
      NC=0
      NCI=0
      NUMINC=NUMINC+1
      IF(NUMINC.GT.MAXINC) GO TO 220
      WRITE (6,230) NUMINC
      WRITE (7,230) NUMINC
C     CALCULATION OF LOAD VECTOR FOR INCREMENT NUMBER NUMINC.
      CALL LOAD(MAXINC)
C     CALCULATION OF TOTAL LOAD VECTOR.
      DO 22 I=1,NEQNS
      RTOT(I)=RTOT(I)+R2(1,I)
   22 CONTINUE
      IF(NUMINC.GT.1) GO TO 24
C     DISPLACEMENTS FOR FIRST HALF LOAD VECTOR ARE CALCULATED.
      DO 23 I=1,NEQNS
      R2(2,I)=R2(1,I)/2.0
   23 CONTINUE
C     SET OF STIFFNESS EQUATIONS IS SOLVED FOR DISPLACEMENTS.
      CALL SOLVE(NS1,NS2,NS3,NS4,TS,AL,AR,RT,RL,RR)
      CALL LOAD(MAXINC)
C     USING THE KUTTA-RUNGE METHOD, THE BEAM STIFFNESS IS ADJUSTED
C     BEFORE THE DISPLACEMENT VECTOR FOR THE CURRENT LOAD INCREMENT IS
C     CALCULATED
   24 DO 25 I=1,NEQNS
      D1(I)=D2(2,I)
   25 CONTINUE
      IF(NUMINC.GT.1) GO TO 30
      CALL TIME(1,0,ITIME)
      COSTMY=COST(0)
      WRITE (6,300) ITIME,COSTMY
C     ALL STRESS LEVELS ARE CALCULATED FOR CURRENT HALF-LOAD.
   30 CALL STRESS(NC)
      IF(NUMINC.GT.1) GO TO 40
      CALL TIME(1,0,ITIME)
      COSTMY=COST(0)
      WRITE (6,310) ITIME,COSTMY
C     MATERIAL STIFFNESSES ARE ADJUSTED USING THE RUNGE-KUTTA METHOD.
   40 CALL KUTTA(NS1,NS2,NS3,TS,AL,AR)
      IF(NUMINC.GT.1) GO TO 50
      CALL TIME(1,0,ITIME)
      COSTMY=COST(0)
      WRITE (6,320) ITIME,COSTMY
C     (STRESS,STRAIN) STATE OF ALL BEAM COMPONENTS IS NOW CALCULATED
C     FOR FULL LOAD INCREMENT USING ADJUSTED STIFFNESS MATRIX
C     SOLUTION OF STRUCTURE DISPLACEMENTS.
   50 CALL SOLVE(NS1,NS2,NS3,NS4,TS,AL,AR,RT,RL,RR)
      DO 55 I=1,NEQNS
      D1(I)=D2(1,I)
      IF(ICOUNT.EQ.1) GO TO 55
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      DTOT(I)=D2(1,I)
   55 CONTINUE
C     CALCULATION OF ALL COMPONENT STRESSES FOR FULL LOAD VECTOR.
      CALL STRESS(NC)
C     FAILURE CHECK FOR BEAM
      CALL FAIL
C     CHECK TO DETECT ANY FURTHER CRACKING
      NCJ=0
      CALL TIME(1,0,ITIME)
      COSTHY=COST(0)
      WRITE (6,330) ITIME,COSTHY
      WRITE (7,330) ITIME,COSTHY
      DO 60 I=1,NELCH
      IF(NCRACK(I).EQ.1) GO TO 60
      SIGMAT=FT
      IF(TSGCC(I).GE.0.0) GO TO 58
      SIGMAT=FT*(1.0-TSGCC(I)/FC)
   58 IF(TSGCT(I).LT.SIGMAT) GO TO 60
      NCRACK(I)=-1
      NCJ=NCJ+1
      NCI=NCI+1
      WRITE (6,340) I
      WRITE (7,340) I
   60 CONTINUE
      CALL TIME(1,0,ITIME)
C     CHECK TO DETERMINE WHETHER ANY SIGNIFICANT DEVIATIONS IN MATERIAL
C     PROPERTITIES HAVE OCCURRED.
      CALL DEVIAT(ID)
      IF(ID.LT.IDEV.AND.NCJ.LT.2) GO TO 100
C     ADJUSTMENT OF STRUCTURAL STIFFNESS MATRIX TO PERMIT TOTAL LOAD
C     ITERATION FOR RESTORING EQUILIBRIUM OF ALL STRUCTURAL ELEMENTS
C     IF: SIGNIFICANT DEVIATION HAS OCCURRED.
      CALL TIME(1,0,ITIME)
      COSTHY=COST(0)
      WRITE (6,360) ITIME,COSTHY
      NC=NC+1
      IF(NC.GT.NAXIT) GO TO 380
      WRITE (6,365) NC
      WRITE (7,365) NC
      ICOUNT=2
      IF(NC.GT.1) GO TO 78
C     FOR THE FIRST ITERATION, TOTAL STRESS CONDITION IS
C     CALCULATED
      DO 65 J=1,3
      DO 65 I=1,NELT
      TSGCON(I,J)=TSGCON(I,J)+SIGCON(I,J)
      TECON(I,J)=TECON(I,J)+ECON(I,J)
   65 CONTINUE
      DO 70 J=1,NDIRNS
      DO 70 I=1,NELT
      IF(INRESH(I).EQ.0) GO TO 70
      TSGHS(I,J)=TSGHS(I,J)+SIGHS(I,J)
      TEHS(I,J)=TEHS(I,J)+EHS(I,J)
   70 CONTINUE
      DO 75 I=1,NREO
      TSGREO(I)=TSGREO(I)+SIGREO(I)
      TEREO(I)=TEREO(I)+EREO(I)
   75 CONTINUE
C     REDOING OF REINFORCEMENT RESTRAINT LOAD VECTOR
   78 DO 76 I=1,NEQNS
      HEOTOT(I)=0.0
   76 CONTINUE
C     MODIFICATION OF TOTAL STIFFNESS MATRIX
      CALL KUTTA(NS1,NS2,NS3,TS,AL,AR)
      CALL TIME(1,0,ITIME)
      COSTHY=COST(0)
      WRITE (6,370) ITIME,COSTHY
C     FIRST COLUMN OF LOAD VECTOR B2(2,NEQNS) IS ALTERED FROM CURRENT
C     LOAD INCREMENT TO CURRENT TOTAL LOAD
      DO 80 I=1,NEQNS
      B2(1,I)=BTOT(I)+HEOTOT(I)
   80 CONTINUE
      GO TO 50
  100 IF(ICOUNT.EQ.2) GO TO 140
C     CALCULATION OF TOTAL STRESS CONDITION AND NODAL DEFLECTIONS OF ALL
C     STRUCTURAL ELEMENTS AFTER APPLICATION OF LOAD INCREMENT
  110 DO 112 I=1,NEQNS
      DTOT(I)=DTOT(I)+D2(1,I)
  112 CONTINUE
      DO 115 J=1,3
      DO 115 I=1,NELT
      TSGCON(I,J)=TSGCON(I,J)+SIGCON(I,J)
      TECON(I,J)=TECON(I,J)+ECON(I,J)
  115 CONTINUE
      DO 120 J=1,NDIRNS
      DO 120 I=1,NELT
      IF(INRESH(I).EQ.0) GO TO 120
      TSGHS(I,J)=TSGHS(I,J)+SIGHS(I,J)
      TEHS(I,J)=TEHS(I,J)+EHS(I,J)
  120 CONTINUE
      DO 125 I=1,NREO
      TSGREO(I)=TSGREO(I)+SIGREO(I)
      TEREO(I)=TEREO(I)+EREO(I)
  125 CONTINUE
C     PRINTOUT OF ALL RELEVANT DATA
  140 CALL OUTPUT(NCI)
C     FAILURE CHECK FOR BEAM
      CALL FAIL
      IF(ICOUNT.EQ.1) GO TO 20
C     RECONSTITUTION OF TOTAL STIFFNESS MATRIX IF NON-LINEAR ITERATIVE
C     PROCESS WAS USED IN LAST LOAD INCREMENT. CHANGE IS NECESSARY SINCE
C     THE INCREMENTAL PROCESS IS TO BE RESUMED.
      DO 150 J=1,14
      DO 150 I=1,NELT
      A(I,J)=0.0
  150 CONTINUE
      DO 155 J=1,2
      DO 155 I=1,NREO
      B(I,J)=0.0
  155 CONTINUE
      ICOUNT=1
      CALL KUTTA(NS1,NS2,NS3,TS,AL,AR)
C     CALCULATION OF DISPLACEMENT VECTOR OF FOLLOWING HALF L
C     INCREMENT USING RECONSTITUTED TOTAL STIFFNESS MATRIX
      CALL LOAD(NAXINC)
      DO 160 I=1,NEQNS
      B2(1,I)=0.0
  160 CONTINUE
      CALL SOLVE(NS1,NS2,NS3,NS4,TS,AL,AR,BT,BL,BB)
      GO TO 20
```

```
C     FORMAT STATEMENTS
  200 WRITE (6,205)
  205 FORMAT(///' ** ERROR IN FIRST SYSTEM SUBROUTINE SETDSN IN MAIN PRO
     1GRAMME **')
      STOP
  210 WRITE (6,215)
  215 FORMAT(///' ** ERROR IN SECOND SYSTEM SUBROUTINE SETDSN IN MAIN PR
     1OGRAMME **')
      STOP
  220 WRITE (6,225)
      WRITE (7,225)
  225 FORMAT(//' ******************************************',/,
     1' ** FULL LOAD HAS BEEN APPLIED TO BEAM **',/,
     2' ******************************************'//)
      STOP
  230 FORMAT(////' *******************************',/,
     1' *  LOAD INCREMENT NUMBER ',I2,'  *',/,
     2' *******************************')
  260 FORMAT(/' BEFORE DATA IS READ IN, CPU TIME =',I8,
     1'  PROGRAMME COST =',F7.2)
  270 FORMAT(/' AFTER DATA IS READ IN, CPU TIME =',I8,
     1'  PROGRAMME COST =',F7.2)
  290 FORMAT(/' AFTER TOTAL STIFFNESS MATRIX HAS BEEN ASSEMBLED AND CERT
     1AIN MATRICES PRESET TO ZERO,',/,' CPU TIME =',I8,
     2'  PROGRAMME COST =',F7.2)
  300 FORMAT(/' BEFORE ROUTINE STRESS IS CALLED IN FIRST LOAD INCREMENT
     1,',/,' CPU TIME =',I8,'  PROGRAMME COST =',F7.2)
  310 FORMAT(/' AFTER ROUTINE STRESS HAS BEEN CALLED AND BEFORE ROUTINE
     1KUTTA IS CALLED IN THE FIRST LOAD INCREMENT,',/,' CPU TIME =',
     2I8,'  PROGRAMME COST =',F7.2)
  320 FORMAT(/' AFTER ROUTINE KUTTA HAS BEEN CALLED IN FIRST LOAD INCREM
     1ENT,',/,' CPU TIME =',I8,'  PROGRAMME COST =',F7.2)
  330 FORMAT(/' BEFORE BEAM IS CHECKED FOR FURTHER CRACKING,',/,
     1' CPU TIME =',I8,'  PROGRAMME COST =',F7.2)
  340 FORMAT(//' ******** CONCRETE ELEMENT NUMBER ',I3,
     1' HAS JUST CRACKED ********')
  360 FORMAT(/' BEFORE STIFFNESS ADJUSTMENTS HAVE BEEN MADE,',/,
     1' CPU TIME =',I8,'  PROGRAMME COST =',F7.2)
  365 FORMAT(/' **** ITERATION CYCLE NO.',I3,' ****'/)
  370 FORMAT(/' AFTER STIFFNESS ADJUSTMENTS HAVE BEEN MADE,',/,
     1' CPU TIME =',I8,'  PROGRAMME COST =',F7.2)
  380 WRITE (6,385) NC
      WRITE (7,385) NC
  385 FORMAT(//' ***** NUMBER OF ITERATIONS IS EXCESSIVE *****',
     1/,' NUMBER OF ITERATIONS =',I2)
      STOP
      END
```

APPENDIX D

SUBROUTINES LISTING

```
      SUBROUTINE WARP(TS,NS1,NS2)
C***********************************************************************
C       THIS SUBROUTINE CALCULATES AND ADDS INTO THE TOTAL STIFFNESS
C     MATRIX, THE WARPING RESTRAINT OF THE DIAPHRAGMS.
C***********************************************************************
C
      COMMON/BLOCK2/NEL,NELT,NELCHK,NELD,INDCEL(185),INELSZ(185),
     1INDOWL(185),WIDTHC(185),CANGLE(185),CALTER(155),
     2INELTY(215),ELTHN(215),NODEEL(215,4),NDREF(215),DPHMOD(215)
      COMMON/BLOCK5/NNODES,ICNODE(220),X(220),Y(220),Z(220)
      COMMON/BLOCK6/CONMOD,REMOD,PREMOD,SLPMOD,SHSMOD,PC,PT,
     1PAGG,PUR,PUP,PUMS,DF,ESLIP,ECULT,EPULT,EPOLT,RMSULT,P1,P2,P3,P4,
     2PU,ERREO,EXPRE,CONDEV,DEVCON,DEVREO,PREDEV,SLPDEV,IDEV,AVCSP,RELAX
      DIMENSION TS(NS1,NS2),XD(4),YD(4)
C     CALCULATION OF DIAGONAL LENGTH. A TRAPEZOIDAL ELEMENT WILL BE
C     TREATED AS A RECTANGLE OF THE SAME AREA.
      DO 5 I=1,4
      XD(I)=Z(NODEEL(NEL,I))
      YD(I)=Y(NODEEL(NEL,I))
    5 CONTINUE
      XAV=(XD(2)+XD(3)-XD(1)-XD(4))*.5
      YAV=(YD(3)+YD(4)-YD(1)-YD(2))*.5
      DIAG=SQRT(XAV**2+YAV**2)
C     CALCULATION OF WARPING AND ADDITION INTO TOTAL STIFFNESS MATRIX
C     ELTHN(NEL) IS THE EQUIVALENT CLASSICAL DIAPHRAGH THICKNESS
      STIF=CONMOD*ELTHN(NEL)**3/(3.0*(1.0+PU)*DIAG**2)
      DO 10 I=1,4
      II=NODEEL(NEL,I)
      IF(ICNODE(II).EQ.3) GO TO 10
      DO 10 J=1,4
      JJ=NODEEL(NEL,J)
      CALL LOCATE(II,JJ,NROW,NCOL)
      IF(NROW.LT.NCOL) GO TO 10
      NR=NROW-NCOL+1
      NN=I+J
      STP=STIF*(-1)**NN
      TS(NR,NCOL)=TS(NR,NCOL)+STP
   10 CONTINUE
      RETURN
      END
      SUBROUTINE DIAPHR(TS,NS1,NS2)
C***********************************************************************
C       THIS SUBROUTINE CALCULATES AND ADDS INTO THE TOTAL STIFFNESS
C     MATRIX, THE STIFFNESS OF BOTH ACTUAL AND PSEUDO DIAPHRAGHS
C***********************************************************************
C
      COMMON/BLOCK2/NEL,NELT,NELCHK,NELD,INDCEL(185),INELSZ(185),
     1INDOWL(185),WIDTHC(185),CANGLE(185),CALTER(155),
     2INELTY(215),ELTHN(215),NODEEL(215,4),NDREF(215),DPHMOD(215)
      DIMENSION B2(2,2),DP(8,8),TS(NS1,NS2)
      CALL ISOBLS(DP)
C     PARTITIONING OF ELEMENT STIFFNESS INTO (2,2) BLOCKS
      DO 15 I=1,4
      II=NODEEL(NEL,I)
      DO 15 J=1,4
      JJ=NODEEL(NEL,J)
      DO 10 K=1,2
      DO 10 L=1,2
      LL=(L-1)*4+I
      KK=(K-1)*4+J
      B2(L,K)=DP(LL,KK)*DPHMOD(NEL)*ELTHN(NEL)
   10 CONTINUE
      CALL DPHADD(II,JJ,B2,TS,NS1,NS2)
   15 CONTINUE
      RETURN
      END
      SUBROUTINE DPHADD(II,JJ,B2,TS,NS1,NS2)
C***********************************************************************
C       THIS SUBROUTINE ADDS BLOCK B2(2,2) INTO TOAL STIFFNESS MATRIX
C***********************************************************************
C
      DIMENSION B2(2,2),TS(NS1,NS2),NR(2),NC(2)
      CALL LOCATE(II,JJ,NROW,NCOL)
C     LOCAL X AXIS CORRESPONDS TO GLOBAL Z AXIS, AND LOCAL Y AXIS
C     CORRESPONDS TO GLOBAL Y AXIS.
      DO 5 I=1,2
      NR(I)=NROW+3-I
      NC(I)=NCOL+3-I
    5 CONTINUE
C     BLOCK(2,2) IS TRANSFORMED IF EITHER NODE IS COMMON TO AN
C     INCLINED ELEMENT.
      CALL TSFORM(II,JJ,B2)
      DO 10 J=1,2
      L=NC(J)
      DO 10 I=1,2
      K=NR(I)
      IF(K.LT.L) GO TO 10
      KK=K-L+1
      TS(KK,L)=TS(KK,L)+B2(I,J)
   10 CONTINUE
      RETURN
      END
      SUBROUTINE TSFORM(II,JJ,B2)
C***********************************************************************
C       THIS SUBROUTINE TRANSFORMS THE DIAPHRAGM STIFFNESS BLOCK B2(2,2)
C     IF EITHER BLOCK NODE IS ALSO AN INCLINED ELEMENT NODE.
C***********************************************************************
C
      DIMENSION LT(2,2),B3(2,2),NODE(2),NELNO(2),B2(2,2)
      INTEGER*4 CALTER
      REAL*4 LT
      NODE(1)=II
      NODE(2)=JJ
      CALL NELLOC(NODE,NELNO)
      IF(NELNO(1).EQ.0.AND.NELNO(2).EQ.0) RETURN
C     AT LEAST ONE BLOCK NODE IS AN INCLINED ELEMENT NODE
      DO 5 J=1,2
      DO 5 I=1,2
      B3(I,J)=0.0
    5 CONTINUE
      IF(NELNO(2).EQ.0) GO TO 20
      NL=NELNO(2)
      CALL TSMAT(LT,NL)
      DO 10 J=1,2
      DO 10 I=1,2
```

```
$SIGNON TAYL RIBBON=CA PRINT=TN
JOB-TYPE=BATCH, PRIO=NORMAL, CLASS=INTERNAL/TEACHING,RESEARCH
**LAST SIGNON WAS: 11:29:54
 USER "TAYL" SIGNED ON AT  12:42:08 ON THU JUN 30/77
$LIST FILE50(116,930)
             DO 10 K=1,2
             B3(I,J)=B3(I,J)+B2(I,K)*LT(K,J)
   10        CONTINUE
             DO 15 J=1,2
             DO 15 I=1,2
             B2(I,J)=B3(I,J)
             B3(I,J)=0.0
   15        CONTINUE
   20        IF(NELNO(1).EQ.0)  RETURN
             NL=NELNO(1)
             CALL TSHAT(LT,NL)
             DO 25 J=1,2
             DO 25 I=1,2
             DO 25 K=1,2
             B3(I,J)=B3(I,J)+LT(K,I)*B2(K,J)
   25        CONTINUE
             DO 30 J=1,2
             DO 30 I=1,2
             B2(I,J)=B3(I,J)
             B3(I,J)=0.0
   30        CONTINUE
             RETURN
             END
             SUBROUTINE NELLOC(NODE,NELNO)
C***********************************************************************
C       THIS SUBROUTINE SEARCHES FOR AN INCLINED ELEMENT THAT CONTAINS
C       THE BLOCK NODE NUMBERS.
C       IF THE SEARCH IS UNSUCCESSFUL, A ZERO ELEMENT NUMBER IS RETURNED.
C***********************************************************************
C
             COMMON/BLOCK2/NEL,NELT,NELCHK,NELD,INDCEL(185),INELSZ(185),
            1INDOWL(185),WIDTHC(185),CANGLE(185),CALTER(185),
            2INELTY(215),ELTHN(215),NODEEL(215,4),NDREF(215),DPNHOD(215)
             COMMON/BLOCK5/NNODES,ICNODE(220),X(220),Y(220),Z(220)
             DIMENSION NODE(2),NELNO(2)
             INTEGER*4 CALTER
             NELNO(1)=0
             NELNO(2)=0
             DO 10 I=1,2
             IF(ICNODE(NODE(I)).EQ.1) GO TO 10
             DO 8 J=1,NELT
             IF(INDCEL(J).NE.-1)  GO TO 8
             DO 5 K=1,4
             IF(NODEEL(J,K).NE.NODE(I)) GO TO 5
             NELNO(I)=J
             GO TO 10
    5        CONTINUE
    8        CONTINUE
   10        CONTINUE
             RETURN
             END
             SUBROUTINE TSHAT(LT,NL)
C***********************************************************************
C       THIS SUBROUTINE CALCULATES THE TRANSFORMATION MATRIX LT(2,2) FOR
C       A (2*2) DIAPHRAGM STIFFNESS BLOCK, RECOGNIZING THE ROW AND COLUMN
C       NUMBERING IS IN THE LOCAL Z-Y ORDER.
C***********************************************************************
C
             COMMON/BLOCK2/NEL,NELT,NELCHK,NELD,INDCEL(185),INELSZ(185),
            1INDOWL(185),WIDTHC(185),CANGLE(185),CALTER(185),
            2INELTY(215),ELTHN(215),NODEEL(215,4),NDREF(215),DPNHOD(215)
             COMMON/BLOCK5/NNODES,ICNODE(220),X(220),Y(220),Z(220)
             DIMENSION LT(2,2)
             REAL*4 LT
             B=SQRT((Y(NODEEL(NL,2))-Y(NODEEL(NL,1)))**2+(Z(NODEEL(NL,2))
            1-Z(NODEEL(NL,1)))**2)
             C=(Y(NODEEL(NL,2))-Y(NODEEL(NL,1)))/B
             S=(Z(NODEEL(NL,2))-Z(NODEEL(NL,1)))/B
             LT(1,1)=C
             LT(2,1)=S
             LT(1,2)=-S
             LT(2,2)=C
             RETURN
             END
             SUBROUTINE ISOBLS(DP)
C***********************************************************************
C       THIS SUBROUTINE CALCULATES THE STIFFNESS OF AN ISOPARAMETRIC
C       BI-LINEAR SERENDIPITY ELEMENT.
C***********************************************************************
C
             COMMON/BLOCK2/NEL,NELT,NELCHK,NELD,INDCEL(185),INELSZ(185),
            1INDOWL(185),WIDTHC(185),CANGLE(185),CALTER(185),
            2INELTY(215),ELTHN(215),NODEEL(215,4),NDREF(215),DPNHOD(215)
             COMMON/BLOCK5/NNODES,ICNODE(220),X(220),Y(220),Z(220)
             COMMON/BLCK14/IDEFLN,ICON3,ICONPR,INTEN,IREO,ILOAD,ITLOAD,ISTIF,
            1NAXIT,NNTCON,NNTREO,NNTPRE
             DIMENSION XX(4),YY(4),B(3,8),AX(4),AY(4),C(3,3),A(3,8),W(2),G(2),
            1DP(8,8)
             REAL*4 JOB(2,2),JOBINV(2,2)
             DATA W/1.0,1.0/,G/-.57735027,.57735027/
C            DEFINITION OF NODAL COORDINATES
             DO 10 I=1,4
             XX(I)=Z(NODEEL(NEL,I))
             YY(I)=Y(NODEEL(NEL,I))
   10        CONTINUE
             DO 12 J=1,8
             DO 12 I=1,8
             DP(I,J)=0.0
   12        CONTINUE
             DO 50 M=1,2
             DO 50 N=1,2
             XI=G(M)
             ETA=G(N)
C            DEFINITION OF JOCOBIAN MATRIX
             JOB(1,1)=(XX(2)+XX(3)-XX(1)-XX(4)+ETA*(XX(1)+XX(3)-XX(2)
            1-XX(4)))*.25
             JOB(2,1)=(XX(3)+XX(4)-XX(1)-XX(2)+XI*(XX(1)+XX(3)-XX(2)
            1-XX(4)))*.25
             JOB(1,2)=(YY(2)+YY(3)-YY(1)-YY(4)+ETA*(YY(1)+YY(3)-YY(2)
            1-YY(4)))*.25
             JOB(2,2)=(YY(3)+YY(4)-YY(1)-YY(2)+XI*(YY(1)+YY(3)-YY(2)
            1-YY(4)))*.25
C            DEFINITION OF INVERSE OF JOCOBIAN MATRIX
             DETJ=JOB(1,1)*JOB(2,2)-JOB(1,2)*JOB(2,1)
             JOBINV(1,1)=JOB(2,2)/DETJ
```

```
      JOBINV(2,1)=-JOB(2,1)/DETJ
      JOBINV(1,2)=-JOB(1,2)/DETJ
      JOBINV(2,2)=JOB(1,1)/DETJ
C     DEFINITION OF MATRIX B(3,8)
      AX(1)=(JOBINV(1,1)*(ETA-1.0)+JOBINV(1,2)*(ZI-1.0))*.25
      AX(2)=(JOBINV(1,1)*(1.0-ETA)+JOBINV(1,2)*(-ZI-1.0))*.25
      AX(3)=(JOBINV(1,1)*(1.0+ETA)+JOBINV(1,2)*(ZI+1.0))*.25
      AX(4)=(JOBINV(1,1)*(-1.0-ETA)+JOBINV(1,2)*(1.0-ZI))*.25
      AY(1)=(JOBINV(2,1)*(ETA-1.0)+JOBINV(2,2)*(ZI-1.0))*.25
      AY(2)=(JOBINV(2,1)*(1.0-ETA)+JOBINV(2,2)*(-1.0-ZI))*.25
      AY(3)=(JOBINV(2,1)*(1.0+ETA)+JOBINV(2,2)*(1.0+ZI))*.25
      AY(4)=(JOBINV(2,1)*(-1.0-ETA)+JOBINV(2,2)*(1.0-ZI))*.25
      DO 15 I=1,4
      J=4+I
      B(1,I)=AX(I)
      B(1,J)=0.0
      B(2,I)=0.0
      B(2,J)=AY(I)
      B(3,I)=AY(I)
      B(3,J)=AX(I)
   15 CONTINUE
C     DEFINITION OF UNIT CONSTITUTIVE MATRIX. THE INDIVIDUAL STIFFNESS TERMS
C     WILL BE MULTIPLIED BY THE ACTUAL SHEAR MODULUS IN ROUTINE DPMSTF.
      DO 20 J=1,3
      DO 20 I=1,3
      C(I,J)=0.0
   20 CONTINUE
      IF(INELTY(NEL).NE.3) GO TO 23
C     THE FULL CONSTITUTIVE MATRIX IS USED FOR ACTUAL DIAPHRAGMS
C     (INELTY(NEL)=3), WHEREAS ONLY THE UNIT SHEAR TERM IS INCLUDED
C     FOR PSEUDO DIAPHRAGMS.
      C(1,1)=2.5
      C(2,1)=.5
      C(1,2)=.5
      C(2,2)=2.5
   23 C(3,3)=1.0
C     CALCULATION OF ELEMENT STIFFNESS CONTRIBUTION
      DO 25 J=1,8
      DO 25 I=1,3
      A(I,J)=0.0
   25 CONTINUE
      DO 35 I=1,3
      DO 35 J=1,8
      DO 35 K=1,3
      A(I,J)=A(I,J)+C(I,K)*B(K,J)
   35 CONTINUE
      DO 45 I=1,8
      DO 45 J=1,8
      DO 45 K=1,3
      DP(I,J)=DP(I,J)+B(K,I)*A(K,J)*DETJ*W(M)*W(N)
   45 CONTINUE
   50 CONTINUE
      IF(ISTIF.EQ.0) RETURN
      WRITE (6,100) NEL
  100 FORMAT(//' STIFFNESS MATRIX FOR DIAPHRAGM ELEMENT NO.',I3,' :')
      DO 120 I=1,8
      WRITE (6,110) (DP(I,J),J=1,8)
  110 FORMAT(8F11.5)
  120 CONTINUE
      RETURN
      END
      SUBROUTINE SHRMOD(WC)
C*************************************************************************
C      THIS SUBROUTINE CALCULATES THE AGGREGATE INTERLOCK (LBS/SQ. INCH)
C     DEVELOPED ALONG A CRACK OF WIDTH WC
C*************************************************************************
C
      COMMON/BLOCK1/NEQNS,NBAND,NBLK,NUMINC,ICOUNT
      COMMON/BLOCK2/NEL,NELT,NELCHK,NELD,INDCEL(185),INELSX(185),
     1INDOWL(185),WIDTHC(185),CANGLE(185),CALTER(185),
     2INELTY(215),ELTHN(215),NODEEL(215,4),NDREF(215),DPMMOD(215)
      COMMON/BLOCK5/NNODES,ICNODE(220),X(220),Y(220),Z(220)
      COMMON/BLOCK6/CONMOD,REOMOD,PREMOD,SLPMOD,SHSMOD,FC,FT,
     1PAGG,PUR,PUP,PUMS,DF,ESLIP,ECULT,ERULT,EPULT,EMSULT,P1,P2,P3,P4,
     2PU,ERREO,EEPRE,CONDEV,DEVCON,DEVREO,PREDEV,SLPDEV,IDEV,AVCSP,RELAX
      COMMON/BLCK10/ESTF(12,12),BS(3,3),B1(3,12,9),B2(3,12,9),D(3,3),
     1DCONC(185,3,3),DCRACK(185),DAGG(185)
      COMMON/BLCK13/NYMESH(185,4),NDOWEL(185),NCRACK(185),NYAGG(185),
     1NYREO(190),NYCREO(190)
      INTEGER*4 CALTER
      IF(NYAGG(NEL).EQ.1) RETURN
C     IF CONCRETE CRACK CLOSES SOON AFTER TRANSFER, UNCRACKED STRENGTH
C     IS RECOVERED.
      IF(WC.LT.0.0) GO TO 15
      AA=SQRT(-FC/5000.0)
      WI=1.0/WC
      AGMOD=57.0*AA*WI*SQRT(WI)
      G=CONMOD/((1.0+PU)*2.0)
      IF(AGMOD.GT.G) AGMOD=G
      GO TO 20
   15 AGMOD=CONMOD/((1.0+PU)*2.0)
C     AT LATER STAGES OF LOADING, THE AGGREGATE INTERLOCK STIFFNESS IS
C     NOT PERMITTED TO INCREASE AS THE CRACK CLOSES.
   20 IF(AGMOD.GT.DAGG(NEL).AND.NUMINC.GT.3) RETURN
      DAGG(NEL)=AGMOD
      IF(WC.LT.1.0E-6.OR.INDOWL(NEL).EQ.0.OR.NDOWEL(NEL).EQ.1) RETURN
C     CALCULATION OF DOWEL STIFFNESS AND ADDITION TO AGGREGATE
C     INTERLOCK.
      IF(NDOWEL(NEL).EQ.1) RETURN
      IF(INDCEL(NEL)) 22,24,26
   22 HT=SQRT((Y(NODEEL(NEL,2))-Y(NODEEL(NEL,1)))**2+
     1(Z(NODEEL(NEL,2))-Z(NODEEL(NEL,1)))**2)
      GO TO 30
   24 HT=Z(NODEEL(NEL,2))-Z(NODEEL(NEL,1))
      GO TO 30
   26 HT=Y(NODEEL(NEL,2))-Y(NODEEL(NEL,1))
   30 CALL DOWEL(HT)
      IF(NDOWEL(NEL).EQ.1) RETURN
C     THE DOWEL STIFFNESS IS CONVERTED TO AN EQUIVALENT SHEAR MODULUS
C     ACROSS THE CRACK AND IS ADDED TO THE AGGREGATE INTERLOCK MODULUS.
      DSTF=80000.0*WIDTHC(NEL)*WC*(-FC)**.333/(HT*ELTHN(NEL))
      DSTFL=.2*DAGG(NEL)
      IF(DSTF.GT.DSTFL) DSTF=DSTFL
      DAGG(NEL)=DAGG(NEL)+DSTF
      RETURN
      END
      SUBROUTINE WCRACK(WC)
C*************************************************************************
C      THIS SUBROUTINE CALCULATES THE CRACK WIDTH IN A CRACKED FINITE
C     ELEMENT
```

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C*****************************************************************
C
      COMMON/BLOCK1/NEQNS,NBAND,NBLK,NUMINC,ICOUNT
      COMMON/BLOCK2/NEL,NELT,NELCHK,NELD,INDCEL(185),INELSZ(185),
     1INDOEL(185),NIDTNC(185),CANGLE(185),CALTER(185),
     2INELTY(215),ELTHN(215),NODEEL(215,4),NDREF(215),DPMMOD(215)
      COMMON/BLOCK6/CONMOD,REOMOD,PREMOD,SLPMOD,SHSMOD,FC,FT,
     1FAGG,FUN,FUP,FUNS,DF,ESLIP,ECULT,EPULT,EPULT,EMSULT,P1,P2,P3,P4,
     2PU,ERREO,EEPRE,CONDEV,DEVCON,DEVREO,PREDEV,SLPDEV,IDEV,AVCSP,RELAX
      COMMON/BLCK12/SIGCON(185,3),ECON(185,3),SIGMS(185,4),EMS(185,4),
     1TSGCON(185,3),TECON(185,3),TSGCC(185),TSGCT(185),TSGAGG(185),
     2TECC(185),TECT(185),TSGMS(185,4),TEMS(185,4),SIGREO(190),
     3ERREO(190),TSGREO(190),TERREO(190)
      COMMON/BLCK13/NYMESH(185,4),NDOWEL(185),NCRACK(185),NYAGG(185),
     1NYREO(190),NYCREO(190)
      INTEGER*4 CALTER
      DIMENSION SIG(3)
      IF(NYAGG(NEL).EQ.1) RETURN
      IF(ICOUNT.EQ.2) GO TO 10
      DO 5 I=1,3
      SIG(I)=TECON(NEL,I)+ECON(NEL,I)
 5    CONTINUE
      GO TO 20
 10   DO 15 I=1,3
      SIG(I)=TECON(NEL,I)
 15   CONTINUE
C     CALCULATION OF CENTROIDAL CONCRETE STRAIN PERPENDICULAR TO CRACK
C     DIRECTION
 20   THETA=CANGLE(NEL)*3.1415926/180.0
      C=COS(THETA)
      S=SIN(THETA)
      STRAIN=SIG(1)*C**2+SIG(2)*S**2+SIG(3)*S*C
      IF(STRAIN.GT.0.0) GO TO 30
      IF(NUMINC.LE.3) GO TO 24
      WRITE (6,23) NEL,STRAIN
 23   FORMAT(//' ** CRACK IN ELEMENT NUMBER',I3,' HAS CLOSED **',
     1/,' STRAIN =',E14.5)
      WC=1.0E-8
      RETURN
 24   WRITE (6,25) NEL,STRAIN
 25   FORMAT(////' *** CRACK IN ELEMENT NUMBER',I3,' HAS CLOSED ***'
     1,/,' STRAIN ACROSS CRACK NOW IS',E14.5/,' ELEMENT WILL NOW BE CONS
     1IDERED UNCRACKED; NCRACK(NEL) =0')
      NCRACK(NEL)=0
 30   WC=STRAIN*AVCSP
      RETURN
      END
      SUBROUTINE REOSTF(RS)
C*****************************************************************
C      THIS SUBROUTINE CALCULATES THE REINFORCEMENT STIFFNESS FOR A
C     GIVEN (STRESS,STRAIN) CONDITION IN BOTH INCREMENTAL AND TOTAL
C     LOAD SITUATIONS
C*****************************************************************
C
      COMMON/BLOCK1/NEQNS,NBAND,NBLK,NUMINC,ICOUNT
      COMMON/BLOCK4/NR,NREO,NODES(190,2),INRDN(190),INRTY(190),
     1RALTER(190),RAREA(190),CAREA(190),RSTF(190),TSGPRE(190),TEPRE(190)
      COMMON/BLOCK6/CONMOD,REOMOD,PREMOD,SLPMOD,SHSMOD,FC,FT,
     1FAGG,FUN,FUP,FUNS,DF,ESLIP,ECULT,ERULT,EPULT,EMSULT,P1,P2,P3,P4,
     2PU,ERREO,EEPRE,CONDEV,DEVCON,DEVREO,PREDEV,SLPDEV,IDEV,AVCSP,RELAX
      COMMON/BLCK12/SIGCON(185,3),ECON(185,3),SIGMS(185,4),EMS(185,4),
     1TSGCON(185,3),TECON(185,3),TSGCC(185),TSGCT(185),TSGAGG(185),
     2TECC(185),TECT(185),TSGMS(185,4),TEMS(185,4),SIGREO(190),
     3ERREO(190),TSGREO(190),TERREO(190)
      COMMON/BLCK13/NYMESH(185,4),NDOWEL(185),NCRACK(185),NYAGG(185),
     1NYREO(190),NYCREO(190)
      INTEGER*4 RALTER
      IF(ICOUNT.EQ.2) GO TO 5
      STRAIN=TERREO(NR)+ERREO(NR)
      ASTRSS=TSGREO(NR)+SIGREO(NR)
      GO TO 8
 5    STRAIN=TERREO(NR)
      ASTRSS=TSGREO(NR)
 8    IF(INRTY(NR)) 50,10,100
C     CALCULATION OF PRESTRESS MODULUS
 10   EE=EEPRE
      IF(STRAIN.GT.EE) GO TO 20
      RS=PREMOD
      RETURN
 20   IF(STRAIN.GE.EPULT) GO TO 200
      IF(ICOUNT.EQ.2) GO TO 25
      RS=470000.0
      RETURN
 25   STRESS=255360.0+470000.0*STRAIN
      PERCTG=(ASTRSS-STRESS)*100.0/STRESS
      IF(PERCTG.GE.PREDEV) GO TO 30
      RS=STRESS/STRAIN
      RETURN
C     RELAXATION IS EMPLOYED TO INCREASE RATE OF CONVERGENCE
 30   ASTRN=STRAIN*RELAX
      STRESS=255360.0+470000.0*ASTRN
      RS=STRESS/ASTRN
      RETURN
C     CALCULATION OF CONVENTIONAL REO MODULUS
 50   IF(STRAIN.LT.-ERREO.OR.STRAIN.GT.ERREO) GO TO 60
      RS=REOMOD
      RETURN
 60   IF(STRAIN.LE.-ERREO.OR.STRAIN.GE.ERULT) GO TO 200
      IF(ICOUNT.EQ.2) GO TO 70
      RS=430000.0
      RETURN
 70   RS=0.0
C     YIELDING CONVENTIONAL REINFORCEMENT BAR IS REPLACED BY ITS
C     EQUIVALENT STRUCTURAL NODAL FORCES.
      CALL FORCE(STRAIN)
      RETURN
C     CALCULATION OF BOND SLIP MODULUS
 100  D=STRAIN
      IF(STRAIN.LT.0.0) STRAIN=-STRAIN
      IF(D.GE.ESLIP) GO TO 200
      IF(ICOUNT.EQ.2) GO TO 110
      RS=(1950000.0-4.7E+09*D+4.17E+12*D**2-1.32E+15*D**3)*
     1SQRT(-FC/5000.0)
      IF(RS.LT.0.0) RS=1.0
      RETURN
 110  RS=(1950000.0-2.35E+09*D+1.39E+12*D**2-.33E+15*D**3)*
     1SQRT(-FC/5000.0)
      IF(RS.LT.0.0) RS=1.0
      RETURN
 200  NYREO(NR)=1
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      IF(INRTY(NR).EQ.0) GO TO 220
      RS=0.0
  201 SIGREO(NR)=0.0
      IF(INRTY(NR)) 202,204,206
  202 TSGREO(NR)=FUR
      GO TO 208
  204 TSGREO(NR)=FUF
      GO TO 208
  206 TSGREO(NR)=0.0
  208 IF(STRAIN.LT.0.0) TSGREO(NR)=-TSGREO(NR)
      IF(INRTY(NR)) 210,220,230
  210 WRITE (6,215) NR
  215 FORMAT(/' **** CONVENTIONAL REINFORCEMENT ELEMENT NUMBER',
     1I3,' HAS FAILED ****')
      RETURN
  220 IF(ICOUNT.EQ.[illegible]
      STRESS=TSGREO(NR)+SIGR[illegible]
      GO TO 22[illegible]
  22[illegible] STRESS=[illegible]GREO([illegible]
      WRITE (6,226) [illegible] ),TERREO(NR)
[illegible] FO[illegible] [illegible]*******',/,' ***  BEAM  FAILURE  ***'
     1[illegible] ****',//,' PRESTRESS REINFORCEMENT NUMBER'
[illegible] [illegible]ENSILE STRESS IN CABLE =',E14.5/,
[illegible] [illegible]BLE =',E14.5)
[illegible]
      [illegible] (6 [illegible]) NR
      [illegible](/' **** BOND SLIP LINKAGE ELEMENT NUMBER',I3,
     [illegible]HAS FAILED ****')
      RETURN
      END
      SUBROUTINE FORCE(STRAIN)
C     ****************************************************************
C     THIS SUBROUTINE CALCULATES THE NODAL RESTRAINT FORCES OF A
C     YIELDING REINFORCEMENT ELEMENT, AND ADDS THE FORCES INTO THE TOTAL
C     REINFORCEMENT RESTRAINT LOAD VECTOR REOTOT(NREO)
C*********************************************************************
C
      COMMON/BLOCK1/NEQNS,NBAND,NBLK,NUMINC,ICOUNT
      COMMON/BLOCK4/NR,NREO,NODES(190,2),INRDN(190),INRTY(190),
     1RALTER(190),RAREA(190),CAREA(190),RSTF(190),TSGPRE(190),TEPRE(190)
      COMMON/BLCK11/RTOT(750),REOTOT(750),DTOT(750),R2(2,750),D2(2,750)
      INTEGER*4 RALTER
      STRESS=53800.0+430000.0*STRAIN
      IF(STRAIN.LT.0.0) STRESS=-54202.0+430000.0*STRAIN
      II=NODES(NR,1)
      JJ=NODES(NR,2)
      KK=NBLK
      NBLK=1
      CALL LOCATE(II,JJ,NROW,NCOL)
      N1=NROW+INRDN(NR)+1
      N2=NCOL+INRDN(NR)+1
      REOTOT(N2)=REOTOT(N2)-STRESS*RAREA(NR)
      REOTOT(N1)=REOTOT(N1)+STRESS*RAREA(NR)
      NBLK=KK
      RETURN
      END
      SUBROUTINE NDCONC(ND)
C*********************************************************************
C     THIS SUBROUTINE CALCULATES THE CONCRETE CONSTITUTIVE MATRIX IN
C     EITHER THE INCREMENTAL OR TOTAL LOAD CASES FOR THE CRACKED OR
C     UNCRACKED ELEMENT.
C*********************************************************************
C
      COMMON/BLOCK1/NEQNS,NBAND,NBLK,NUMINC,ICOUNT
      COMMON/BLOCK2/NEL,NELT,NELCHK,NELD,INDCEL(185),INELSZ(185),
     1INDOWL(185),WIDTHC(185),CANGLE(185),CALTER(185),
     2INELTY(215),ELTHN(215),NODEEL(215,4),NDREF(215),DPNMOD(215)
      COMMON/BLOCK6/CONMOD,REOMOD,PREMOD,SLPMOD,SHSMOD,FC,FT,
     1FAGG,FUR,FUP,FUHS,OF,ESLIP,ECULT,ERULT,EPULT,EHSULT,P1,P2,P3,P4,
     2PU,ERREO,EEPRE,CONDEV,DEVCON,DEVREO,PREDEV,SLPDEV,IDEV,AVCSP,RELAX
      COMMON/BLCK10/ESTF(12,12),BS(3,3),B1(3,12,9),B2(3,12,9),D(3,3),
     1DCONC(185,3,3),DCRACK(185),DAGG(185)
      COMMON/BLCK12/SIGCON(185,3),ECON(185,3),SIGHS(185,4),EHS(185,4),
     1TSGCON(185,3),TECON(185,3),TSGCC(185),TSGCT(185),TSGAGG(185),
     2TECC(185),TECT(185),TSGHS(185,4),TEHS(185,4),SIGREO(190),
     3EREO(190),TSGREO(190),TEREO(190)
      COMMON/BLCK13/NYHESN(185,4),NDOWEL(185),NCPACK(185),NYAGG(185),
     1NYREO(190),NYCREO(190)
      INTEGER*4 CALTER
      DIMENSION DD(3,3),TM(3,3),TT1(3,3),DTT1(3,3)
      REAL*4 ND(3,3)
      DO 3 I=1,3
      DO 3 J=1,3
      ND(J,I)=0.0
      DD(J,I)=0.0
      DTT1(J,I)=0.0
    3 CONTINUE
C     CALCULATION OF PRINCIPAL STRESSES AND STRAINS
      CALL PRINCS
      CALL PRINCE
      ET=TECT(NEL)
      EC=TECC(NEL)
      ST=TSGCT(NEL)
      SC=TSGCC(NEL)
      ES=FC/ECULT
      EEPT=ET/ECULT
      EEPC=EC/ECULT
      IF(NCRACK(NEL).NE.0) GO TO 30
C     DEFINITION OF POISSON RATIO
      IF(ST.GE.0.0.AND.SC.GE.0.0) P=P2
      IF(ST.LT.0.0.AND.SC.LT.0.0) P=P1
      IF(ST.GE.0.0.AND.SC.LT.0.0) P=P3
      IF(ST.EQ.0.0) GO TO 5
      A1=SC/ST
    5 A2=ST/SC
      IF(ST.GE.0.0.OR.ET.GE.0.0) GO TO 10
C***** INCREMENTAL LOADING *****
      IF(ICOUNT.EQ.2) GO TO 12
C     COMPRESSIVE STRESS IN PRINCIPAL DIRECTION.
      E1B=CONMOD*(1.0-EEPT**2)/(1.0+(CONMOD/(ES*(1.0-PU*A1))-2.0)*
     1EEPT+EEPT**2)**2
      GO TO 14
C     TENSILE STRESS IN PRINCIPAL DIRECTION FOR BOTH INCREMETAL AND
C     TOTAL LOADS.
   10 E1B=CONMOD
      GO TO 18
C***** TOTAL LOADING *****
C     COMPRESSIVE STRESS IN PRINCIPAL DIRECTION.
   12 E1B=CONMOD/(1.0+(CONMOD/((1.0-PU*A1)*ES)-2.0)*EEPT+EEPT**2)
      GO TO 16
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C     SIMILAR STIFFNESS DERIVAIONS FOR ORTHOGONAL DIRECTION.
   14 E2B=CONMOD*(1.0-EEPC**2)/(1.0+(CONMOD/(ES*(1.0-PU*A2))-2.0)*
     1EEPC+EEPC**2)**2
      GO TO 25
   16 E2B=CONMOD/(1.0+(CONMOD/((1.0-PU*A2)*ES)-2.0)*EEPC+EEPC**2)
      GO TO 25
   18 IF(SC.GE.0.0.OR.FC.GE.0.0) GO TO 20
      IF(ICOUNT.EQ.2) GO TO 19
      E2B=CONMOD*(1.0-EEPC**2)/(1.0+(CONMOD/(ES*(1.0
     1-PU*A2))-2.0)*EEPC+EEPC**2)**2
      GO TO 25
   19 E2B=CONMOD/(1.0+(CONMOD/((1.0-PU*A2)*ES)-2.0)*EEPC+EEPC**2)
      GO TO 25
   20 E2B=CONMOD
   25 DELTA=E1B/(E2B/E2B-P**2)
      DD(1,1)=DELTA*E1B/E2B
      DD(2,1)=DELTA*P
      DD(1,2)=DELTA*P
      DD(2,2)=DELTA
      DD(3,3)=E1B*E2B/(E1B+E2B+2*E2B*P)
      GO TO 40
C     FORMATION OF PRINCIPAL CONSTITUTIVE MATRIX FOR A CRACKED ELEMENT
   30 IF(NCRACK(NEL).LT.-1.OR.NCRACK(NEL).GT.1) GO TO 37
      IF(SC.GE.0.0) GO TO 36
      IF(ICOUNT.EQ.1) GO TO 34
      D22=CONMOD/(1.0+(CONMOD/ES-2.0)*EEPC+EEPC**2)
      GO TO 38
   34 D22=CONMOD*(1.0-EEPC**2)/(1.0+(CONMOD/ES-2.0)*EEPC+EEPC**2)**2
      GO TO 38
   36 D22=CONMOD/(1.0-PU**2)
      GO TO 38
   37 D22=1000.0
   38 DCRACK(NEL)=D22
      DD(2,2)=D22
      DD(3,3)=DAGG(NEL)
C     TRANSFORMATION OF CONSTITUTIVE MATRIX , CHANGING FROM PRINCIPAL
C     STRESS AXES TO LOCAL ELEMENT AXES
   40 IF(EEPC.GT..99) GO TO 80
      THETA=-CANGLE(NEL)*3.1415926/180.0
      C=COS(THETA)
      S=SIN(THETA)
C     DEFINITION OF TRANSFORMATION MATRICES.
      TM(1,1)=C**2
      TM(2,1)=S**2
      TM(3,1)=2.0*S*C
      TM(1,2)=S**2
      TM(2,2)=C**2
      TM(3,2)=-2.0*S*C
      TM(1,3)=-S*C
      TM(2,3)=S*C
      TM(3,3)=C**2-S**2
      DO 50 I=1,3
      DO 50 J=1,3
      DO 50 K=1,3
      DTT1(J,I)=DTT1(J,I)+DD(K,J)*TM(K,I)
   50 CONTINUE
C     NOTE THAT IN ABOVE MATRIX MULTIPLICATION, MATRIX DD(3,3) IS
C     SYMMETRIC
      DO 60 I=1,3
      DO 60 J=1,3
      DO 60 K=1,3
    6 ND(J,I)=ND(J,I)+TM(K,J)*DTT1(K,I)
   60 CONTINUE
      DO 70 I=1,3
      IF(ND(I,I).LT.0.0) ND(I,I)=1000.0
   70 CONTINUE
      GO TO 90
   80 WRITE (6,82) NEL,EEPC
   82 FORMAT(///' **** ELEMENT NUMBER',I3,' HAS EXCEEDED ULTIMATE STRAIN
     1 ****',/,'     EEPC =',E14.5)
      STOP
   90 RETURN
      END
      SUBROUTINE KUTTA(NS1,NS2,NS3,TS,AL,AR)
C*********************************************************************
C      THIS SUBROUTINE ADJUSTS THE STRUCTURAL COMPONENT STIFFNESSES
C      FOR THE NEW LOAD INCREMENT USING THE KUTTA-RUNGE METHOD. ALSO,
C      FAILURE OF CONVENTIONAL REO, DOWEL AND SLIP LINKAGES IS TAKEN
C       INTO ACCOUNT IN STIFFNESS FORMULATION
C*********************************************************************
C
       COMMON/BLOCK1/NEQNS,NBAND,NBLK,NUMINC,ICOUNT
       COMMON/BLOCK2/NEL,NELT,NELCHK,NELD,INDCEL(185),INELSZ(185),
      1INDOWL(185),WIDTHC(185),CANGLE(185),CALTER(185),
      2INELTY(215),ELTHN(215),NODEEL(215,4),NDREF(215),DPEMOD(215)
       COMMON/BLOCK3/NELSH,INMESH(185),INSEMS(185),NDIRNS,PERSTL(185,4),
      1ANGLE(185,4),SHSTF(185,3,3),SHESH(185,1)
       COMMON/BLOCK4/NR,NREO,NODES(190,2),INRDN(190),INRTY(190),
      1RALTER(190),RAREA(190),CAREA(190),RSTF(190),TSGPRE(190),TEPPT(190)
       COMMON/BLOCK5/NNODES,ICNODE(220),X(220),Y(220),Z(220)
       COMMON/BLOCK6/CONMOD,REOMOD,PREMOD,SLPMOD,SHSMOD,FC,FT,
      1FAGG,FUR,FUP,FUMS,DP,ESLIP,ECULT,EPULT,EPULT,EMSULT,P1,P2,P3,P4,
      2PU,EEREO,EEPRE,CONDEV,DEVCON,DEVREO,PREDEV,SLPDEV,IDEV,AVCSP,RELAX
       COMMON/BLOCK9/LPOINT(20,2),FDUB1,FDUB2,NUMREC,
      1NUMBLK,ISOLVE,LEN
       COMMON/BLCK10/ESTF(12,12),BS(3,3),B1(3,12,9),B2(3,12,9),D(3,3),
      1DCONC(185,3,3),DCRACK(185),DAGG(185)
       COMMON/BLCK12/SIGCON(185,3),ECON(185,3),SIGMS(185,4),EMS(185,4),
      1TSGCON(185,3),TECON(185,3),TSGCC(185),TSGST(185),TSGAGG(185),
      2TECC(185),TECT(185),TSGMS(185,4),TEMS(185,4),SIGREO(190),
      3EREO(190),TSGREO(190),TEREO(190)
       COMMON/BLCK13/NYMESH(185,4),NDOWEL(185),NCRACK(185),NYAGG(185),
      1NYREO(190),NYCREO(190)
       DIMENSION TS(NS1,NS2),AL(NS3),AR(NS3),DCDIF(185,3,3),ASTPDF(190),
      1DSM(3,3)
       REAL*4 ND(3,3)
       INTEGER*4 CALTER,RALTER,FDUB1,INFO(4),SALTER(185)
       INTEGER*2 LEN
       DATA MOD/Z00020000/
C**** ADJUSTMENT OF ALL CONCRETE ELEMENT CONSTITUTIVE MATRICES
       DO 50 I=1,NELT
       NEL=I
       IF(NCRACK(I).EQ.-1) NCRACK(I)=1
       SUM1=0.0
       SUM2=0.0
       DO 5 J=1,3
       DO 5 K=1,3
       ND(K,J)=0.0
     5 CONTINUE
       IF(NCRACK(I).EQ.0) GO TO 10
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C     AGGREGATE INTERLOCK STIFFNESS IS CALCULATED TO REDEFINE SHEAR
C     MODULUS
      CALL NCRACK(NC)
      CALL SHMOD(NC)
   10 CALL NDCONC(ND)
C     CHECK ON ANY CHANGE IN STEEL MESH STIFFNESS. STIFFNESS OF
C     REINFORCED CONCRETE ELEMENT WILL BE CHANGED AT EVERY LOAD
C     INCREMENT ONCE YIELDING HAS OCCURRED.
      IF(INMESH(NEL).EQ.0) GO TO 30
      SALTER(NEL)=0
      DO 25 J=1,NDIRNS
      IF(NYMESH(NEL,J).EQ.1) GO TO 25
      IF(ICOUNT.EQ.1) GO TO 22
      STRAIN=TENS(NEL,J)
      GO TO 24
   22 STRAIN=TENS(NEL,J)+EMS(NEL,J)
   24 IF(STRAIN.GT.-EERRO.AND.STRAIN.LT.EERRO) GO TO 25
      SALTER(NEL)=1
      GO TO 30
   25 CONTINUE
C     CHECK ON, AND CALCULATION OF CONCRETE CONSTITUTIVE MATRIX
C     CHANGE
   30 DO 35 J=1,3
      SUM1=SUM1+ND(J,J)
      SUM2=SUM2+DCONC(I,J,J)
   35 CONTINUE
      PERDEV=(SUM1-SUM2)*100.0/SUM1
      IF(PERDEV.LT.0.0) PERDEV=-PERDEV
      IF(PERDEV.LE.DEVCON) GO TO 48
      CALTER(I)=1
      DO 40 J=1,3
      DO 40 K=1,3
C     THE STIFFNESS CHANGES ADDED TO THE TOTL STIFFNESS MATRIX ARE
C     CALCULATED USING THE CHANGE IN THE CONSTITUTIVE MATRIX. THE
C     CURRENT CONSTITUTIVE MATRIX DCONC(NEL,I,J) IS THEN RESET.
      DCDIF(I,K,J)=ND(K,J)-DCONC(I,K,J)
      DCONC(I,K,J)=ND(K,J)
   40 CONTINUE
      GO TO 50
   48 CALTER(I)=0
   50 CONTINUE
C     ADJUSTMENT OF ALL REINFORCEMENT ELEMENT STIFFNESSES
      DO 60 I=1,NREO
      NN=I
      IF(NYREO(I).EQ.1) GO TO 58
      CALL REOSTF(RS)
      IF(RSTF(I).EQ.RS) GO TO 58
      IF(RSTF(I).EQ.0.0) RSTF(I)=1.0
      PERDEV=(RSTF(I)-RS)*100.0/RSTF(I)
      IF(RSTF(I).EQ.1.0) RSTF(I)=0.0
      IF(PERDEV.LT.0.0) PERDEV=-PERDEV
      IF(PERDEV.LT.DEVREO) GO TO 58
      RALTER(I)=1
      RSTFDF(I)=RS-RSTF(I)
      RSTF(I)=RS
      GO TO 60
   58 RALTER(I)=0
   60 CONTINUE
C        THOSE CONCRETE AND REINFORCEMENT ELEMENTS THAT HAVE UNDERGONE
C     STIFFNESS CHANGES NOW HAVE THEIR RESPECTIVE TOTAL STIFFNESS
C     CONTRIBUTIONS ADJUSTED. THIS PROCESS IS DONE BLOCK BY BLOCK.
C
C     TWO STIFFNESS BLOCKS IN THE PREVIOUS LOAD INCREMENT ARE READ INTO
C     CORE SO THAT APPROPRIATE STIFFNESS CHANGES CAN BE MADE
      REWIND 1
      NBLK=0
C     STIFFNESS TERMS ARE READ INTO SECOND BLOCK.
   80 K=1
      K1=LEN
      DO 90 I=1,NUMREC
      CALL READ(AR(K),LEN,0,LNUM,FDUB1,&305)
      K=K+LEN/4
      IF(I.EQ.(NUMREC-1)) LEN=(NS3-K+1)*4
   90 CONTINUE
      LEN=K1
      IF(NBLK.NE.0) GO TO 110
  100 NBLK=NBLK+1
      IF(NBLK.GT.NUMBLK) GO TO 300
C     SECOND BLOCK IS MOVED INTO FIRST BLOCK POSITION.
      DO 105 I=1,NS3
      AL(I)=AR(I)
      AR(I)=0.0
  105 CONTINUE
      IF(NBLK.EQ.NUMBLK) GO TO 110
      GO TO 80
C     CHANGES IN STIFFNESS ARE NOW COMPILED BLOCK BY BLOCK AND ARE ADDED
C    TO EXISTING IN-CORE STIFFNESSES
  110 MAXEQN=NBLK*NBAND
      IF(MAXEQN.GT.NEQNS) MAXEQN=NEQNS
      MINEQN=(NBLK-1)*NBAND+1
      MAXOLD=(NBLK-1)*NBAND
      NEQ=MAXEQN
      CALL NODLOC(NEQ,NN)
      NMAX=NN
      NEQ=MINEQN
      CALL NODLOC(NEQ,NN)
      NMIN=NN
      NEQ=MAXOLD
      CALL NODLOC(NEQ,NN)
      NNOLD=NN
C     THE LOWEST AND HIGHEST NODE NUMBERS CONTRIBUTING TO THE FIRST
C     BLOCK STIFFNESS ARE NMIN AND NMAX RESPECTIVELY.
C     A SEARCH IS NOW CONDUCTED TO FIND ANY ELEMENT WHOSE COMBINED
C     CONCRETE, AGGREGATE INTERLOCK AND DOWEL STIFFNESS HAS
C     CHANGED. THE STIFFNESS CHANGE IS ADDED TO THE IN-CORE STIFFNESS
C     ELEMENTS
      DO 130 I=1,NELT
      IF(CALTER(I).EQ.0.AND.SALTER(I).EQ.0) GO TO 130
      NEL=I
      DO 120 J=1,4
      IF((NODEEL(I,J).LT.NMIN).OR.(NODEEL(I,J).GT.NMAX)) GO TO 120
      DO 114 K=1,4
      IF(NODEEL(I,K).LE.NNOLD) GO TO 130
  114 CONTINUE
      DO 115 L=1,3
      DO 115 M=1,3
      D(M,L)=DCDIF(I,M,L)
  115 CONTINUE
C     ADDITION OF STEEL MESH STIFFNESS
      IF(INMESH(NEL).EQ.0.OR.SALTER(NEL).EQ.0) GO TO 118
```

```
      CALL STFSH(DSH)
      DO 116 L=1,3
      DO 116 M=1,3
      D(M,L)=D(M,L)+DSH(M,L)-SHSTF(NEL,M,L)
      SHSTF(NEL,M,L)=DSH(M,L)
  116 CONTINUE
  118 CALL ELSTF(NELOLD)
C     PARTITIONING OF ELEMENT STIFFNESS DIFFERENCE INTO COMPONENT SUB-
C     BLOCKS, AND ADDITION OF SUB-BLOCKS INTO TOTAL STIFFNESS MATRIX.
      CALL ADDNEL(NS1,NS2,NS3,TS,AL,AR)
      GO TO 130
  120 CONTINUE
  130 CONTINUE
C     ADDITION OF STIFFNESS DIFFERENCES OF SINGLE REINFORCING AND
C     PRESTRESS BARS AND BOND SLIP LINKAGES TO TS(NBAND,NBAND*2)
      DO 150 I=1,NREO
      IF(RALTER(I).EQ.0) GO TO 150
      NR=I
      DO 145 J=1,2
      IF((NODES(I,J).LT.NMIN).OR.(NODES(I,J).GT.NMAX)) GO TO 145
      DO 140 K=1,2
      IF(NODES(I,K).EQ.NNEOLD) GO TO 150
  140 CONTINUE
      STIF=RSTFDF(I)
      CALL STFADD(NS1,     ,NS3,TS,AL,AR,STIF)
      GO TO 150
  145 CONTINUE
  150 CONTINUE
C     THE ADJUSTED STIFFNESS BLOCK IS THEN MODIFIED AND WRITTEN BACK ON
C     TEMPORARY DISC FILE -FILE1
      CALL MODIFI(NS1,NS2,NS3,T6,AL,AR)
      INFO(2)=LPOINT(NBLK,1)
      CALL POINT(PDUB1,INFO,2)
      J=1
      J1=LEN
      DO 160 I=1,NUMREC
      CALL WRITE(AL(J),LEN,MOD,LNUM,PDUB1,6325)
      J=J+LEN/4
      IF(I.EQ.(NUMREC-1)) LEN=(NS3-J+1)*4
  160 CONTINUE
      LEN=J1
      GO TO 100
  300 DO 302 I=1,NEL
      CALTER(I)=0
  302 CONTINUE
      DO 304 I=1,NREO
      RALTER(I)=0
  304 CONTINUE
      RETURN
  305 WRITE (6,310)
  310 FORMAT(//' **** ERROR IN SYSTEM SUBROUTINE READ IN SUBROUTINE
     1KUTTA ***')
      STOP
  325 WRITE (6,330)
  330 FORMAT(///' *** ERROR IN SYSTEM SUBROUTINE WRITE CALLED IN SUBROUT
     1INE KUTTA ***')
      STOP
      END
      SUBROUTINE DOWEL(HT)
C*****************************************************************
C     THIS SUBROUTINE CHECKS THAT THE DOWEL STIFFNESS MECHANISM FOR A
C     CRACKED ELEMENT HAS NOT FAILED.
C*****************************************************************
C
      COMMON/BLOCK1/NEQNS,NBAND,NBLK,NUMINC,ICOUNT
      COMMON/BLOCK2/NEL,NELT,NELCHK,NELD,INDCEL(185),INELSZ(185),
     1INDOWL(185),WIDTHC(185),CANGLE(185),CALTER(185),
     2INELTI(215),ELTHN(215),MODREL(215,4),NDREF(215),DPNMOD(215)
      COMMON/BLOCK6/CONMOD,REOMOD,PREMOD,SLPMOD,SHSMOD,FC,FT,
     1FAGG,FUR,FUP,FURS,DP,ESLIP,ECULT,EPULT,EPULT,EMSULT,P1,P2,P3,P4,
     2PU,EREO,EPRE,CONDEV,DEVCON,DEVPRO,PREDEV,SLPDEV,IDEV,AVCSP,RELAX
      COMMON/BLCK12/SIGCON(185,3),ECON(185,3),SIGMS(185,4),EMS(185,4),
     1TSGCON(185,3),TECON(185,3),TSGCC(185),TSGCT(185),TSGAGG(185),
     2TECC(185),TECT(185),TSGMS(185,4),TEMS(185,4),SIGREO(190),
     3EREO(190),TSGREO(190),TEREO(190)
      COMMON/BLCK13/NTMESH(185,4),NDOWEL(185),NCRACK(185),NYAGG(185),
     1NYREO(190),NYCREO(190)
      INTEGER*4 CALTER
      DIMENSION F(3)
      THETA=CANGLE(NEL)*3.141593/180.0
      C=COS(THETA)
      S=SIN(THETA)
      IF(ICOUNT.EQ.1) GO TO 10
      DO 5 J=1,3
      E(J)=TECON(NEL,J)
    5 CONTINUE
      GO TO 20
   10 DO 15 J=1,3
      E(J)=TECON(NEL,J)+ECON(NEL,J)
   15 CONTINUE
C     GAMMA IS THE SHEAR STRAIN AT THE CENTROID PARALLEL TO THE CRACK.
   20 GAMMA=-2.0*C*S*(E(1)-E(2))+(C**2-S**2)*E(3)
      DELTA=HT*GAMMA
      IF(DELTA.LT.DP.AND.DELTA.GT.-DP) RETURN
      NDOWEL(NEL)=1
      WRITE (6,25) NEL,DELTA
   25 FORMAT(/' **** DOWEL STIFFNESS MECHANISM FOR CRACKED ELEMENT',I5,'
     1 HAS FAILED: SHEAR DISPLACEMENT ACROSS CRACK =',E11.4)
      RETURN
      END
END OF FILE
```

$LIST FILE51

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      SUBROUTINE MOHR(ANGLE,II,ESTEEL)
C*********************************************************************
C       THIS SUBROUTINE CALCULATES THE STRAIN IN THE UNIAXIAL DIRECTION
C     OF A STEEL MESH BAR AT AN ANGLE OF ANGLE(NEL,I) TO THE HORIZONTAL
C*********************************************************************
C
      COMMON/BLOCK1/NEQNS,NBAND,NBLK,NUMINC,ICOUNT
      COMMON/BLOCK2/NEL,NELT,NELCHK,NELD,INDCEL(185),INELSZ(185),
     1INDOWL(185),WIDTHC(185),CANGLE(185),CALTER(185),
     2INELTY(215),ELTHN(215),NODEEL(215,4),NDREF(215),DPMMOD(215)
      COMMON/BLCK12/SIGCON(185,3),ECON(185,3),SIGMS(185,4),EMS(185,4),
     1TSGCON(185,3),TECON(185,3),TSGCC(185),TSGCT(185),TSGAGG(185),
     2TECC(185),TECT(185),TSGMS(185,4),TEMS(185,4),SIGREO(190),
     3EREO(190),TSGREO(190),TEREO(190)
      DIMENSION ANGLE(185,4)
      INTEGER*4 CALTER
      IF(ICOUNT.EQ.2) GO TO 5
      E1=ECON(NEL,1)
      E2=ECON(NEL,2)
      E3=ECON(NEL,3)
      GO TO 10
    5 E1=TECON(NEL,1)
      E2=TECON(NEL,2)
      E3=TECON(NEL,3)
   10 THETA=ANGLE(NEL,II)*3.1415926/180.0
      ESTEEL=(COS(THETA))**2*E1+(SIN(THETA))**2*E2+SIN(THETA)*COS(TH
     1ETA)*E3
      RETURN
      END
      SUBROUTINE PRINCE
C*********************************************************************
C      THIS SUBROUTINE CALCULATES THE TOTAL PRINCIPAL TENSILE AND
C     COMPRESSIVE STRAINS (TECC(NEL),TECT(NEL)) THAT CORRESPOND TO THEIR
C     RESPECTIVE PRINCIPAL STRESSES
C*********************************************************************
C
      COMMON/BLOCK1/NEQNS,NBAND,NBLK,NUMINC,ICOUNT
      COMMON/BLOCK2/NEL,NELT,NELCHK,NELD,INDCEL(185),INELSZ(185),
     1INDOWL(185),WIDTHC(185),CANGLE(185),CALTER(185),
     2INELTY(215),ELTHN(215),NODEEL(215,4),NDREF(215),DPMMOD(215)
      COMMON/BLCK12/SIGCON(185,3),ECON(185,3),SIGMS(185,4),EMS(185,4),
     1TSGCON(185,3),TECON(185,3),TSGCC(185),TSGCT(185),TSGAGG(185),
     2TECC(185),TECT(185),TSGMS(185,4),TEMS(185,4),SIGREO(190),
     3EREO(190),TSGREO(190),TEREO(190)
      COMMON/BLCK13/NYMESH(185,4),NDOWEL(185),NCRACK(185),NYAGG(185),
     1NYREO(190),NYCREO(190)
      INTEGER*4 CALTER
      IF(ICOUNT.EQ.2) GO TO 5
      E1=ECON(NEL,1)+TECON(NEL,1)
      E2=ECON(NEL,2)+TECON(NEL,2)
      E3=(ECON(NEL,3)+TECON(NEL,3))/2.0
      GO TO 10
    5 E1=TECON(NEL,1)
      E2=TECON(NEL,2)
      E3=TECON(NEL,3)/2.0
   10 IF(NCRACK(NEL).EQ.0) GO TO 20
C     ONCE A CONCRETE ELEMENT HAS CRACKED, PRINCIPAL STRAIN DIRECTIONS
C     ARE NOT PERMITTED TO CHANGE
      THETA=CANGLE(NEL)*3.1415926/180.0
      C=COS(THETA)
      S=SIN(THETA)
      TECT(NEL)=E1*C**2+E2*S**2+E3*S*C*2.0
      TECC(NEL)=E1*S**2+E2*C**2-E3*S*C*2.0
      RETURN
   20 E4=(SQRT((E1-E2)**2+4.0*E3**2))/2.0
      E5=(E1+E2)/2.0+E4
      E6=(E1+E2)/2.0-E4
      TECT(NEL)=E5
      TECC(NEL)=E6
      RETURN
      END
      SUBROUTINE PRINCS
C*********************************************************************
C       THIS SUBROUTINE CACULATES THE TOTAL PRINCIPAL TENSILE AND
C     COMPRESSIVE STRESSES (TSGCC(NEL) AND TSGCT(NEL)) , AND THE ANGLE
C     (CANGLE(NEL)) OF THE PRINCIPAL TENSILE STRESS TO THE HORIZONTAL
C     (IN THE CASE OF A CRACKED ELEMENT, ANGLE REMAINS CONSTANT AFTER
C     CRACKING)
C*********************************************************************
C
      COMMON/BLOCK1/NEQNS,NBAND,NBLK,NUMINC,ICOUNT
      COMMON/BLOCK2/NEL,NELT,NELCHK,NELD,INDCEL(185),INELSZ(185),
     1INDOWL(185),WIDTHC(185),CANGLE(185),CALTER(185),
     2INELTY(215),ELTHN(215),NODEEL(215,4),NDREF(215),DPMMOD(215)
      COMMON/BLCK12/SIGCON(185,3),ECON(185,3),SIGMS(185,4),EMS(185,4),
     1TSGCON(185,3),TECON(185,3),TSGCC(185),TSGCT(185),TSGAGG(185),
     2TECC(185),TECT(185),TSGMS(185,4),TEMS(185,4),SIGREO(190),
     3EREO(190),TSGREO(190),TEREO(190)
      COMMON/BLCK13/NYMESH(185,4),NDOWEL(185),NCRACK(185),NYAGG(185),
     1NYREO(190),NYCREO(190)
      INTEGER*4 CALTER
      IF(NCRACK(NEL).EQ.2) GO TO 25
      IF(ICOUNT.EQ.2) GO TO 5
      S1=SIGCON(NEL,1)+TSGCON(NEL,1)
      S2=SIGCON(NEL,2)+TSGCON(NEL,2)
      S3=-SIGCON(NEL,3)-TSGCON(NEL,3)
      GO TO 10
    5 S1=TSGCON(NEL,1)
      S2=TSGCON(NEL,2)
      S3=-TSGCON(NEL,3)
   10 IF(NCRACK(NEL).EQ.1) GO TO 20
      S4=(SQRT((S1-S2)**2+4.0*S3**2))/2.0
      S5=(S1+S2)/2.0+S4
      S6=(S1+S2)/2.0-S4
      TSGCT(NEL)=S5
      TSGCC(NEL)=S6
      IF(S3.LT.1.0E-05.AND.S3.GT.-1.0E-05) GO TO 15
      TN=(S1-S5)/S3
      CANGLE(NEL)=ATAN(TN)*180.0/3.1415926
      IF(NCRACK(NEL).EQ.-1) GO TO 20
      RETURN
   15 IF(S1.GT.S2) CANGLE(NEL)=0.0
      IF(S2.GE.S1) CANGLE(NEL)=90.0
      RETURN
   20 TSGCT(NEL)=0.0
      THETA=CANGLE(NEL)*3.1415926/180.0
      C=COS(THETA)
      S=SIN(THETA)
```

```
      TSGCC(NEL)=S1*S**2+S2*C**2+S3*2.0*S*C
      RETURN
   25 TSGCT(NEL)=0.0
      TSGCC(NEL)=0.0
      RETURN
      END
      SUBROUTINE CCALC(A,B,AA)
C*********************************************************************
C       THIS SUBROUTINE CALCULATES THE MATRIX (B) THAT DEFINES THE
C     RELATIONSHIP BETWEEN CENTROIDAL FINITE ELEMENT STRAINS AND NODAL
C     DEFORMATIONS
C*********************************************************************
C
      COMMON/BLOCK2/NEL,NELT,NELCHK,NELD,INDCEL(185),INELSZ(185),
     1INDOWL(185),WIDTHC(185),CANGLE(185),CALTER(185),
     2INELTY(215),ELTHN(215),NODEEL(215,4),NDREF(215),DPHMOD(215)
      DIMENSION AA(12,3),A1(12,3),A2(12,3)
      INTEGER*4 CALTER
      DO 10 I=1,3
      DO 10 J=1,12
      A1(J,I)=0.0
      A2(J,I)=0.0
   10 CONTINUE
C     FORMATION OF MATRIX A1(12,3)
      A8=1.0/(A*8.0)
      A4=1.0/(A*4.0)
      B4=1.0/(B*4.0)
      B8=1.0/(B*8.0)
      A1(1,1)=-A8
      A1(4,1)=-3.0*A8
      A1(6,1)=-B*A4
      A1(7,1)=A8
      A1(10,1)=3.0*A8
      A1(12,1)=-B*A4
      A1(2,2)=-3.0*B8
      A1(3,2)=-A*B4
      A1(5,2)=B8
      A1(8,2)=3.0*B8
      A1(9,2)=A1(3,2)
      A1(11,2)=-B8
      A1(1,3)=-B4
      A1(2,3)=-A4
      A1(4,3)=B4
      A1(5,3)=A1(2,3)
      A1(7,3)=A1(4,3)
      A1(8,3)=A4
      A1(10,3)=A1(1,3)
      A1(11,3)=A1(8,3)
C     FORMATION OF MATRIX A2(12,3)
      A2(1,1)=A1(4,1)
      A2(3,1)=B*A4
      A2(4,1)=A1(1,1)
      A2(7,1)=A1(10,1)
      A2(9,1)=A2(3,1)
      A2(10,1)=A1(7,1)
      A2(2,2)=A1(11,2)
      A2(5,2)=A1(8,2)
      A2(6,2)=A*B4
      A2(8,2)=A1(5,2)
      A2(11,2)=A1(2,2)
      A2(12,2)=A2(6,2)
      DO 15 I=1,12
      A2(I,3)=A1(I,3)
   15 CONTINUE
      IF(INELTY(NEL).EQ.2) GO TO 30
      DO 25 J=1,3
      DO 25 I=1,12
      AA(I,J)=A1(I,J)
   25 CONTINUE
      RETURN
   30 DO 35 J=1,3
      DO 35 I=1,12
      AA(I,J)=A2(I,J)
   35 CONTINUE
      RETURN
      END
      SUBROUTINE STRESS(NC)
C*********************************************************************
C       THIS SUBROUTINE CALCULATES THE FOLLOWING BEAM COMPONENT TOTAL
C     STRESSES; CENTROIDAL CONCRETE ELEMENT STRESSES, CENTROIDAL STEEL
C     MESH STRESSES, CONVENTIONAL AND PRESTRESS STEEL STRESSES, AND
C     BOND LINKAGE SLIP STRESSES.
C*********************************************************************
C
      COMMON/BLOCK1/NEQNS,NBAND,NBLK,NUMINC,ICOUNT
      COMMON/BLOCK2/NEL,NELT,NELCHK,NELD,INDCEL(185),INELSZ(185),
     1INDOWL(185),WIDTHC(185),CANGLE(185),CALTER(185),
     2INELTY(215),ELTHN(215),NODEEL(215,4),NDREF(215),DPHMOD(215)
      COMMON/BLOCK3/NELSH,INMESH(185),INSTMS(185),NDIRNS,PERSTL(185,4),
     1ANGLE(185,4),SHSTF(185,3,3),SMESH(185,1)
      COMMON/BLOCK4/NR,NREO,NODES(190,2),INRDN(190),INRTY(190),
     1RALTER(190),RAREA(190),CAREA(190),RSTF(190),TSGPRE(190),TEPRE(190)
      COMMON/BLOCK5/NNODES,ICNODE(220),X(220),Y(220),Z(220)
      COMMON/BLOCK6/CONMOD,REOMOD,PREMOD,SLPMOD,SHSMOD,FC,FT,
     1FAGG,FUR,FUP,FUMS,DF,ESLIP,ECULT,ERULT,EPULT,EMSULT,P1,P2,P3,P4,
     2PU,EEREO,EEPRE,CONDEV,DEVCON,DEVREO,PREDEV,SLPDEV,IDEV,AVCSP,RELAX
      COMMON/BLOCK8/NDISPL,NODED(30),KODED(30),VALUED(30),D1(750)
      COMMON/BLCK10/ESTF(12,12),BS(3,3),B1(3,12,9),B2(3,12,9),D(3,3),
     1DCONC(185,3,3),DCRACK(185),DAGG(185)
      COMMON/BLCK12/SIGCON(185,3),ECON(185,3),SIGMS(185,4),EMS(185,4),
     1TSGCON(185,3),TECON(185,3),TSGCC(185),TSGCT(185),TSGAGG(185),
     2TECC(185),TECT(185),TSGMS(185,4),TEMS(185,4),SIGREO(190),
     3EREO(190),TSGREO(190),TEREO(190)
      COMMON/BLCK13/NYMESH(185,4),NDOWEL(185),NCPACK(185),NYAGG(185),
     1NYREO(190),NYCREO(190)
      COMMON/BLCK15/NELTOP,NLTP(40),NLBM(40),SIGXT1,SIGXT2,SIGXB1,
     1SIGXB2,SIGXS1,SIGXS2,EXT1,EXT2,EXB1,EXB2,EXS1,EXS2
      REAL*4 LENGTH,LT(5,5)
      INTEGER*4 CALTER,RALTER
      DIMENSION AA(12,3),E(3),R(12),DLOCAL(5),DGLOBL(5),
     1SIGMA(3),DC(3,3),TSGSHP(185,3),TESHR(185,3)
C     CALCULATION OF CONCRETE AND STEEL MESH STRESSES
      DO 60 I=1,NELT
      NEL=I
    5 A=(X(NODEEL(NEL,3))-X(NODEEL(NEL,2)))/2.0
      IF(INDCEL(NEL)) 6,7,8
    6 B=(SQRT((Y(NODEEL(NEL,2))-Y(NODEEL(NEL,1)))**2+(Z(NODEEL(NEL,2))
     1-Z(NODEEL(NEL,1)))**2))/2.0
      GO TO 10
    7 B=(Z(NODEEL(NEL,2))-Z(NODEEL(NEL,1)))/2.0
```

```
      GO TO 10
    8 B=(Y(NODEEL(NEL,2))-Y(NODEEL(NEL,1)))/2.0
C**** CALCULATION OF NODAL DISPLACEMENT VECTOR S(12) FOR CONCRETE
C     ELEMENT
   10 DO 25 J=1,4
      II=NODEEL(I,J)
      JJ=1
      NBLK=1
      CALL LOCATE(II,JJ,NROW,NCOL)
      R((J-1)*3+1)=D1(NROW)
      IF(ICNODE(II).EQ.1) GO TO 15
      R((J-1)*3+2)=D1(NROW+1)
      R((J-1)*3+3)=D1(NROW+2)
      IF(ICNODE(II).EQ.2) R((J-1)*3+3)=D1(NROW+3)
      IF(INDCEL(I).EQ.0) R((J-1)*3+3)=-R((J-1)*3+3)
      GO TO 25
   15 IF(INDCIL(I)) 16,20,22
C     TRANSFORMATION OF DISPLACEMENTS OF CORNER NODE OF AN INCLINED
C     ELEMENT FROM GLOBAL TO LOCAL FORM
   16 C=(Y(NODEEL(NEL,2))-Y(NODEEL(NEL,1)))/(2.0*B)
      S=(X(NODEEL(NEL,2))-X(NODEEL(NEL,1)))/(2.0*B)
C     NOTE THAT THE TRANSPOSED TRANSFORMATION MATRIX LT(5,5) IS BELOW
      DO 17 K=1,5
      DO 17 L=1,5
      LT(L,K)=0.0
   17 CONTINUE
      LT(1,1)=1.0
      LT(2,2)=C
      LT(3,2)=S
      LT(2,3)=-S
      LT(3,3)=C
      LT(4,4)=C
      LT(5,4)=S
      LT(4,5)=-S
      LT(5,5)=C
      DO 18 K=1,5
      DGLOBL(K)=D1(NROW+K-1)
      DLOCAL(K)=0.0
   18 CONTINUE
      DO 19 L=1,5
      DO 19 K=1,5
      DLOCAL(L)=DLOCAL(L)+LT(K,L)*DGLOBL(K)
   19 CONTINUE
      R((J-1)*3+2)=DLOCAL(2)
      R((J-1)*3+3)=DLOCAL(5)
      GO TO 25
   20 R((J-1)*3+2)=D1(NROW+2)
      DD=-1.0*D1(NROW+3)
      R((J-1)*3+3)=DD
      GO TO 25
   22 R((J-1)*3+2)=D1(NROW+1)
      R((J-1)*3+3)=D1(NROW+4)
   25 CONTINUE
C     INITIAL INCLUSION OF CONCRETE SHRINKAGE STRESSES
      IF(NUMINC.GT.1.OR.NC.GE.1) GO TO 30
      DO 26 J=1,NELT
      TSGCON(J,1)=SIGXS1
      TSGCON(J,2)=SIGXS2
      TECON(J,1)=EXS1
      TECON(J,2)=EXS2
   26 CONTINUE
      DO 27 J=1,NELTOP
      J1=NLTP(J)
      J2=NLBM(J)
      TSGCON(J1,1)=SIGXT1
      TSGCON(J1,2)=SIGXT2
      TSGCON(J2,1)=SIGXB1
      TSGCON(J2,2)=SIGXB2
      TECON(J1,1)=EXT1
      TECON(J1,2)=EXT2
      TECON(J2,1)=EXB1
      TECON(J2,2)=EXB2
   27 CONTINUE
      DO 28 K=1,3
      DO 28 J=1,NELT
      TSGSHR(J,K)=TSGCON(J,K)
      TESHR(J,K)=TECON(J,K)
   28 CONTINUE
C**** CALCULATION OF CENTROIDAL STRAINS
   30 CALL CCALC(A,B,AA)
      DO 34 J=1,3
      E(J)=0.0
      DO 32 K=1,12
      E(J)=E(J)+AA(K,J)*R(K)
   32 CONTINUE
      IF(ICOUNT.EQ.2) GO TO 33
      ECON(NEL,J)=E(J)
      GO TO 34
   33 TECON(NEL,J)=E(J)
      IF(NCRACK(NEL).NE.0) GO TO 34
      TECON(NEL,J)=E(J)+TESHR(NEL,J)
   34 CONTINUE
C**** CALCULATION OF CONCRETE CENTRO:    STRESSES
      DO 35 J=1,3
      SIGMA(J)=0.0
   35 CONTINUE
      IF(NCRACK(NEL).EQ.2) GO TO 39
      IF(NCRACK(NEL).NE.0) GO TO 37
      DO 36 K=1;3
      DO 36 J=1,3
      DC(J,K)=DCONC(I,J,K)
   36 CONTINUE
      GO TO 41
C     FORMULATION OF CRACKED CONCRETE CONSTITUTIVE MATRIX FOR STRESS
C     CALCULATION-SHEAR STRENGTH ACROSS CRACK IS NOT INCLUDED
   37 THETA=-CANGLE(NEL)*3.1415926/180.0
      C=COS(THETA)
      S=SIN(THETA)
      DC(1,1)=S**4
      DC(2,1)=(C**2)*(S**2)
      DC(3,1)=(S**3)*C
      DC(1,2)=(C**2)*(S**2)
      DC(2,2)=C**4
      DC(3,2)=S*(C**3)
      DC(1,3)=(S**3)*C
      DC(2,3)=S*(C**3)
      DC(3,3)=(C**2)*(S**2)
      DO 38 K=1,3
      DO 38 J=1,3
C     DCRACK(NEL) IS THE CONCRETE STIFFNESS IN THE DIRECTION
```

```
C       PARA  EL TO THE CRACK.
        DC(J,K)=DC(J,K)*DCRACK(NEL)
   38   CONTINUE
        GO TO 41
C       ELEMENT CRACKED IN TWO DIRECTIONS REMAINS UNSTRESSED
   39   DO 40 K=1,3
        DO 40 J=1,3
        DC(J,K)=0.0
   40   CONTINUE
   41   DO 44 K=1,3
        DO 42 J=1,3
        SIGMA(K)=SIGMA(K)+DC(K,J)*E(J)
   42   CONTINUE
        IF(ICOUNT.EQ.2) GO TO 43
        SIGCON(NEL,K)=SIGMA(K)
        GO TO 44
   43   TSGCON(NEL,K)=SIGMA(K)
        IF(NCRACK(NEL).NE.0) GO TO 44
        TSGCON(NEL,K)=SIGMA(K)+TSGSHR(NEL,K)
   44   CONTINUE
C**** CALCULATION OF AGGREGATE INTERLOCK STRESS
        IF(NCRACK(NEL).NE.1) GO TO 55
        IF(NYAGG(NEL).EQ.1) GO TO 55
        IF(ICOUNT.EQ.2) GO TO 48
        DO 46 J=1,3
        E(J)=E(J)+TECON(NEL,J)
   46   CONTINUE
   48   TSGAGG(NEL)=((E(1)-E(2))*2.0*S*C+(C**2-S**2)*E(3))*DAGG(NEL)
C**** CALCULATION OF PRINCIPAL CONCRETE STRESSES AND CORRESPONDING
C       STRAINS
   55   CALL PRINCS
        CALL PRINCE
        IF(INMESH(NEL).EQ.0) GO TO 60
C**** CALCULATION OF STEEL MESH COMPONENT STRESSES AND STRAINS. WHEN
C       MESH HAS YIELDED, STRESS LEVEL REMAINS AT YIELD STRESS.
        DO 58 J=1,NDIRNS
        IF(NYMESH(NEL,J).EQ.1) GO TO 58
        II=J
        CALL MOHR(ANGLE,II,ESTEEL)
        SM=SMESH(NEL,J)
        IF(NYMESH(NEL,J).EQ.1) GO TO 58
        SIGSTL=ESTEEL*SM
        IF(ICOUNT.EQ.2) GO [illegible]
        SIGMS(NEL,J)=SIGSTL
        EMS(NEL,J)=ESTEEL
        GO TO 58
   56   TSGMS(NEL,J)=SIGSTL
        TEMS(NEL,J)=ESTEEL
   58   CONTINUE
   60   CONTINUE
C**** CALCULATION OF CONVENTIONAL AND PRESTRESS REINFORCEMENT AND BOND
C       STRESSES
C       INCLUSION OF INITIAL PRESTRESS STRAND STRESS AND STRAIN
        IF(NUMINC.GT.1.OR.NC.GE.1) GO TO 65
        DO 64 I=1,NRBO
        IF(INRTY(I).NE.0) GO TO 64
        TSGRBO(I)=TSGPRE(I)
        TERBO(I)=TEPRE(I)
   64   CONTINUE
   65   DO 110 I=1,NRBO
        IF(NYREO(I).EQ.1) GO TO 110
        I
        NODES(I,1)
        =NODES(I,2)
        CALL LOCATE(II,JJ,NROW,NCOL)
        IF(INRTY(I)) 66,66,90
   66   IF(INRDN(I)) 68,70,75
   68   LENGTH=X(NODES(I,2))-X(NODES(I,1))
        DISPL=D1(NROW)-D1(NCOL)
        GO TO 85
   70   LENGTH=Y(NODES(I,2))-Y(NODES(I,1))
        DISPL=D1(NROW+1)-D1(NCOL+1)
        GO TO 85
   75   IF(INRDN(I).GT.1) GO TO 76
        LENGTH=Z(NODES(I,2))-Z(NODES(I,1))
        NROWZ=NROW+1
        NCOLZ=NCOL+1
        IF(ICNODE(II).EQ.1) NROWZ=NROW+2
        IF(ICNODE(JJ).EQ.1) NCOLZ=NCOL+2
        DISPL=D1(NROWZ)-D1(NCOLZ)
        GO TO 85
   76   LENGTH=SQRT((Z(NODES(I,2))-Z(NODES(I,1)))**2+(Y(NODES(I,2))
       1-Y(NODES(I,1)))**2)
        C=(Y(II)-Y(JJ))/LENGTH
        S=(Z(II)-Z(JJ))/LENGTH
        IF(ICNODE(II).EQ.1) GO TO 77
        DISPII=D1(NROW+1)
        GO TO 78
   77   DISPII=C*D1(NROW+1)+S*D1(NROW+2)
   78   IF(ICNODE(JJ).EQ.1) GO TO 79
        DISPJJ=D1(NCOL+1)
        GO TO 80
   79   DISPJJ=C*D1(NCOL+1)+S*D1(NCOL+2)
   80   DISPL=DISPII-DISPJJ
   85   STRAIN=DISPL/LENGTH
        GO TO 95
   90   STRAIN=D1(NROW)-D1(NCOL)
        IF(STRAIN.LT.0.0) STRAIN=-STRAIN
   95   IF(ICOUNT.EQ.2) GO TO 100
        ERBO(I)=STRAIN
        GO TO 105
  100   TERBO(I)=STRAIN
  105   RS=RSTF(I)
        SIGR=STRAIN*RS
        IF(ICOUNT.EQ.2) GO TO 106
        SIGRBO(I)=SIGR
        GO TO 110
  106   TSGRBO(I)=SIGR
        IF(INRTY(I)) 107, 108, 110
  107   IF(RS.NE.0.0) GO TO 110
        SIGR=53802.0+430000.0*STRAIN
        IF(STRAIN.LT.0.0) SIGR=-53802.0+430000.0*STRAIN
        TSGRBO(I)=SIGR
        GO TO 110
C       WHEN PRESTRESS STRAND COMMENCES TO YIELD, INITIAL STRESSES AND
C       STRAINS MUST BE MODIFIED TO REFLECT REDUCED STRAND STIFFNESS.
  108   STRSSI=TSGPRE(I)*PREMOD/RS
        STRNI=TEPRE(I)*PREMOD/RS
        TSGRBO(I)=SIGR+STRSSI
        TERBO(I)=STRAIN+STRNI
```

```
  110 CONTINUE
      RETURN
      END
      SUBROUTINE SMSSTF(I)
C***********************************************************************
C      THIS SUBROUTINE CALCULATES THE STIFFNESS OF A PARTICULAR LAYER
C     OF A STEEL MESH FOR EITHER THE INCREMENTAL OR TOTAL LOAD CASES
C***********************************************************************
C
      COMMON/BLOCK1/NEQNS,NBAND,NBLK,NUMINC,ICOUNT
      COMMON/BLOCK2/NEL,NELT,NELCHK,NELD,INDCEL(185),INELSZ(185),
     1INDOWL(185),WIDTHC(185),CANGLE(185),CALTER(185),
     2INELTY(215),ELTHN(215),NODEEL(215,4),NDREF(215),DPMMOD(215)
      COMMON/BLOCK3/NELSH,INMESH(185),INSZMS(185),NDIRNS,PERSTL(185,4),
     1ANGLE(185,4),SMSTF(185,3,3),SMESH(185,1)
      COMMON/BLOCK6/CONMOD,NEOMOD,PREMOD,SLPMOD,SMSMOD,FC,FT,
     1FAGG,FUR,FUP,FUMS,DP,ESLIP,ECULT,ERULT,EPULT,EMSULT,P1,P2,P3,P4,
     2PU,EERMO,EEPRE,CONDEV,DEVCON,DEVREO,PREDEV,SLPDEV,IDEV,AVCSP,RELAX
      COMMON/BLCK12/SIGCON(185,3),ECON(185,3),SIGMS(185,4),EMS(185,4),
     1TSGCON(185,3),TECON(185,3),TSGCC(185),TSGCT(185),TSGAGG(185),
     2TECC(185),TECT(185),TSGMS(185,4),TEMS(185,4),SIGREO(190),
     3EREO(190),TSGREO(190),TEREO(190)
      COMMON/BLCK13/NYMESH(185,4),NDOWEL(185),NCRACK(185),NYAGG(185),
     1NYREO(190),NYCREO(190)
      INTEGER*4 CALTER
      IF(NYMESH(NEL,I).EQ.1) GO TO 40
      EE=EERMO
      IF(ICOUNT.EQ.2) GO TO 10
      ASTRSS=TSGMS(NEL,I)+SIGMS(NEL,I)
      STRAIN=TEMS(NEL,I)+EMS(NEL,I)
      IF(STRAIN.GT.-EE.AND.STRAIN.LT.EE) GO TO 5
      IF(STRAIN.LE.-EMSULT.OR.STRAIN.GE.EMSULT) GO TO 25
      SMESH(NEL,I)=430000.0
      RETURN
    5 SMESH(NEL,I)=SMSMOD
      RETURN
   10 ASTRSS=TSGMS(NEL,I)
      STRAIN=TEMS(NEL,I)
      IF(STRAIN.LT.-EMSULT.OR.STRAIN.GT.EMSULT) GO TO 25
      IF(STRAIN.GE.-EE.AND.STRAIN.LE.EE) GO TO 5
      STRESS=53800.0+430000.0*STRAIN
      PERCTG=(ASTRSS-STRESS)*100.0/STRESS
      IF(PERCTG.GE.PREDEV) GO TO 20
      SMESH(NEL,I)=STRESS/STRAIN
      RETURN
C     RELAXATION IS EMPLOYED TO INCREASE RATE OF CONVERGENCE
   20 ASTRN=STRAIN*RELAX
      STRESS=53800.0+430000.0*ASTRN
      SMESH(NEL,I)=STRESS/ASTRN
      RETURN
   25 NYMESH(NEL,I)=1
      TSGMS(NEL,I)=FUMS
      IF(STRAIN.LT.0.0) TSGMS(NEL,I)=-FUMS
      SM=0.0
      WRITE (6,30) I,NEL
   30 FORMAT(/' **** STEEL MESH LAYER NUMBER',I4,' IN ELEMENT NUMBER ',I
     15,' HAS FAILED **** '/)
      RETURN
   40 SM=0.0
      RETURN
      END
      SUBROUTINE PRESET
C***********************************************************************
C      THIS SUBROUTINE INITIALISES ALL VARIABLES, VECTORS AND ARRAYS
C     WHOSE INCORPORATION INTO THE PROGRAM LOGIC NECESSITATES SUCH
C     PRESETTING
C***********************************************************************
C
      COMMON/BLOCK1/NEQNS,NBAND,NBLK,NUMINC,ICOUNT
      COMMON/BLOCK2/NEL,NELT,NELCHK,NELD,INDCEL(185),INELSZ(185),
     1INDOWL(185),WIDTHC(185),CANGLE(185),CALTER(185),
     2INELTY(215),ELTHN(215),NODEEL(215,4),NDREF(215),DPMMOD(215)
      COMMON/BLOCK3/NELSH,INMESH(185),INSZMS(185),NDIRNS,PERSTL(185,4),
     1ANGLE(185,4),SMSTF(185,3,3),SMESH(185,1)
      COMMON/BLOCK4/NR,NREO,NODES(190,2),INNDN(190),INRTY(190),
     1RALTER(190),RAREA(190),CAREA(190),RSTF(190),TSGPRE(190),TEPRE(190)
      COMMON/BLCK11/RTOT(750),REOTOT(750),DTOT(750),R2(2,750),D2(2,750)
      COMMON/BLCK12/SIGCON(185,3),ECON(185,3),SIGMS(185,4),EMS(185,4),
     1TSGCON(185,3),TECON(185,3),TSGCC(185),TSGCT(185),TSGAGG(185),
     2TECC(185),TECT(185),TSGMS(185,4),TEMS(185,4),SIGREO(190),
     3EREO(190),TSGREO(190),TEREO(190)
      COMMON/BLCK13/NYMESH(185,4),NDOWEL(185),NCRACK(185),NYAGG(185),
     1NYREO(190),NYCREO(190)
      DIMENSION A(185,33),B(190,4),C(750,3),NE(185,7)
      INTEGER*4 CALTER,RALTER
C     SIZES OF ARRAYS IN DIMENSION BLOCK ARE AS BELOW
C     A(NELT,17+4*NDIRNS)  B(NREO,4)  C(NEQNS,2)  NE(NELT,3+NDIRNS)
      EQUIVALENCE (SIGCON(1,1),A(1,1)),(SIGREO(1),B(1,1)),
     1(RTOT(1),C(1,1)),(NYMESH(1,1),NE(1,1))
      DO 5 J=1,33
      DO 5 I=1,NELT
      A(I,J)=0.0
    5 CONTINUE
      DO 10 J=1,4
      DO 10 I=1,NREO
      B(I,J)=0.0
   10 CONTINUE
      DO 15 J=1,3
      DO 15 I=1,NEQNS
      C(I,J)=0.0
   15 CONTINUE
      DO 25 J=1,7
      DO 25 I=1,NELT
      NE(I,J)=0
   25 CONTINUE
      DO 30 I=1,NREO
      NYREO(I)=0
      NYCREO(I)=0
   30 CONTINUE
      RETURN
      END
      SUBROUTINE FAIL
C***********************************************************************
C      THIS SUBROUTINE CHECKS THAT NO STRUCTURAL ELEMENTS HAVE FAILED
C     RESULTING IN COMPLETE STRUCTURAL FAILURE.
C***********************************************************************
C
      COMMON/BLOCK1/NEQNS,NBAND,NBLK,NUMINC,ICOUNT
      COMMON/BLOCK2/NEL,NELT,NELCHK,NELD,INDCEL(185),INELSZ(185),
     1INDOWL(185),WIDTHC(185),CANGLE(185),CALTER(185),
```

```
     2INELTY(215),ELTHN(215),NODEEL(215,4),NODEF(215),DPHMOD(215)
      COMMON/BLOCK4/NR,NREO,NODES(190,2),INRDN(190),INRTY(190),
     1RALTER(190),RAREA(190),CAREA(190),BSTP(190),TSGPRE(190),TEPRE(190)
      COMMON/BLOCK6/CONMOD,REOMOD,PREMOD,SLPMOD,SHSMOD,FC,FT,
     1FAGG,FUR,FUP,FUHS,DF,ESLIP,ECULT,ERULT,EPULT,EHSULT,P1,P2,P3,P4,
     2PU,EERREO,EEPRE,CONDEV,DEVCON,DEVREO,PREDEV,SLPDEV,IDEV,AVCSP,RELAX
      COMMON/BLCK12/SIGCON(185,3),ECON(185,3),SIGHS(185,4),EHS(185,4),
     1TSGCON(185,3),TECON(185,3),TSGCC(185),TSGCT(185),TSGAGG(185),
     2TECC(185),TECT(185),TSGHS(185,4),TEHS(185,4),SIGREO(190),
     3EREO(190),TSGREO(190),TEREO(190)
      INTEGER*4 CALTER,RALTER
C     CHECK ON CRUSHING FAILURE OF CONCRETE
    5 DO 50 I=1,NELT
      NEL=I
      IF(TSGCC(I).GE.0.0) GO TO 50
      IF(TSGCT(I).LT.0.0) GO TO 10
      IF(TSGCC(I).LE.FC) GO TO 100
      GO TO 50
   10 ALPHA=TSGCT(I)/TSGCC(I)
      IF(ALPHA.GT..2) GO TO 20
      FFAIL=(1.0+ALPHA)*FC
      GO TO 30
   20 FFAIL=1.2*FC
   30 IF(TSGCC(NEL).LE.FFAIL) GO TO 100
   50 CONTINUE
C     CHECK ON FAILURE OF PRESTRESS STRANDS
      DO 90 I=1,NREO
      IF(INRTY(I).NE.0) GO TO 90
      IF(TEREO(I).GE.EPULT) GO TO 110
   90 CONTINUE
      RETURN
  100 WRITE (6,105) I,TSGCC(I),TSGCT(I)
      WRITE (7,105) I,TSGCC(I),TSGCT(I)
  105 FORMAT(' ************************',/,' ***  BEAM  FAILURE  ***',/,'
     1 ************************',//,' ELEMENT NUMBER',I5,' HAS FAILED BY
     2CRUSHING OF CONCRETE',/,' PRINCIPAL COMPRESSIVE STRESS =',E13.5/,
     3' PRINCIPAL TENSILE STRESS =',E13.5)
      STOP
  110 WRITE (6,115) I,TEREO(I)
      WRITE (7,115) I,TEREO(I)
  115 FORMAT(' ************************',/,' ***  BEAM  FAILURE  ***',/,'
     1' ************************',//,' PRESTRESS REINFORCEMENT NUMBER ',I5
     2,' HAS FAILED IN TENSION',/,' TENSILE STRAIN IN CABLE AT FAILURE =
     2',E13.5)
      STOP
      END
      SUBROUTINE DEVIAT(ID)
C***********************************************************************
C     THIS SUBROUTINE CHECKS THE (STRESS,STRAIN) CONDITION OF ALL
C     MATERIALS TO DETECT IF ANY DEVIATION FROM THE RESPECTIVE
C     MATERIAL BEHAVIOUR HAS OCCURRED. ALSO, THE AGGREGATE INTERLOCK
C     LINKAGE FOR CRACKED ELEMENTS IS CHECKED FOR FAILURE.
C***********************************************************************
C
      COMMON/BLOCK1/NEQNS,NBAND,NBLK,NUMINC,ICOUNT
      COMMON/BLOCK2/NEL,NELT,NELCHK,NELD,INDCEL(185),INELSZ(185),
     1INDOWL(185),WIDTHC(185),CANGLE(185),CALTER(185),
     2INELTY(215),ELTHN(215),NODEEL(215,4),NODEF(215),DPHMOD(215)
      COMMON/BLOCK3/NELSH,INHESH(185),INSZHS(185),NDIRNS,PERSTL(185,4),
     1ANGLE(185,4),SHSTF(185,3,3),THESH(185,1)
      COMMON/BLOCK4/NR,NREO,NODES(190,2),INRDN(190),INRTY(190),
     1RALTER(190),RAREA(190),CAREA(190),BSTP(190),TSGPRE(190),TEPRE(190)
      COMMON/BLOCK6/CONMOD,REOMOD,PREMOD,SLPMOD,SHSMOD,FC,FT,
     1FAGG,FUR,FUP,FUHS,DF,ESLIP,ECULT,ERULT,EPULT,EHSULT,P1,P2,P3,P4,
     2PU,EERREO,EEPRE,CONDEV,DEVCON,DEVREO,PREDEV,SLPDEV,IDEV,AVCSP,RELAX
      COMMON/BLCK12/SIGCON(185,3),ECON(185,3),SIGHS(185,4),EHS(185,4),
     1TSGCON(185,3),TECON(185,3),TSGCC(185),TSGCT(185),TSGAGG(185),
     2TECC(185),TECT(185),TSGHS(185,4),TEHS(185,4),SIGREO(190),
     3EREO(190),TSGREO(190),TEREO(190)
      COMMON/BLCK10/ESTF(12,12),BS(3,3),B1(3,12,9),B2(3,12,9),D(3,3),
     1DCONC(185,3,3),DCRACK(185),DAGG(185)
      COMMON/BLCK13/NYMESH(185,4),NDOWEL(185),NCRACK(185),NYAGG(185),
     1NYREO(190),NYCREO(190)
      COMMON/BLCK14/IDEPLN,ICON3,ICONPR,INPSH,IREO,ILOAD,ITLOAD,ISTIF,
     1MAXIT,NWTCON,NWTREO,NWTPRE
      ID=0
C     IF ANY SIGNIFICANT DEVIATION OCCURS, ID IS INCREMENTED.
C     FIRSTLY, CONCRETE BEHAVIOUR IN PRINCIPAL DIRECTIONS IS CHECKED
      DO 20 I=1,NELT
      IF(NCRACK(I).EQ.-1) GO TO 20
      IF(NCRACK(I).LT.-1.OR.NCRACK(I).GT.1) GO TO 12
      NEL=I
      CALL PRINCS
      CALL PRINCE
      STRESS=TSGCC(NEL)
      STRSO=TSGCT(NEL)
      STRAIN=TECC(NEL)
      IF(STRESS.GE.-1000.0.OR.STRAIN.GE.0.0) GO TO 12
C     IF THE PRINCIPAL COMPRESSIVE STRESS IS LESS THAN 1000 PSI,
C     DEVIATION WILL BE SMALL IN ALL OF THE CALCULATIONS BELOW. SIGA
C     IS THE CORRECT VALUE OF STRESS THAT CORRESPONDS TO THE
C     MATERIAL ELEMENT STRAIN.
      ES=FC/ECULT
      ALPHA=STRSO/STRESS
      IF(STRESS.GE.0.0.AND.STRSO.GE.0.0) P=P2
      IF(STRESS.LT.0.0.AND.STRSO.LT.0.0) P=P1
      IF(STRESS.LT.0.0.AND.STRSO.GE.0.0) P=P3
      IF(NCRACK(NEL).NE.0) GO TO 5
      SIGA=STRAIN*CONMOD/((1.0-P*ALPHA)*(1.0+(CONMOD/((1.0-PU*ALPHA)*ES
     1)-2.0)*STRAIN/ECULT+(STRAIN/ECULT)**2))
      GO TO 10
    5 SIGA=STRAIN*CONMOD/(1.0+(CONMOD/ES-2.0)*STRAIN/ECULT+
     1(STRAIN/ECULT)**2)
   10 PERCTG=(STRESS-SIGA)*100.0/SIGA
      CNDEV1=CONDEV*2.0
      IF(STRESS.LT.-1000.0) GO TO 11
      IF(PERCTG.GE.-CONDEV.AND.PERCTG.LE.CNDEV1) GO TO 12
   11 IF(PERCTG.GE.-CONDEV.AND.PERCTG.LE.CONDEV) GO TO 12
      IF(PERCTG.LT.-CONDEV.AND.NUMINC.EQ.1) GO TO 12
      WRITE (6,100) I,STRESS,STRSO,STRAIN,ALPHA,SIGA,PERCTG
      WRITE (7,100) I,STRESS,STRSO,STRAIN,ALPHA,SIGA,PERCTG
  100 FORMAT(/' CONCRETE DEVIATION CHECK FOR ELEMENT NO.',I3/
     1,' STRESS =',E14.5,'   STRSO =',E14.5,'   STRAIN =',E14.5/
     2,'   ALPHA =',E14.5,'   SIGA =',E14.5,'   PERCTG =',E14.5)
      WRITE (6,120) ICOUNT,(TSGCON(NEL,J),J=1,3),(TECON(NEL,J),J=1,3)
  120 FORMAT(' ICOUNT =',I2/,' TSGCON(NEL,3) =',3E14.5/,
     1' TECON(NEL,3) =',3E14.5)
      ID=ID+NWTCON
      WRITE (6,105)
      WRITE (7,105)
```

```
  105 FORMAT(' ***** ELEMENT HAS DEVIATED *****')
C     STEEL MESH IS CHECKED FOR DEVIATION. IF DEVIAION IS SIGNIFICANT
C     IN MORE THAN TWO ELEMENT MESHES, ITERATION WILL BE INVOKED.
   12 DO 17 J=1,NDIRNS
      IF(NYMESH(NEL,J).EQ.1) GO TO 17
      IF(ICOUNT.EQ.2) GO TO 13
      ASTRSS=TSGMS(NEL,J)+SIGMS(NEL,J)
      STRAIN=TEMS(NEL,J)+EMS(NEL,J)
      GO TO 14
   13 ASTRSS=TSGMS(NEL,J)
      STRAIN=TEMS(NEL,J)
   14 IF(STRAIN.LT..00185) GO TO 17
      STRESS=53800.0+STRAIN*430000.0
      PERCTG=(ASTRSS-STRESS)*100.0/STRESS
      IF(PERCTG.LT.PREDEV) GO TO 17
      WRITE (6,160) J,NEL,ASTRSS,STRAIN,STRESS,PERCTG
      WRITE (7,160) J,NEL,ASTRSS,STRAIN,STRESS,PERCTG
  160 FORMAT(//' **** STEEL MESH DIRN.',I2,' IN ELEMENT NUMBER',I4,
     1' HAS DEVIATED SIGNIFICANTLY ****',/,' ASTRSS=',E14.5,
     2' STRAIN=',E14.5,' STRESS=',E14.5,' PERCTG=',E14.5)
      ID=ID+0
   17 CONTINUE
C     AGGREGATE INTERLOCK LINKAGE FAILURE CHECK
      IF(NCRACK(NEL).NE.1) GO TO 20
      IF(TSGAGG(NEL).GT.FAGG.AND.TSGAGG(NEL).LT.-FAGG) GO TO 20
      NYAGG(NEL)=1
      WRITE (6,130) NEL,TSGAGG(NEL)
  130 FORMAT(//' ***** AGGREGATE INTERLOCK LINKAGE FOR ELEMENT NO.',I3,
     1' HAS FAILED ****',/,'   SHEAR STRESS ACROSS CRACK =',E14.5)
      TSGAGG(NEL)=0.0
      DAGG(NEL)=100.0
      GO TO 20
   20 CONTINUE
C     REINFORCEMENT ELEMENTS' BEHAVIOUR IS EXAMINED TO DETECT ANY
C     SIGNIFICANT DEVIATION. NEITHER CONVENTIONAL REINFORCEMENT
C     NOR BOND LINKAGE DEVIATION WILL INITIATE A FURTHER ITERATION
C     CYCLE.
      DO 35 I=1,NREO
      IF(NYREO(I).EQ.1) GO TO 35
      IF(ICOUNT.EQ.2) GO TO 24
      STRESS=SIGREO(I)+TSGREO(I)
      STRAIN=EREO(I)+TEREO(I)
      GO TO 25
   24 STRESS=TSGREO(I)
      STRAIN=TEREO(I)
   25 IF(INRTY(I)) 26,27,28
C     CHECK TO DETECT CONVENTIONAL REINFORCEMENT ELEMENT THAT HAS
C     JUST YIELDED
   26 IF(STRAIN.LT.EEREO.OR.NYCREO(I).EQ.1) GO TO 35
      NYCREO(I)=1
      ID=ID+NNTREO
      WRITE (6,140) I,STRESS, STRAIN
      WRITE (7,140) I,STRESS, STRAIN
  140 FORMAT(/' CONVENTIONAL REINFORCEMENT ELEMENT NUMBER',I3,
     1' HAS JUST YIELDED',/,' STRESS=',E14.5,' STRAIN=',E14.5)
      GO TO 35
C     DEVIATION CHECK FOR PRESTRESS STRAND
   27 IF(STRAIN.GT.EEPRE) NYCREO(I)=1
      IF(STRAIN.LE.EEPRE) GO TO 35
      SIGA=255360.0+470000.0*STRAIN
      DEV=PREDEV
C     THE STRESS-STRAIN STATE OF ANY PRESTRESS STRAND THAT HAS ALREADY
C     YIELDED IS PRINTED OUT
      IF(NYCREO(I).NE.1) GO TO 29
      WRITE (6,150) I,STRESS,STRAIN
      WRITE (7,150) I,STRESS,STRAIN
  150 FORMAT(//' **** PRESTRESS ELEMENT NUMBER',I3,' IS YIELDING ****'
     1,/,' STRESS=',E14.5,'  STRAIN=',E14.5)
      GO TO 29
   28 DD=STRAIN
      SIGA=(1950000.0*DD-2.35E+09*DD**2+1.39E+12*DD**3-.33E+15*DD**4)*
     1SQRT(-FC/5000.0)
      DEV=SLPDEV
   29 IF(STRESS.LT.0.0) SIGA=-SIGA
      PERCTG=(STRESS-SIGA)*100.0/SIGA
      IF(PERCTG.GT.-DEV.AND.PERCTG.LT.DEV) GO TO 35
      IF(INRTY(I).EQ.1) GO TO 32
      WRITE (6,110) I,STRESS,STRAIN,SIGA,PERCTG
      WRITE (7,110) I,STRESS,STRAIN,SIGA,PERCTG
  110 FORMAT(/' PRESTRESS ELEMENT NUMBER ',I3,
     1'HAS DEVIATED SIGNIFICANTLY',/,' STRESS=',E14.5,
     2' STRAIN=',E14.5,'  SIGA=',E14.5,'  PERCTG =',E14.5)
      ID=ID+NNTPRE
      GO TO 35
   32 IF(STRAIN.LT..0001) GO TO 35
      WRITE (6,33) I,PERCTG,DEV
   33 FORMAT(/' BOND SLIP REINFORCEMENT NO.',I3,' HAS DEVIATED SIGNIFICA
     1NTLY',/,' PERCENTAGE DEVIATION =',E15.5,
     2' WHILE ALLOWABLE DEVIATION PERCENTAGE =',E15.5/)
   35 CONTINUE
      RETURN
      END
END OF FILE
```

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LIST FILE52(1,1721)
      SUBROUTINE TOTSTF(NS1,NS2,NS3,TS,AL,AR)
C*********************************************************************
C       THIS SUBROUTINE ASSEMBLES THE TOTAL STIFFNESS MATRIX BLOCK BY
C      BLOCK (EACH BLOCK BEING A BAND WIDTH SQUARE). AS EACH BLOCK IS
C      COMPLETELY FILLED,IT IS WRITTEN OUT ON DISC STORAGE
C*********************************************************************
C
      COMMON/BLOCK1/NEQNS,NBAND,NBLK,NUMINC,ICOUNT
      COMMON/BLOCK2/NEL,NELT,NELCHK,NELD,INDCEL(185),INELSZ(185),
     1INDONL(185),WIDTHC(185),CANGLE(185),CALTER(185),
     2INELTY(215),ELTHN(215),NODEEL(215,4),NDREF(215),DPMMOD(215)
      COMMON/BLOCK3/NELSH,INMESH(185),INSZMS(185),NDIRMS,PERSTL(185,4),
     1ANGLE(185,4),SHSTF(185,3,3),SHMSH(185,1)
      COMMON/BLOCK4/NR,NREO,NODES(190,2),INRDN(190),INRTY(190),
     1NALTER(190),RARRA(190),CAREA(190),RSTF(190),FSGPRE(190),TEPRE(190)
      COMMON/BLOCK5/NNODES,ICNODE(220),X(220),Y(220),Z(220)
      COMMON/BLOCK6/CONMOD,REOMOD,PREMOD,SLPMOD,SHSMOD,FC,FT,
     1FAGG,FUR,FUP,FUMS,DP,ESLIP,ECULT,ERULT,EPULT,EMSULT,P1,P2,P3,P4,
     2PU,ERREO,ERPRE,CONDEV,DEVCON,DEVREO,PREDEV,SLPDEV,IDEV,AVCSP,RELAX
      COMMON/BLOCK9/LPOINT(20,2),FDUB1,FDUB2,NUMREC,NUMBLK,
     1ISOLVE,LEN
      COMMON/BLCK10/ESTF(12,12),BS(3,3),B1(3,12,9),B2(3,12,9),D(3,3),
     1DCONC(185,3,3),DCRACK(185),DAGG(185)
      COMMON/BLCK13/NYMESH(185,4),NDOWEL(185),NCRACK(185),NYAGG(185),
     1NYREO(190),NYCREO(190)
      COMMON/BLCK14/IDEPLN,ICON3,ICONPR,IMESH,IREO,ILOAD,ITLOAD,ISTIF,
     1MAXIT,NNTCON,NNTREO,NNTPRE
      DIMENSION TS(NS1,NS2),AL(NS3),AR(NS3),DC(3,3),DSM(3,3)
      INTEGER*4 FDUB1,FDUB2,INFO(4),CALTER,RALTER
      INTEGER*2 LEN
      DATA MOD/Z00020000/
C     CALCULATION OF NUMBER OF RECORDS OR LINES PLACED IN FILE
      ISOLVE=0
      NBYTES=(NBAND**2)*4
      ANUMRC=FLOAT(NBYTES)/LEN
      NUMREC=NBYTES/LEN
      IF((ANUMRC-NUMREC).GT.0.0) NUMREC=NUMREC+1
      WRITE (6,600) NBYTES,LEN,NUMREC
  600 FORMAT(/' NBYTES =',I6,' LEN =',I6,' NUMREC =',I2)
      NUMBLK=NEQNS/NBAND
      ANUMBK=FLOAT(NEQNS)/NBAND
      IF((ANUMBK-NUMBLK).GT.0.0) NUMBLK=NUMBLK+1
C     INITIAL ZEROING OF STIFFNESS BLOCK IN CORE-TS(NBAND,NBAND*2)
      J1=NBAND*2
      DO 5 J=1,J1
      DO 5 I=1,NBAND
      TS(I,J)=0.0
    5 CONTINUE
C     FORMULATION OF ELEMENTARY CONCRETE STIFFNESS MATRIX
      DO 6 J=1,3
      DO 6 I=1,3
      DC(I,J)=0.0
    6 CONTINUE
      DO 7 I=1,2
      DC(I,I)=1.0
      DC(I,3-I)=P1
    7 CONTINUE
      DC(3,3)=(1.0-P1)/2.0
      DO 8 J=1,3
      DO 8 I=1,3
      DC(I,J)=DC(I,J)*CONMOD/(1.0-P1**2)
      DO 8 K=1,NELT
      DCONC(K,I,J)=DC(I,J)
    8 CONTINUE
      DO 9 I=1,NELT
      DAGG(I)=DC(3,3)
    9 CONTINUE
      NBLK=0
      MAXOLD=0
      NELOLD=0
      NELSHS=0
   10 NBLK=NBLK+1
      IF(NBLK.GT.NUMBLK) GO TO 300
C     CALCULATION OF THE LOWEST AND HIGHEST NODE NUMBERS(NMAX AND NMIN)
C     WHOSE STIFFNESSES ARE TO BE ADDED INTO THE TOTAL STIFFNESS
C      BLOCK NUMBER NBLK. ALSO, THE HIGHEST NODE NUMBER (NMXOLD) OF
C     THE PREVIOUS BLOCK IS CALCULATED TO BE USED IN CHECKING WHICH
C     ELEMENTS HAVE ALREADY BEEN ADDED INTO PREVIOUS BLOCKS.
      MAXNEW=NBLK*NBAND
      IF(MAXNEW.GT.NEQNS) MAXNEW=NEQNS
      MINEQN=(NBLK-1)*NBAND+1
      MAXOLD=(NBLK-1)*NBAND
      IF(NBLK.EQ.1) MAXOLD=0
      NEQ=MAXNEW
      CALL NODLOC(NEQ,NN)
      NMAX=NN
      NEQ=MINEQN
      CALL NODLOC(NEQ,NN)
      NMIN=NN
      NEQ=MAXOLD
      CALL NODLOC(NEQ,NN)
      NMXOLD=NN
      IF(ISTIF.EQ.0) GO TO 12
      WRITE (6,605) NBLK,NMIN,NMAX,NMXOLD
  605 FORMAT(///' **** NBLK=',I2,' *****',/,' NMIN=',I3,'     NMAX=',I3
     1,'    NMXOLD(PREVIOUS NMAX) =',I3)
C     SEARCH FOR CONCRETE ELEMENTS CONTRIBUTING TO STIFFNESS OF BLOCK
C     UNDER CONSIDERATION
   12 NELTOT=NELT+NELD
      DO 150 I=1,NELTOT
      NEL=I
      DO 100 J=1,4
      IF((NODEEL(I,J).LT.NMIN).OR.(NODEEL(I,J).GT.NMAX)) GO TO 100
      DO 15 K=1,4
      IF(NODEEL(I,K).LE.NMXOLD) GO TO 150
   15 CONTINUE
C     CHECK TO DETECT A DIAPHRAGM ELEMENT
      IF(INELTY(NEL).GE.3) GO TO 60
C     BEFORE CONCRETE ELEMENT STIFFNESS IS ADDED IN,THE PRESENCE OF ANY
C     STEEL REINFORCING MESH IS VERIFIED
      IF(INMESH(NEL).EQ.0) GO TO 30
C     BY ADDING CONCRETE AND STEEL MESH STIFFNESSES,STIFFNESS OF
C     REINFORCED CONCRETE ELEMENT IS ACHIEVED IN ONE FORMULATION
      CALL STFSH(DSM,NELSHS)
      DO 20 L=1,3
      DO 20 K=1,3
      D(K,L)=DC(K,L)+DSM(K,L)
      SHSTF(NEL,K,L)=DSM(K,L)
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   20 CONTINUE
      GO TO 40
   30 DO 35 L=1,3
      DO 35 K=1,3
      D(K,L)=DC(K,L)
   35 CONTINUE
   40 CALL ELSTF(NELOLD)
      IF(ISTIF.EQ.0) GO TO 50
      WRITE (6,610) I
  610 FORMAT(/' ELEMENTARY STIFFNESS MATRIX D(3,3) FOR ELEMENT NUMBER',I
     13,' IS AS BELOW')
      DO 620 K=1,3
      WRITE (6,615) (D(K,L),L=1,3)
  615 FORMAT(3E16.5)
  620 CONTINUE
C     PARTITIONING OF ELEMENT STIFFNESS INTO BLOCKS,AND ADDITION OF
C     BLOCKS INTO TS(NBAND,NBAND*2)
   50 CALL ADDNEL(NS1,NS2,NS3,TS,AL,AR)
      GO TO 150
C     INCLUSION OF DIAPHRAGM STIFFNESS
   60 IF(INELTY(NEL).GT.4) GO TO 70
      CALL DIAPHM(TS,NS1,NS2)
      GO TO 150
C     INCLUSION OF WARPING RESTRAINT WHERE APPLICABLE
   70 CALL WARP(TS,NS1,NS2)
      GO TO 150
  100 CONTINUE
  150 CONTINUE
C     ADDITION OF STIFFNESSES OF SINGLE REINFORCING AND PRESTRESS BARS
C     AND BOND SLIP LINKAGES TO TS(NBAND,NBAND*2)
      DO 250 I=1,NREO
      NR=I
      DO 225 J=1,2
      IF((NODES(I,J).LT.NMIN).OR.(NODES(I,J).GT.NMAX)) GO TO 225
      DO 210 K=1,2
      IF(NODES(I,K).LE.NHIOLD) GO TO 250
  210 CONTINUE
      IF(ISTIF.EQ.0) GO TO 212
      WRITE (6,625) I,NBLK
  625 FORMAT(/' REINFORCEMENT ELEMENT NUMBER',I3,' HAS BEEN ADDED INTO B
     1LOCK NO.',I2)
  212 IF(INRTY(I)) 214,216,218
  214 STIF=REOMOD
      GO TO 220
  216 STIF=PREMOD
      GO TO 220
  218 STIF=SLPMOD
  220 RSTF(I)=STIF
      CALL STFADD(NS1,NS2,NS3,TS,AL,AR,STIF)
      GO TO 250
  225 CONTINUE
  250 CONTINUE
C     STIFFNESS BLOCK IS WRITTEN ON TEMPORARY DISC FILE -FILE1
      CALL NOTE(FDUB1,INFO,&510)
      IF(ISTIF.EQ.0) GO TO 255
      WRITE (6,630) NBLK,INFO(2)
  630 FORMAT(//' WRITE POINTER FOR BLOCK NUMBER',I3,' =',I8)
  255 LPOINT(NBLK,1)=INFO(2)
      CALL MODIFY(NS1,NS2,NS3,TS,AL,AR)
      J=1
      J1=LEN
      DO 260 I=1,NUMREC
      CALL WRITE(AL(J),LEN,0,LNUM,FDUB1,&520)
      J=J+LEN/4
      IF(I.EQ.(NUMREC-1)) LEN=(NS3-J+1)*4
  260 CONTINUE
      LEN=J1
C     SHIFTING OF SECOND UNCOMPLETED BLOCK INTO FIRST BLOCK LOCATION
      DO 270 I=1,NS3
      AL(I)=AR(I)
      AR(I)=0.0
  270 CONTINUE
      GO TO 10
  300 WRITE (6,310)
  310 FORMAT(/' STRUCTURE STIFFNESS MATRIX HAS BEEN FULLY ASSEMBLED AND
     1WRITTEN ON -FILE1'/)
      GO TO 320
  500 WRITE (6,505)
  505 FORMAT(//' *** ERROR IN GETTING FILE -FILE1 ***'//)
      STOP
  510 WRITE (6,515)
  515 FORMAT(//' *** ERROR IN SUBROUTINE NOTE ***'//)
      STOP
  520 WRITE (6,525) NBLK,NUMREC
  525 FORMAT(//' *** ERROR IN WRITING BLOCK',I3,' ,RECORD',I3,' ON DISC
     1***'//)
      STOP
  530 WRITE (6,535)
  535 FORMAT(//' *** ERROR IN WRITING OUT LAST BLOCK ***'//)
      STOP
  320 RETURN
      END
      SUBROUTINE STFSH(DSH,NELSHS)
C*************************************************************************
C      THIS SUBROUTINE CALCULATES THE ELEMENTARY STIFFNESS MATRIX
C     DSH(3,3) FOR STEEL MESH REINFORCEMENT
C*************************************************************************
C
      COMMON/BLOCK1/NEQNS,NBAND,NBLK,NUMINC,ICOUNT
      COMMON/BLOCK2/NEL,NELT,NELCHK,NELD,INDCEL(185),INELSZ(185),
     1INDOWL(185),WIDTHC(185),CANGLE(185),CALTER(185),
     2INELTY(215),ELTHN(215),NODEEL(215,4),NDREF(215),DPHMOD(215)
      COMMON/BLOCK3/NELSH,INMESH(185),INSZHS(185),NDIRNS,PERSTL(185,4),
     1ANGLE(185,4),SHSTF(185,3,3),SHESH(185,1)
      COMMON/BLOCK6/CONMOD,REOMOD,PREMOD,SLPMOD,SHSMOD,FC,FT,
     1FAGG,FUR,FUP,FUMS,DP,ESLIP,ECULT,ERULT,EPULT,EMSULT,P1,P2,P3,P4,
     2PU,EERRO,ERPRE,CONDEV,DEVCON,DEVREO,PREDEV,SLPDEV,IDEV,AVCSP,RELAX
      COMMON/BLCK13/NYMESH(185,4),NDOWEL(185),NCRACK(185),NYAGG(185),
     1NYREO(190),NYCREO(190)
      COMMON/BLCK14/IDEFLN,ICON3,ICONPR,IMESH,IREO,ILOAD,ITLOAD,ISTIF,
     1MAXIT,NNTCON,NNTREO,NNTPRE
      INTEGER*4 CALTER
      DIMENSION DD(3,3),DSH(3,3)
      IF(NELSHS.EQ.0) GO TO 10
   10 NELSHS=1
      DO 20 J=1,3
      DO 20 I=1,3
      DSH(I,J)=0.0
   20 CONTINUE
      DO 50 I=1,NDIRNS
```

```
      IF(NYMESH(NEL,I).EQ.1) GO TO 50
      THETA=ANGLE(NEL,I)*3.141593/180.0
      C=COS(THETA)
      S=SIN(THETA)
      DD(1,1)=C**4
      DD(2,1)=C**2*S**2
      DD(3,1)=C**3*S
      DD(1,2)=DD(2,1)
      DD(2,2)=S**4
      DD(3,2)=C*S**3
      DD(1,3)=DD(3,1)
      DD(2,3)=DD(3,2)
      DD(3,3)=DD(2,1)
   25 CALL SHSSTF(I)
      IF(NYMESH(NEL,I).EQ.1) GO TO 50
      DO 30 K=1,3
      DO 30 J=1,3
      DSH(J,K)=DSH(J,K)+DD(J,K)*PERSTL(NEL,I)*SMESH(NEL,I)/100.0
   30 CONTINUE
   50 CONTINUE
   60 IF(ISTIF.EQ.0) RETURN
      WRITE (6,638)
  638 FORMAT(///' **** OUTPUT FROM STFSH ****')
      WRITE (6,640) NEL
  640 FORMAT(/' THE ELEMENTARY STEEL MESH STIFFNESS MATRIX DSH(3,3) FOR
     1ELEMENT NUMBER',I3,' IS AS BELOW'/)
      DO 650 I=1,3
      WRITE (6,645) (DSH(I,J),J=1,3)
  645 FORMAT(3E15.5)
  650 CONTINUE
  100 RETURN
      END
      SUBROUTINE ELSTF(NELOLD)
C**********************************************************************
C       THIS SUBROUTINE CALCULATES THE MCCLEOD FINITE ELEMENT STIFFNESS
C     OF A RECTANGULAR ELEMENT WHOSE ELEMENTARY STIFFNESS MATRIX IS
C     D(3,3)
C**********************************************************************
C
      COMMON/BLOCK1/NEQNS,NBAND,NBLK,NUMINC,ICOUNT
      COMMON/BLOCK2/NEL,NELT,NELCHK,NELD,INDCEL(185),INELSZ(185),
     1INDOWL(185),WIDTHC(185),CANGLE(185),CALTER(185),
     2INELTY(215),ELTHN(215),NODEEL(215,4),NDREF(215),DPHMOD(215)
      COMMON/BLOCK3/NELSH,INMESH(185),INSZHS(185),NDIRNS,PERSTL(185,4),
     1ANGLE(185,4),SHSTF(185,3,3),SMESH(185,1)
      COMMON/BLOCK5/NNODES,ICNODE(220),X(220),Y(220),Z(220)
      COMMON/BLCK10/ESTF(12,12),BS(3,3),B1(3,12,9),B2(3,12,9),D(3,3),
     1DCONC(185,3,3),DCRACK(185),DAGG(185)
      COMMON/BLCK13/NYMESH(185,4),NDOWEL(185),NCRACK(185),NYAGG(185),
     1NYZRO(190),NYCRRO(190)
      COMMON/BLCK14/IDEFLN,ICON3,ICONPR,INESH,IREO,ILOAD,ITLOAD,ISTIP,
     1MAXIT,NUTCON,NUTRRO,NUTPRE
      INTEGER*4 CALTER
      DIMENSION W(3),G(3),STF1(12,12),STF2(12,12),CC1(3,12),CC2(3,12)
      DATA W/.5555556,.8888889,.5555556/,G/-.7745966 7,0.0,.77459667/
C
      IPRINT=0
      IF(NUMINC.GE.0) GO TO 3
      IF(NELOLD.EQ.0) GO TO 2
      IF(INELSZ(NEL).NE.INELSZ(NELOLD)) GO TO 2
      IF((INMESH(NEL).EQ.INMESH(NELOLD)).AND.(INSZHS(NEL).EQ.INSZHS(
     1NELOLD))) GO TO 100
C     CALCULATION OF ELEMENT DIMENSIONS
    2 NELOLD=NEL
      IPRINT=1
C     NELOLD IS THE ELEMENT NUMBER OF THE LAST ELEMENT WHOSE STIFFNESS
C     WAS CALCULATED.
    3 A=(X(NODEEL(NEL,3))-X(NODEEL(NEL,2)))/2.0
      IF(INDCEL(NEL)) 4,6,8
    4 B=(SQRT((Y(NODEEL(NEL,2))-Y(NODEEL(NEL,1)))**2+(Z(NODEEL(NEL,2))-Z
     1(NODEEL(NEL,1)))**2))/2.0
      GO TO 10
    6 B=(Z(NODEEL(NEL,2))-Z(NODEEL(NEL,1)))/2.0
      GO TO 10
    8 B=(Y(NODEEL(NEL,2))-Y(NODEEL(NEL,1)))/2.0
C     ELEMENT STIFFNESS FORMULATION USING GAUSS-LEGENDRE INTEGRATION
   10 IF(A.NE.0.0.AND.B.NE.0.0) GO TO 12
      WRITE (6,200) A,B,NEL
  200 FORMAT(///' *********     WARNING     *************',/
     1,'  A=',E13.4,'  AND B=',E13.4,'  FOR ELEMENT NO.',I4///)
   12 DO 15 J=1,12
      DO 15 I=1,12
      STF1(I,J)=0.0
      STF2(I,J)=0.0
   15 CONTINUE
      IF(ISTIF.EQ.0) GO TO 18
      WRITE (6,653)
  653 FORMAT(///' **** OUTPUT FROM ELSTF ****')
      WRITE (6,655) NEL,A,B
  655 FORMAT(/' FOR ELEMENT',I3,' A=',F6.1,' AND B=',F6.1)
   18 DO 40 N=1,3
      DO 40 M=1,3
      XX=G(M)*A
      YY=G(N)*B
C     L IS THE GAUSS-LEGENDRE STATION NUMBER
      L=(N-1)*3+M
      CALL BCALC(A,B,XX,YY,L)
      DO 20 J=1,12
      DO 20 I=1,3
      CC1(I,J)=0.0
      CC2(I,J)=0.0
   20 CONTINUE
   28 DO 30 J=1,12
      DO 30 I=1,3
      DO 30 K=1,3
      CC1(I,J)=CC1(I,J)+D(I,K)*B1(K,J,L)
      CC2(I,J)=CC2(I,J)+D(I,K)*B2(K,J,L)
   30 CONTINUE
      DO 35 J=1,12
      DO 35 I=1,12
      DO 35 K=1,3
      STF1(I,J)=STF1(I,J)+ELTHN(NEL)*A*B*B1(K,I,L)*CC1(K,J)*W(N)*W(M)
      STF2(I,J)=STF2(I,J)+ELTHN(NEL)*A*B*B2(K,I,L)*CC2(K,J)*W(N)*W(M)
   35 CONTINUE
   40 CONTINUE
C     ELEMENT STIFFNESS MATRICES FOR TYPE 1 AND TYPE 2 ELEMENTS HAVE
C     BEEN CALCULATED
  100 IF(INELTY(NEL).EQ.1) GO TO 110
      DO 105 J=1,12
      DO 105 I=1,12
```

```
      ESTF(I,J)=STF2(I,J)
  105 CONTINUE
      GO TO 150
  110 DO 115 J=1,12
      DO 115 I=1,12
      ESTF(I,J)=STF1(I,J)
  115 CONTINUE
  150 IF(IPRINT.EQ.0.OR.ISTIF.EQ.0) RETURN
      WRITE (6,155)
  155 FORMAT(//' THE STIFFNESS MATRIX OF A TYPE 1 ELEMENT IS AS BELOW'/)
      DO 165 I=1,12
      WRITE (6,180) (STF1(I,J),J=1,12)
  165 CONTINUE
      WRITE (6,170)
  170 FORMAT(//' THE STIFFNESS MATRIX OF A TYPE 2 ELEMENT IS AS BELOW'/)
      DO 175 I=1,12
      WRITE (6,180) (STF2(I,J),J=1,12)
  175 CONTINUE
  180 FORMAT(12E11.5)
      RETURN
      END
      SUBROUTINE STFADD(NS4,NS2,NS3,TS,AL,AR,STIF)
C*********************************************************************
C      THIS SUBROUTINE ADDS TO THE STIFFNESS MATRIX IN CORE
C     TS(NBAND,NBAND*2),THE STIFFNESS OF SINGLE REINFORCING AND
C     PRESTRESS BARS,AND BOND SLIP LINKAGES IN INITIAL COMPUTATION OF
C     THE TOTAL STIFFNESS MATRIX
C*********************************************************************
C
       COMMON/BLOCK1/NEQNS,NBAND,NBLK,NSHINC,ICOUNT
       COMMON/BLOCK4/NR,NREO,NODES(190,2),INRDN(190),INRTY(190),
      1NALTER(190),NAREA(190),CAREA(190),NSTF(190),TSGPRE(190),TEPRE(190)
       COMMON/BLOCK5/NNODES,ICNODE(220),X(220),Y(220),Z(220)
       COMMON/BLOCK6/CONMOD,RNOMOD,PREMOD,SLPMOD,SHSMOD,FC,FT,
      1FAGG,FUR,FUP,FUNS,DP,ESLIP,ECULT,ERULT,EPULT,ENSULT,P1,P2,P3,P4,
      2PU,ENREO,ENPRE,CONDEV,DEVCON,DEVREO,PREDEV,SLPDEV,IDEV,AVCSP,RELAX
       COMMON/BLCK13/NYNESN(185,4),NDOWEL(185),NCRACK(185),NYAGG(185),
      1NYREO(190),NYCREO(190)
       INTEGER*4 NALTER
       DIMENSION TS(NS1,NS2),AL(NS3),AR(NS3),J(2),K(2)
       II=NODES(NR,1)
       JJ=NODES(NR,2)
       CALL LOCATE(II,JJ,NROW,NCOL)
       IF(INRTY(NR).GT.0) GO TO 100
C      ADDITION IN OF STIFFNESSES OF SINGLE REINFORCING AND PRESTRESSED
C      BARS
       IF(INRDN(NR).GT.1) GO TO 50
C      CONSIDERING BARS IN HORIZONTAL OR VERTICAL ELEMENTS
       IF(INRDN(NR)) 4,6,8
    4  ALGTH=X(JJ)-X(II)
       GO TO 10
    6  ALGTH=Y(JJ)-Y(II)
       GO TO 10
    8  ALGTH=Z(JJ)-Z(II)
   10  STF=NAREA(NR)*STIF/ALGTH
       I1=NROW+INRDN(NR)+1
       IF(ICNODE(II).EQ.0.AND.INRDN(NR).EQ.1) I1=I1-1
       J1=NCOL+INRDN(NR)+1
       IF(ICNODE(JJ).EQ.0.AND.INRDN(NR).EQ.1) J1=J1-1
       TS(1,I1)=TS(1,I1)+STF
       TS(1,J1)=TS(1,J1)+STF
       IF(NROW.GT.NCOL) GO TO 15
       TS(J1-I1+1,I1)=TS(J1-I1+1,I1)-STF
       RETURN
   15  TS(I1-J1+1,J1)=TS(I1-J1+1,J1)-STF
       RETURN
C      CONSIDERING VERTICAL BARS IN OBLIQUE PLANES
   50  ALGTH=SQRT((Y(JJ)-Y(II))**2+(Z(JJ)-Z(II))**2)
       STF=NAREA(NR)*STIF/ALGTH
       C=(Y(JJ)-Y(II))/ALGTH
       S=(Z(JJ)-Z(II))/ALGTH
       J(1)=NROW+1
       K(1)=NCOL+1
       J(2)=NCOL+1
       K(2)=NROW+1
       DO 80 I=1,2
       I1=NODES(NR,I)
       IF(ICNODE(I1).EQ.0) GO TO 60
       TS(1,J(I))=TS(1,J(I))+STF*C**2
       TS(1,J(I)+1)=TS(1,J(I)+1)+STF*S**2
       TS(2,J(I))=TS(2,J(I))+STF*C*S
       IF(J(I).LT.K(I)) GO TO 80
       TS(J(I)-K(I)+1,K(I))=TS(J(I)-K(I)+1,K(I))-STF*C
       TS(J(I)-K(I)+2,K(I))=TS(J(I)-K(I)+2,K(I))-STF*S
       GO TO 80
   60  TS (1,J(I))=TS(1,J(I))+STF
       IF(J(I).LT.K(I)) GO TO 80
       TS(J(I)-K(I)+1,K(I))=TS(J(I)-K(I)+1,K(I))-C*STF
       TS(J(I)-K(I),K(I)+1)=TS(J(I)-K(I),K(I)+1)-S*STF
   80  CONTINUE
       RETURN
C      ADDING IN BOND SLIP STIFFNESSES
  100  STF=STIF*CAREA(NR)
       TS(1,NROW)=TS(1,NROW)+STF
       TS(1,NCOL)=TS(1,NCOL)+STF
       IF(II.LT.JJ) GO TO 110
       TS(NROW-NCOL+1,NCOL)=TS(NROW-NCOL+1,NCOL)-STF
       RETURN
  110  TS(NCOL-NROW+1,NROW)=TS(NCOL-NROW+1,NROW)-STF
       RETURN
       END
       SUBROUTINE BLKADD(II,JJ,NS1,NS2,NS3,TS,AL,AR)
C*********************************************************************
C      THIS SUBROUTINE ADDS THE REINFORCED CONCRETE ELEMENT STIFFNESS
C      SUB-BLOCKS BS(3,3) INTO THE TOTAL STIFFNESS MATRIX IN CORE.
C*********************************************************************
C
        COMMON/BLOCK2/NEL,NELT,NELCHK,NELD,INDCEL(185),INELSZ(185),
       1INDOWL(185),WIDTHC(185),CANGLE(185),CALTER(185),
       2INELTY(215),ELTHN(215),NODEEL(215,4),NDREF(215),DPHMOD(215)
        COMMON/BLOCK5/NNODES,ICNODE(220),X(220),Y(220),Z(220)
        COMMON/BLCK10/ESTF(12,12),BS(3,3),B1(3,12,9),B2(3,12,9),D(3,3),
       1DCONC(185,3,3),DCRACK(185),DAGG(185)
        COMMON/BLCK13/NYNESN(185,4),NDOWEL(185),NCRACK(185),NYAGG(185),
       1NYREO(190),NYCREO(190)
        COMMON/BLCK14/IDEFLN,ICON3,ICONPR,INZSN,INEO,ILOAD,ITLOAD,ISTIF,
       1NAXIT,NYTCON,NYTREO,NYTPRE
        DIMENSION NR(5),NC(5),BS1(5,5),BS2(5,5),BSS(5,5)
        DIMENSION TS(NS1,NS2),AL(NS3),AR(NS3)
        REAL*4 LT(5,5)
```

```
      INTEGER*4 CALTER
C     CALCULATION OF LOCATION IN CORE OF FIRST ELEMENT IN BLOCK
      CALL LOCATE(II,JJ,NROW,NCOL)
      NR(1)=NROW
      NC(1)=NCOL
      I1=3
      J1=3
C     THE STIFFNESS BLOCK BS(3,3) HAS TO BE CONVERTED FROM A LOCAL
C     STIFFNESS MATRIX TO A GLOBAL STIFFNESS MATRIX IF THE ELEMENT
C     NEL IS INCLINED AND EITHER II OR JJ NODES ARE CORNER NODES
      IF(INDCEL(NEL).GE.0) GO TO 100
      IF(ICNODE(II).NE.1.AND.ICNODE(JJ).NE.1) GO TO 100
      DO 10 J=1,5
      DO 10 I=1,5
      LT(I,J)=0.0
      BS1(I,J)=0.0
      BS2(I,J)=0.0
      BSS(I,J)=0.0
   10 CONTINUE
C     DEFINITION OF TRANSFORMATION MATRIX LT(5,5)
      B=(SQRT((Y(NODEEL(NEL,2))-Y(NODEEL(NEL,1)))**2+(Z(NODEEL(NEL,2))-Z
     1(NODEEL(NEL,1)))**2))
      C=(Y(NODEEL(NEL,2))-Y(NODEEL(NEL,1)))/B
      S=(Z(NODEEL(NEL,2))-Z(NODEEL(NEL,1)))/B
      LT(1,1)=1.0
      LT(2,2)=C
      LT(3,2)=-S
      LT(2,3)=S
      LT(3,3)=C
      LT(4,4)=C
      LT(5,4)=-S
      LT(4,5)=S
      LT(5,5)=C
C     TRANSFORMATION OF BS(3,3) TO GLOBAL FORM
      IF(ICNODE(JJ).EQ.1) GO TO 15
      DO 12 J=1,3
      DO 12 I=1,3
      BS1(I,J)=BS(I,J)
   12 CONTINUE
      NC(2)=NCOL+1
      NC(3)=NCOL+2
      IF(ICNODE(JJ).EQ.2) NC(3)=NCOL+3
      GO TO 22
   15 DO 17 I=1,3
      BS1(I,1)=BS(I,1)
      BS1(I,2)=BS(I,2)
      BS1(I,5)=BS(I,3)
   17 CONTINUE
      J1=J1+2
      DO 18 J=1,5
      NC(J)=NCOL+J-1
   18 CONTINUE
      DO 19 J=1,5
      DO 19 I=1,5
      DO 19 K=1,5
      BS2(I,J)=BS2(I,J)+BS1(I,K)*LT(K,J)
   19 CONTINUE
      DO 20 J=1,5
      DO 20 I=1,5
      BS1(I,J)=BS2(I,J)
      BS2(I,J)=0.0
   20 CONTINUE
   22 IF(ICNODE(II).NE.1) GO TO 40
      DO 25 J=1,5
      BS2(1,J)=BS1(1,J)
      BS2(2,J)=BS1(2,J)
      BS2(5,J)=BS1(3,J)
   25 CONTINUE
      I1=I1+2
      DO 30 I=1,5
      NR(I)=NROW+I-1
   30 CONTINUE
      DO 35 J=1,5
      DO 35 I=1,5
      DO 35 K=1,5
      BSS(I,J)=BSS(I,J)+LT(K,I)*BS2(K,J)
   35 CONTINUE
      GO TO 140
   40 DO 45 J=1,5
      DO 45 I=1,5
      BSS(I,J)=BS1(I,J)
   45 CONTINUE
      NR(2)=NROW+1
      NR(3)=NROW+2
      IF(ICNODE(II).EQ.2) NR(3)=NROW+3
      GO TO 140
C     CALCULATION OF ROW AND COLUMN LOCATIONS OF UNALTERED BLOCK BS(3,3)
  100 DO 105 J=1,3
      DO 105 I=1,3
      BSS(I,J)=BS(I,J)
  105 CONTINUE
      IF(ICNODE(II).EQ.1) GO TO 110
      NR(2)=NROW+1
      NR(3)=NROW+2
      IF(ICNODE(II).EQ.2) NR(3)=NROW+3
      GO TO 120
  110 IF(INDCEL(NEL).EQ.0) GO TO 115
      NR(2)=NROW+1
      NR(3)=NROW+4
      GO TO 120
  115 NR(2)=NROW+2
      NR(3)=NROW+3
  120 IF(ICNODE(JJ).EQ.1) GO TO 130
      NC(2)=NCOL+1
      NC(3)=NCOL+2
      IF(ICNODE(JJ).EQ.2) NC(3)=NCOL+3
      GO TO 140
  130 IF(INDCEL(NEL).EQ.0) GO TO 135
      NC(2)=NCOL+1
      NC(3)=NCOL+4
      GO TO 140
  135 NC(2)=NCOL+2
      NC(3)=NCOL+3
C     ADDITION OF BS(5,5) MATRIX TO TOTAL STIFFNESS MATRIX IN CORE
  140 DO 160 J=1,J1
      DO 160 I=1,I1
      NNR=NR(I)-NC(J)+1
      IF(NNR.LT.1) GO TO 160
      TS(NNR,NC(J))=TS(NNR,NC(J))+BSS(I,J)
  160 CONTINUE
```

```
      IF(NEL.GT.1) RETURN
      IF(ISTIF.EQ.0) RETURN
      WRITE (6,658)
  658 FORMAT(///' **** OUTPUT FROM BLKADD ****')
      WRITE (6,660) NEL,II,JJ
  660 FORMAT(/' CONSIDERING ADDITION OF BS(3,3) OF ELEMENT NO.',I3,' ,TH
     1E ROW NODE NO. OF BLOCK=',I3,' AND THE COLUMN NODE NO.=',I3)
      WRITE (6,665) I1,J1
  665 FORMAT(/' IN THE GLOBAL FORM BSS(5,5),THE NUMBER OF ROWS =',I3,' A
     1ND THE NUMBER OF COLUMNS =',I3)
      WRITE (6,670)
  670 FORMAT(/' THE GLOBAL FORM OF MATRIX,BSS(5,5),IS AS BELOW')
      DO 680 I=1,I1
      WRITE (6,675) (BSS(I,J),J=1,J1)
  675 FORMAT(5E15.5)
  680 CONTINUE
      RETURN
      END
      SUBROUTINE LOCATE(II,JJ,NROW,NCOL)
C*********************************************************************
C      THIS SUBROUTINE CALCULATES THE POSITION OF THE FIRST ELEMENT OF A
C     COMPONENT (3,3) STIFFNESS BLOCK IN THE TOTAL STIFFNESS MATRIX
C     TS(NBAND,NBAND*2) IN CORE AT THAT TIME.ACCOUNT IS TAKEN OF THE
C     FACT THAT (NBLK-1) BLOCKS HAVE BEEN ACCUMULATED AND WRITTEN OUT
C     ON DISC ALREADY
C*********************************************************************
C
      COMMON/BLOCK1/NEQNS,NBAND,NBLK,NUMINC,ICOUNT
      COMMON/BLOCK5/NNODES,ICNODE(220),X(220),Y(220),Z(220)
      DIMENSION N(2),NC(2)
      NC(1)=1
      NC(2)=1
C     CALCULATION OF NUMBER OF EQUATIONS,KK, THAT HAVE ALREADY BEEN
C     WRITTEN OUT ON DISC
      KK=(NBLK-1)*NBAND
      N(1)=II
      N(2)=JJ
C     CALCULATION OF RESPECTIVE NUMBER OF EQUATIONS ASSOCIATED WITH
C     NODES II AND JJ
      DO 20 I=1,2
      IF(N(I).LE.1) GO TO 20
      J1=N(I)-1
      DO 10 J=1,J1
      IF(ICNODE(J)) 4,5,6
    4 NC(I)=NC(I)+1
      GO TO 10
    5 NC(I)=NC(I)+3
      GO TO 10
    6 ICC=ICNODE(J)-2
      IF(ICC) 7,8,9
    7 NC(I)=NC(I)+5
      GO TO 10
    8 NC(I)=NC(I)+4
      GO TO 10
    9 NC(I)=NC(I)+2
   10 CONTINUE
   20 CONTINUE
C     MODIFICATION OF FIRST ROW NUMBER OF NODES II AND JJ CONSIDERING
C     ALL ROWS PRECEDING NODE KK HAVE BEEN WRITTEN OUT
      NROW=NC(1)-KK
      NCOL=NC(2)-KK
      RETURN
      END
      SUBROUTINE HORIZ
C*********************************************************************
C      THIS SUBROUTINE ADJUSTS THE SIGNS OF BLOCK BS(3,3) IF ITS
C     ELEMENT IS HORIZONTAL. THIS IS TO CONVERT THE LOCAL STIFFNESS
C     BLOCK INTO A GLOBAL STIFFNESS BLOCK
C*********************************************************************
C
      COMMON/BLCK10/ESTF(12,12),BS(3,3),B1(3,12,9),B2(3,12,9),D(3,3),
     1DCONC(185,3,3),DCRACK(185),DAGG(185)
      DO 5 I=1,2
      BS(I,3)=-BS(I,3)
      BS(3,I)=-BS(3,I)
    5 CONTINUE
      RETURN
      END
      SUBROUTINE BCALC(A,B,X,Y,L)
C*********************************************************************
C      THIS SUBROUTINE CALCULATES THE 'B' MATRICES FOR ELEMENT TYPES 1&2
C     AT INTEGRATION POINT NUMBER L
C*********************************************************************
C
      COMMON/BLCK10/ESTF(12,12),BS(3,3),B1(3,12,9),B2(3,12,9),D(3,3),
     1DCONC(185,3,3),DCRACK(185),DAGG(185)
      DO 5 J=1,12
      DO 5 I=1,3
      B1(I,J,L)=0.0
      B2(I,J,L)=0.0
    5 CONTINUE
      B1(1,1,L)=-1.0/(8.0*A)-Y**2/(8.0*A*B**2)+Y/(4.0*A*B)
      B1(3,1,L)=Y/(4.0*B**2)-.25/B-X*Y/(4.0*A*B**2)+X/(4.0*A*B)
      B1(2,2,L)=-.375/B+X/(4.0*A*B)+X**2/(8.0*A**2*B)
      B1(3,2,L)=Y/(4.0*A*B)-.25/A-X/(4.0*A**2)+X*Y/(4.0*A**2*B)
      B1(2,3,L)=-A/(4.0*B)+X**2/(4.0*A*B)
      B1(3,3,L)=-X/(2.0*A)+X*Y/(2.0*A*B)
      B1(1,4,L)=-3.0/(8.0*A)+Y**2/(8.0*A*B**2)-Y/(4.0*A*B)
      B1(3,4,L)=-Y/(4.0*B**2)+.25/B+X*Y/(4.0*A*B**2)-X/(4.0*A*B)
      B1(2,5,L)=.125/B-X/(4.0*A*B)+X**2/(8.0*A**2*B)
      B1(3,5,L)=-Y/(4.0*A*B)-.25/A+X/(4.0*A**2)+X*Y/(4.0*A**2*B)
      B1(1,6,L)=-B/(4.0*A)+Y**2/(4.0*A*B)
      B1(3,6,L)=-Y/(2.0*B)+X*Y/(2.0*A*B)
      B1(1,7,L)=1.0/(8.0*A)+Y**2/(8.0*A*B**2)+Y/(4.0*A*B)
      B1(3,7,L)=Y/(4.0*B**2)+.25/B+X*Y/(4.0*A*B**2)+X/(4.0*A*B)
      B1(2,8,L)=.375/B+X/(4.0*A*B)-X**2/(8.0*A**2*B)
      B1(3,8,L)=Y/(4.0*A*B)+.25/A-X/(4.0*A**2)-X*Y/(4.0*A**2*B)
      B1(2,9,L)=-A/(4.0*B)+X**2/(4.0*A*B)
      B1(3,9,L)=X/(2.0*A)+X*Y/(2.0*A*B)
      B1(1,10,L)=.375/A-Y**2/(8.0*A*B**2)-Y/(4.0*A*B)
      B1(3,10,L)=-Y/(4.0*B**2)-.25/B-X*Y/(4.0*A*B**2)-X/(4.0*A*B)
      B1(2,11,L)=-.125/B-X/(4.0*A*B)-X**2/(8.0*A**2*B)
      B1(3,11,L)=-Y/(4.0*A*B)+.25/A+X/(4.0*A**2)-X*Y/(4.0*A**2*B)
      B1(1,12,L)=-B/(4.0*A)+Y**2/(4.0*A*B)
      B1(3,12,L)=Y/(2.0*B)+X*Y/(2.0*A*B)
C***** FORMATION OF B2(3,12,NSTATION)
      B2(1,1,L)=-.375/A+Y**2/(8.0*A*B**2)+Y/(4.0*A*B)
      B2(3,1,L)=-Y/(   2)-.25/B+X*Y/(4.0*A*B**2)+X/(4.0*A*B)
      B2(2,2,L)   (4.0*A*B)-X**2/(8.0*A**2*B)
      B2(3,2,   /(   )-.25/A+X/(4.0*A**2)-X*Y/(4.0*A**2*B)
```

```
      B2(1,3,L)=B/(4.0*A)-Y**2/(4.0*A*B)
      B2(3,3,L)=Y/(2.0*B)-X*Y/(2.0*A*B)
      B2(1,4,L)=-.125/A-Y**2/(8.0*A*B**2)-Y/(4.0*A*B)
      B2(3,4,L)=Y/(4.0*B**2)+.25/B-X*Y/(4.0*A*B**2)-X/(4.0*A*B)
      B2(2,5,L)=.375/B-X/(4.0*A*B)-X**2/(8.0*A**2*B)
      B2(3,5,L)=-Y/(4.0*A*B)-.25/A-X/(4.0*A**2)-X*Y/(4.0*A**2*B)
      B2(2,6,L)=A/(4.0*B)-X**2/(4.0*A*B)
      B2(3,6,L)=-X/(2.0*A)-X*Y/(2.0*A*B)
      B2(1,7,L)=.375/A-Y**2/(8.0*A*B**2)+Y/(4.0*A*B)
      B2(3,7,L)=-Y/(4.0*B**2)+.25/B-X*Y/(4.0*A*B**2)+X/(4.0*A*B)
      B2(2,8,L)=.125/B+X/(4.0*A*B)+X**2/(8.0*A**2*B)
      B2(3,8,L)=Y/(4.0*A*B)+.25/A+X/(4.0*A**2)+X*Y/(4.0*A**2*B)
      B2(1,9,L)=B/(4.0*A)-Y**2/(4.0*A*B)
      B2(3,9,L)=-Y/(2.0*B)-X*Y/(2.0*A*B)
      B2(1,10,L)=.125/A+Y**2/(8.0*A*B**2)-Y/(4.0*A*B)
      B2(3,10,L)=Y/(4.0*B**2)-.25/B+X*Y/(4.0*A*B**2)-X/(4.0*A*B)
      B2(2,11,L)=-.375/B-X/(4.0*A*B)+X**2/(8.0*A**2*B)
      B2(3,11,L)=-Y/(4.0*A*B)+.25/A-X/(4.0*A**2)+X*Y/(4.0*A**2*B)
      B2(2,12,L)=A/(4.0*B)-X**2/(4.0*A*B)
      B2(3,12,L)=X/(2.0*A)-X*Y/(2.0*A*B)
      RETURN
      END
      SUBROUTINE NODLOC(NEQ,NN)
C*********************************************************************
C  THIS SUBROUTINE CALCULATES THE NODE NUMBER NN THAT CONTAINS THE
C NEQ TH. EQUATION
C*********************************************************************
C
      COMMON/BLOCK5/NNODES,ICNODE(220),X(220),Y(220),Z(220)
      I=0
      IF(NEQ.GT.0) GO TO 2
      NN=0
      RETURN
    2 I=I+1
      IF(ICNODE(I)) 4,5,6
    4 NEQ=NEQ-1
      GO TO 10
    5 NEQ=NEQ-3
      GO TO 10
    6 ICC=ICNODE(I)-2
      IF(ICC) 7,8,9
    7 NEQ=NEQ-5
      GO TO 10
    8 NEQ=NEQ-4
      GO TO 10
    9 NEQ=NEQ-2
   10 IF(NEQ.GE.1) GO TO 2
      NN=I
      RETURN
      END
      SUBROUTINE READIN
C*********************************************************************
C       THIS SUBROUTINE READS IN ALL INPUT DATA AND PROVIDES AN ECHO
C      CHECK
C*********************************************************************
C
      COMMON/BLOCK1/NEQNS,NBAND,MBLK,NNINC,ICOUNT
      COMMON/BLOCK2/NEL,NELT,NELCHK,NELD,INDCEL(185),INELSX(185),
     1INDOWL(185),WIDTHC(185),CANGLE(185),CALTER(185),
     2INELTY(215),ELTHN(215),NODEEL(215,4),NDREF(215),DPNNOD(215)
      COMMON/BLOCK3/NELSH,INMESH(185),INSIMS(185),NDIRNS,PERSTL(185,4),
     1ANGLE(185,4),SHSTF(185,3,3),SHESH(185,1)
      COMMON/BLOCK4/NR,NRBO,NODES(190,2),INRDN(190),INRTY(190),
     1RALTER(190),RAREA(190),CAREA(190),RSTP(190),TSGPRE(190),TEPRE(190)
      COMMON/BLOCK5/NNODES,ICNODE(220),X(220),Y(220),Z(220)
      COMMON/BLOCK6/CONMOD,RBOMOD,PREMOD,SLPMOD,SHSMOD,FC,FT,
     1FAGG,FUR,FUP,FUNS,DF,ESLIP,ECULT,ERULT,EPULT,EMSULT,P1,P2,P3,P4,
     2PU,ERRPO,EEPRE,CONDEV,DEVCON,DEVREO,PREDEV,SLPDEV,IDEV,AVCSP,RELAX
      COMMON/BLOCK7/NLDTY,NINCRT(6),NLOADS(6),NODER(40,6),
     1KODER(40,6),VALUER(40,6)
      COMMON/BLOCK8/NDISPL,NODED(30),KODED(30),VALUED(30),D1(750)
      COMMON/BLOCK9/LPOINT(20,2),FDUB1,FDUB2,NUMREC,
     1NUMBLK,ISOLVE,LEN
      COMMON/BLCK12/SIGCON(185,3),ECON(185,3),SIGMS(185,4),EMS(185,4),
     1TSGCON(185,3),TECON(185,3),TSGCC(185),TSGCT(185),TSGAGG(185),
     2TECC(185),TECT(185),TSGMS(185,4),TEMS(185,4),SIGREO(190),
     3ERBO(190),TSGREO(190),TERRO(190)
      COMMON/BLCK13/NYMESH(185,4),NDOWEL(185),NCRACK(185),NYAGG(185),
     1NYREO(190),NYCREO(190)
      COMMON/BLCK14/IDEFLN,ICON3,ICONPR,IMESH,IREO,ILOAD,ITLOAD,ISTIF,
     1MAXIT,NUTCON,NUTREO,NUTPRE
      COMMON/BLCK15/NELTOP,NLTP(40),NLBN(40),SIGXT1,SIGXT2,SIGXB1,
     1SIGXB2,SIGXS1,SIGXS2,EXT1,EXT2,EXB1,EXB2,EXS1,EXS2
      COMMON/BLCK16/PNELT,PNEL(10),PNELSH,PSH(10),PNREO,PNR(10),
     1PDTOT,PDT(10)
      DIMENSION NS(185)
      INTEGER*4 FDUB1,FDUB2,CALTER,RALTER,PNELT,PNEL,PNELSH,PSH,PNREO,
     1PNR,PDTOT,PDT
      INTEGER*2 LEN
      READ (5,90) IWRITE
C***** READ STATEMENTS
      READ (5,90) NEQNS,NBAND,LEN
C     FINITE ELEMENT DATA
      READ (5,90) NELT,NELCHK
      DO 5 I=1,NELT
      READ (5,70) INDCEL(I),INELSX(I),INELTY(I),ELTHN(I),
     1(NODEEL(I,J),J=1,4),INDOWL(I),INMESH(I),INSIMS(I),WIDTHC(I)
    5 CONTINUE
C     CONCRETE SHRINKAGE STRESSES DATA
      READ (5,70) NELTOP,SIGXT1,SIGXT2,SIGXB1,SIGXB2,SIGXS1,SIGXS2,
     1EXT1,EXT2,EXB1,EXB2,EXS1,EXS2
      DO 8 I=1,4
      I1=(I-1)*10+1
      IF(I1.GT.NELTOP) GO TO 8
      I2=I*10
      IF(I2.GT.NELTOP) I2=NELTOP
      READ (5,70) (NLTP(J),J=I1,I2)
      READ (5,70) (NLBN(J),J=I1,I2)
    8 CONTINUE
C     ELEMENT STEEL MESH DATA
      READ (5,90) NELSH
      IF(NELSH.EQ.0) GO TO 12
      READ (5,90) NDIRNS
      DO 10 I=1,NELSH
      READ (5,90) NS(I)
      I1=NS(I)
      READ (5,80)  (PERSTL(I1,J), ANGLE(I1,J),  J = 1,
     1NDIRNS)
   10 CONTINUE
C     DIAPHRAGM INPUT DATA
```

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   12 READ (5,80) NELD
      IF(NELD.EQ.0) GO TO 14
      I1=NELT+1
      I2=NELT+NELD
      DO 13 I=I1,I2
      READ (5,80) INELTY(I),ELTHN(I),(NODEEL(I,J),J=1,4),NDREF(I),
     1DPHNOD(I)
   13 CONTINUE
C     REINFORCEMENT DATA
   14 READ (5,90) NREO
      IF(NREO.EQ.0) GO TO 26
      DO 15 I=1,NREO
      READ (5,90) NODES(I,1),NODES(I,2),INRTY(I)
   15 CONTINUE
      DO 25 I=1,NREO
      IF(INRTY(I)) 20,22,24
   20 READ (5,90) INRDN(I),RAREA(I)
      GO TO 25
   22 READ (5,90) INRDN(I),RAREA(I),TSGPRE(I),TEPRE(I)
      GO TO 25
   24 READ (5,90) CAREA(I)
   25 CONTINUE
C     NODAL DATA
   26 READ (5,90) NNODES
      DO 30 I=1,NNODES
      READ (5,90) ICNODE(I),X(I),Y(I),Z(I)
   30 CONTINUE
C     STRENGTH PARAMETERS
      READ (5,90) CONMOD,REOMOD,PREMOD,SLPMOD,SHSMOD
      READ (5,90) FC,FT,FAGG,FUR,FUP,FUHS
      READ (5,80) EERREO,EEPRE,ECULT,ERULT,EPULT,EHSULT,ESLIP,DF
      READ (5,90) P1,P2,P3,PU
      READ (5,80) CONDEV,DEVCON,DEVREO,PREDEV,SLPDEV,IDEV,AVCSP,RELAX
      READ (5,80) MAXIT,NNTCON,NNTREO,NNTPRE
C     LOADING DATA
      READ (5,90) NLDTY
      DO 40 I=1,NLDTY
      READ (5,90) NINCRT(I),NLOADS(I)
      NL=NLOADS(I)
      DO 35 J=1,NL
      READ (5,90) NODER(J,I),KODER(J,I),VALUER(J,I)
   35 CONTINUE
   40 CONTINUE
C     IMPOSED BOUNDARY CONDITIONS
      READ (5,90) NDISPL
      DO 45 I=1,NDISPL
      READ (5,90) NODED(I),KODED(I),VALUED(I)
   45 CONTINUE
C     SCREEN PRINTOUT CNTROL DATA
      READ (5,70) PNELT
      READ (5,70) (PNEL(I),I=1,PNELT)
      READ (5,70) PNELSH
      READ (5,70) (PSH(I),I=1,PNELSH)
      READ (5,70) PNREO
      READ (5,70) (PNR(I),I=1,PNREO)
      READ (5,70) PDTOT
      READ (5,70) (PDT(I),I=1,PDTOT)
C     PRINTOUT CONTROL INFORMATION
      READ (5,80) IDEFLN,ICON3,ICONPR,INESH,INREO,ILOAD,ITLOAD,ISTIF
C***** WRITE STATEMENTS
      IF(INRITE.EQ.0) RETURN
      WRITE (6,100) NEQNS,NBAND,LEN,NELT,NELCHK
      WRITE (6,105)
      DO 112 I=1,NELT
      WRITE (6,110) I,INDCEL(I),INELSZ(I),INELTY(I),ELTHN(I),
     1(NODEEL(I,J),J=1,4),INDOWL(I),INNESH(I),INSNHS(I),WIDTHC(I)
  112 CONTINUE
      WRITE (6,115) NELTOP,SIGXT1,SIGXT2,SIGXB1,SIGXB2,SIGXS1,
     1SIGXS2,EXT1,EXT2,EXB1,EXB2,EXS1,EXS2
      DO 118 I=1,4
      I1=(I-1)*10+1
      IF(I1.GT.NELTOP) GO TO 118
      I2=I*10
      IF(I2.GT.NELTOP) I2=NELTOP
      WRITE (6,116) I1,I2,(NLTP(J),J=I1,I2)
      WRITE (6,117) I1,I2,(NLBH(J),J=I1,I2)
  118 CONTINUE
      WRITE (6,120) NELSH
      IF(NELSH.EQ.0) GO TO 135
      WRITE (6,125)
      DO 133 I=1,NELSH
      I1=NS(I)
      DO 133 J=1,NDIRNS
      WRITE (6,130) NS(I),NDIRNS,J,PERSTL(I1,J),ANGLE(I1,J)
  133 CONTINUE
  135 WRITE (6,140) NELD
      IF(NELD.EQ.0) GO TO 151
      WRITE (6,147)
      I1=NELT+1
      I2=NELT+NELD
      DO 149 I=I1,I2
      WRITE (6,148) I,INELTY(I),ELTHN(I),(NODEEL(I,J),J=1,4),NDREF(I),
     1DPHNOD(I)
  149 CONTINUE
  151 WRITE (6,150) NREO
      WRITE (6,152)
      DO 170 I=1,NREO
      IF(INRTY(I).GT.0) GO TO 160
      IF(INRTY(I).EQ.0) GO TO 156
      WRITE (6,155) I,NODES(I,1),NODES(I,2),INRTY(I),INRDN(I),RAREA(I)
      GO TO 170
  156 WRITE (6,155) I,NODES(I,1),NODES(I,2),INRTY(I),INRDN(I),RAREA(I),
     1TSGPRE(I),TEPRE(I)
      GO TO 170
  160 WRITE (6,158) I,NODES(I,1),NODES(I,2),INRTY(I),CAREA(I)
  170 CONTINUE
      WRITE (6,175) NNODES
      WRITE (6,180)
      DO 190 I=1,NNODES
      WRITE (6,185) I,ICNODE(I),X(I),Y(I),Z(I)
  190 CONTINUE
      WRITE (6,195)
      WRITE (6,200) CONMOD,REOMOD,PREMOD,SLPMOD,SHSMOD
      WRITE (6,205) FC,FT,FAGG,FUR,FUP,FUHS
      WRITE (6,210) EERREO,EEPRE,ECULT,ERULT,EPULT,EHSULT,ESLIP,DF
      WRITE (6,215) P1,P2,P3,PU
      WRITE (6,218) CONDEV,DEVCON,DEVREO,PREDEV,SLPDEV,IDEV,AVCSP,RELAX
      WRITE (6,219) MAXIT,NNTCON,NNTREO,NNTPRE
      WRITE (6,220) NLDTY
      DO 250 I=1,NLDTY
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      WRITE (6,225)
      WRITE (6,230) I,NINCRT(I),NLOADS(I)
      NL=NLOADS(I)
      WRITE (6,235)
      DO 245 J=1,NL
      WRITE (6,240) NODER(J,I),KODER(J,I),VALUER(J,I)
  245 CONTINUE
  250 CONTINUE
      WRITE (6,255) NDISPL
      WRITE (6,260)
      DO 270 I=1,NDISPL
      WRITE (6,265) NODED(I),KODED(I),VALUED(I)
  270 CONTINUE
      WRITE (6,271)
      WRITE (6,272) PNELT, (PNEL(I),I=1,PNELT)
      WRITE (6,273) PNELSH, (PSH(I),I=1,PNELSH)
      WRITE (6,274) PNRNO, (PNR(I),I=1,PNRNO)
      WRITE (6,275) PDTOT, (PDT(I),I=1,PDTOT)
      WRITE (6,278)
      WRITE (6,280) IDEPLN,ICON3,ICONPR,IMESH,IREO,ILOAD,ITLOAD,ISTIF
      WRITE (6,400)
C***** FORMAT STATEMENTS
   70 FORMAT(14G9.0)
   80 FORMAT(11G12.0)
   90 FORMAT(6G20.0)
  100 FORMAT(//' ***************************************************
     1*************************************************************
     2*************',/,' *******************************************
     3*************************************************************
     4**********************',/,'
     5                * ECHO CHECK OF DATA INPUT *',/,'
     6                      *****************************',///,' T
     7OTAL NUMBER OF EQUATIONS =',I6/,' BAND WIDTH =',I5/,
     8' NUMBER OF BYTES IN EACH RECORD =',I6/
     9,' NUMBER OF FINITE ELEMENTS =',I5/,
     1' NUMBER OF FINITE ELEMENTS CHECKED FOR CRACKING =',I5)
  105 FORMAT(///' ELEMENT     INCLINATION    ELEMENT    ELEMENT     ELEMENT
     1          ELEMENT          DOWEL    STEEL MESH   MESH SIZE   DOWEL',/,
     2' NUMBER      OF ELEMENT         SIZE        TYPE     THICKNESS
     3NODE          INDICATOR  INDICATOR    INDICATOR   WIDTH',/,
     4' INDICATOR     INDICATOR    INDICATOR                          N
     5UMBERS')
  110 FORMAT(3X,I3,2(10X,I2),9X,I1,6X,F6.4,4X,4(I3,3X),1X,I1,10X,
     1I1,14X,I1,6X,F7.3)
  115 FORMAT(//' DATA FOR INITIAL CONCRETE SHRINKAGE STRESSES IS AS BELO
     1W',/,' NUMBER OF ELEMENTS IN EACH FLANGE=',I3/,
     2' LONGITUDIONAL SHRINKAGE STRESS IN TOP FLANGE=',E13.4/,
     3' TRANSVERSE SHRINKAGE STRESS IN TOP FLANGE=',E13.4/,
     4' LONGITUDIONAL SHRINKAGE STRESS IN BOTTOM FLANGE=',E13.4/,
     5' TRANSVERSE SHRINKAGE STRESS IN BOTTOM FLANGE=',E13.4/,
     6' LONGITUDIONAL SHRINKAGE STRESS IN WEBS=',E13.4/,
     7' TRANSVERSE  SHRINKAGE STRESS IN WEBS=',E13.4/,
     8' LONGITUDIONAL SHRINKAGE STRAIN IN TOP FLANGE=',E13.4/,
     9' TRANSVERSE SHRINKAGE STRAIN IN TOP FLANGE=',E13.4/,
     1' LONGITUDIONAL SHRINKAGE STRAIN IN BOTTOM FLANGE=',E13.4/,
     2' TRANSVERSE SHRINKAGE STRAIN IN BOTTOM FLANGE=',E13.4/,
     3' LONGITUDIONAL SHRINKAGE STRAIN IN WEBS=',E13.4/,
     4' TRANSVERSE SHRINKAGE STRAIN IN WEBS=',E13.4//,
     5' FLANGE ELEMENT NUMBERS ARE AS BELOW ; CONSECUTIVE ROWS OF TOP AN
     6D BOTTOM FLANGE ELEMENTS.'/)
  116 FORMAT(' ELTP(',I3,' ,',I3,' )=',10I5)
  117 FORMAT(' ELBM(',I3,' ,',I3,' )=',10I5)
  120 FORMAT(///' NUMBER OF ELEMENTS REINFORCED WITH A STEEL MESH=',
     1I4)
  125 FORMAT(/'     ELEMENT       NUMBER OF       DIRN.     STEEL      STEEL
     1            ',/,'        NUMBER      DIRECTIONS      NUMBER   PERCENTAGE
     2ANGLE    ',/,'                       OF REINFORCEMENT')
  130 FORMAT(6X,I3,11X,I2,10X,I2,7X,F6.4,4X,F5.1)
  140 FORMAT(///' DIAPHRAGM INPUT DATA IS AS BELOW',//,
     1' TOTAL NUMBER OF DIAPHRAGM ELEMENTS IN COMPLETE BEAM =',I2/)
  147 FORMAT(//' DATA FOR ACTUAL AND PSEUDO DIAPHRAGMS IS AS BELOW',/,
     1' ELEMENT ELEMENT  ELEMENT              ELEMENT     REFERENCE   SHEAR'
     2,/,'     NO.      TYPE  THICKNESS       NODE NUMBERS       NO.       MODUL
     3US')
  148 FORMAT(2X,I3,7X,I1,6X,F5.2,5X,4(I3,1X),3X,I2,4X,E13.4)
  150 FORMAT(///' NUMBER OF SINGLE REINFORCING BARS AND BOND SLIP LINKAG
     1ES =',I4)
  152 FORMAT(/' ELEMENT   FIRST   SECOND      REO.     DIRECTION',
     15X,' AREA         INITIAL          INITIAL',/,
     2'  NUMBER    NODE    NODE  TYPE CODE  INDICATOR',
     315X,' PRESTRESS       PRESTRAIN'/)
  155 FORMAT(3X,I3,5X,I3,6X,I3,6X,I2,10X,I2,8X,F6.4,4X,E11.5,2X,E11.5)
  158 FORMAT(3X,I3,5X,I3,4X,I3,6X,I2,17X,F9.4)
  175 FORMAT(///' NUMBER OF FINITE ELEMENT NODES =',I5)
  180 FORMAT(/'     NODE      NODE TYPE         X                   Y                Z',/,'
     1 NUMBER    INDICATOR   COORDINATE   COORDINATE   COORDINATE')
  185 FORMAT(4X,I3,6X,I2,7X,F7.2,2(5X,F7.2))
  195 FORMAT(///' THE STRENGTH AND DEVIATION PARAMETERS ARE AS BELOW'/)
  200 FORMAT(' INITIAL YOUNG|S MODULUS FOR CONCRETE =',E15.5/,' INITIAL
     1YOUNG|S MODULUS FOR CONVENTIONAL REINFORCEMENT =',E15.5/,' INITIAL
     2 YOUNG|S MODULUS FOR PRESTRESSING STEEL =',E15.5/,' INITIAL BOND S
     3LIP LINKAGE STIFFNESS =',E15.5,' LBS/CU. INCH',/,
     4 ' INITIAL STEEL MESH MODULUS =',E15.5)
  205 FORMAT(/' COMPRESSIVE CONCRETE STRENGTH =',E15.5,' LBS./SQ.INCH',/
     1,' TENSILE CONCRETE STRENGTH =',E15.5,' LBS./SQ.INCH',/,' ULTIMATE
     2 AGGREGATE INTERLOCK STRENGTH =',E14.5,' LBS./SQ.IN.',/,' ULTIMATE
     3 TENSILE STRENGTH OF CONVENTIONAL REINFORCEMENT =',E15.5,' LBS./SQ
     4.INCH',/,' ULTIMATE TENSILE STRENGTH OF PRESTRESSING STEEL =',E15.
     55,' LBS./SQ.INCH',/,' ULTIMATE TENSILE STRENGTH OF STEEL MESH BARS
     6 =',E15.5,' LBS/SQ.INCH')
  210 FORMAT(/' YIELD STRAIN FOR CONVENTIONAL REINFORCEMENT =',E14.5/,
     1' YIELD STRAIN FOR PRESTRESS REINFORCEMENT=',E14.5/,
     2       ' MAXIMUM CONCRETE COMPRESSIVE STRAIN =',F8.6/,' MAXIMUM C
     3ONVENTIONAL REINFORCEMENT TENSILE STRAIN =',F8.6/,' MAXIMUM PRESTR
     4ESSING STEEL TENSILE STRAIN =',F8.6/,' MAXIMUM STEEL MESH BAR TENS
     5ILE STRAIN =',F8.6/,' MAXIMUM BOND LINKAGE SLIP =',F8.6,' INCHES
     7',/,' DOWEL FAILURE DISPLACEMENT =',F8.6,' INCHES')
  215 FORMAT(/' POISSON RATIO FOR CONCRETE IN BIAXIAL COMPRESSION =',F6.
     15/,' POISSON RATIO FOR CONCRETE IN BIAXIAL TENSION =',F6.5/,' POIS
     2SON RATIO FOR CONCRETE IN TENSION-COMPRESSION =',F6.5/,' UNIAXIAL
     3POISSON RATIO FOR CONCRETE =',F6.5)
  219 FORMAT(/' MAXIMUM NUMBER OF ITERATIONS BEFORE EXECUTION IS TERMINA
     1TED=',I3/,' CONCRETE DEVIATION WEIGHTING =',I2/,
     2' CONVENTIONAL REINFORCEMENT DEVIATION WEIGHTING =',I2/,
     3' PRESTRESS STRAND DEVIATION WEIGHTING =',I2)
  220 FORMAT(///' LOADING INFORMATION FOLLOWS',//,' NUMBER OF DIFFERENT
     1LOADING TYPES TO BE SUPERIMPOSED ON MODEL =',I3)
  225 FORMAT('  LOAD TYPE NUMBER      NO. OF INCREMENTS     NO. OF NODAL L
     1OADS')
  230 FORMAT(9X,I3,21X,I3,22X,I3)
```

```
  235 FORMAT(' NODE OF APPLICATION    LOAD DIRECTION CODE    LOAD MAGNI
     1TUDE')
  240 FORMAT(9X,I3,21X,I3,13X,E14.6)
  255 FORMAT(///' IMPOSED BOUNDARY CONDITIONS ARE AS BELOW',//,' NUMBER
     1OF IMPOSED BOUNDARY CONDITIONS =',I4)
  260 FORMAT(/'   NODE OF APPLICATION     DIRECTION OF     IMPOSED',/,'
     1OF IMPOSED BOUNDARY   APPLICATION CODE   VALUE',/,'        CONDITI
     2ONS')
  265 FORMAT(10X,I3,18X,I3,8X,E12.4)
  218 FORMAT(' MAXIMUM ALLOWABLE PERCENTAGE DEVIATION OF CONCRETE =',F
     18.3/,' MAXIMUM ALLOWABLE PERCENTAGE DEVIATION OF MAIN DIAGONAL',/,
     2' TERMS OF CONCRETE CONSTITUTIVE MATRIX =',F8.5/,
     4                   ' MAXIMUM ALLOWABLE PERCENTAGE DEVIATION OF REINFOR
     5CEMENT STIFFNESS ',/,' BEFORE CHANGE MUST BE MADE =',F9.5/,
     6                        ' MAXIMUM ALLOWABLE PERCENTAGE DEVIATION OF PRES
     7TRESS BARS =',F8.3/,' MAXIMUM ALLOWABLE PERCENTAGE DEVIATION FOR B
     8OND SLIP LINKAGE =',F8.3/,' CRITICAL NUMBER OF MATERIAL DEVIATIONS
     9 TO INVOKE ITERATIVE METHOD =',I3/,' AVERAGE CRACK SPACING =',F6.3
     9,' INCHES',/,' RELAXATION FACTOR =',F6.3)
  271 FORMAT(///' SCREEN PRINTOUT DATA IS AS BELOW')
  272 FORMAT(/' THE ELEMENT NUMBERS OF THE',I2,' CONCRETE ELEMENTS WHOSE
     1 (STRESS,STRAIN) STATE',/,' WILL BE DISPLAYED, ARE BELOW',
     2/10I4)
  273 FORMAT(/' THE',I2,' STEEL MESH ELEMENT NUMBERS WHOSE (STRESS,STRAI
     1N) STATES WILL BE DISPLAYED, ARE BELOW',/10I4)
  274 FORMAT(/' THE ',I2,' REINFORCEMENT ELEMENTS WHOSE (STRESS,STRAIN)
     1STATES WILL BE DISPLAYED, ARE BELOW',/10I4)
  275 FORMAT(/' THE',I2,' NODES WHOSE DISPLACEMENTS WILL BE DISPLAYED AR
     1E BELOW',/10I4)
  278 FORMAT(///' PRINTOUT FORMAT CONTROL VARIABLES ARE AS BELOW')
  280 FORMAT(/' DEFLECTIONS CONTROL VARIABLE =',I2/,' CENTROIDAL CONCRET
        STRESSES CONTROL VARIABLE =',I2/,' PRINCIPAL CONCRETE STRESSES C
         TROL VARIABLE=',I2/,' STEEL MESH STRESSES CONTROL VARIABLE =',I2
     3/,' REINFORCEMENT ELEMENT STRESSES CONTROL VARIABLE =',I2/,' LOAD
     4INCREMENT CONTROL VARIABLE =',I2/,' TOTAL LOAD CONTROL VARIABLE ='
     5,I2/,' STIFFNESS ASSEMBLAGE CONTROL VARIABLE =',I2/)
  400 FORMAT(///'                                                   *** E
     1ND OF INPUT ECHO CHECK ***',/,' **********************************
     2*******************************************************************
     3**********************************',///)
      RETURN
      END
      SUBROUTINE ADDNEL(NS1,NS2,NS3,TS,AL,AR)
C***********************************************************************
C     THIS SUBROUTINE PARTITIONS AND ADDS TO THE IN-CORE STIFFNESS
C     MATRIX TS(NBAND,2*NBAND), THE ELEMENT STIFFNESS MATRIX ESTF(12,12)
C***********************************************************************
C
      COMMON/BLOCK2/NEL,NELT,NELCHN,NELD,INDCEL(185),INELSE(185),
     1INDOWL(185),NIDTEC(185),CANGLE(185),CALTER(185),
     2IRMLTY(215),MLTNE(215),NODEEL(215,4),NDREF(215),DPMNOD(215)
      COMMON/BLCK10/ESTF(12,12),BS(3,3),B1(3,12,9),B2(3,12,9),D(3,3),
     1DCONC(185,3,3),DCRACK(185),DAGG(185)
      INTEGER*4 CALTER
      DIMENSION TS(NS1,NS2),AL(NS3),AR(NS3)
      DO 10 I=1,4
      II=NODEEL(NEL,I)
      DO 10 J=1,4
      JJ=NODEEL(NEL,J)
      IF(JJ.GT.II) GO TO 10
C     PARTITIONING OF (12*12) MATRIX INTO 16 BS(3,3) SUB-BLOCKS.
      DO 5 K=1,3
      DO 5 L=1,3
      LL=((I-1)*3+L)
      KK=((J-1)*3+K)
      BS(L,K)=ESTF(LL,KK)
    5 CONTINUE
      IF(INDCEL(NEL).NE.0) GO TO 8
      CALL NORIS
C     ADDITION INTO TOTAL STIFFNESS MATRIX
    8 CALL BLKADD(II,JJ,NS1,NS2,NS3,TS,AL,AR)
   10 CONTINUE
      RETURN
      END
      SUBROUTINE SOLVE(NS1,NS2,NS3,NS4,TS,AL,AR,RT,RL,RR)
C***********************************************************************
C     THIS SUBROUTINE SOLVES THE SET OF EQUATIONS IN A BLOCK BY BLOCK
C     GAUSSIAN ELIMINATION PROCESS. ONLY TWO BLOCKS ARE IN CORE AT ANY
C     ONE TIME.
C***********************************************************************
C
      COMMON/BLOCK1/NEQNS,NBAND,NBLK,NUMINC,ICOUNT
      COMMON/BLOCK9/LPOINT(20,2),FDUB1,FDUB2,NUMREC,
     1NUMBLK,ISOLVE,LEN
      COMMON/BLCK11/RTOT(750),REOTOT(750),DTOT(750),R2(2,750),D2(2,750)
      DIMENSION TS(NS1,NS2),AL(NS3),AR(NS3),RT(2,NS2),RL(2,NS1),
     1RR(2,NS1)
      INTEGER*4 FDUB1,FDUB2,INFO(4),ITIMF
      INTEGER*2 LEN
      ISOLVE=ISOLVE+1
      REWIND 1
      REWIND 2
      NBLK=0
      IF(ISOLVE.GT.1) GO TO 20
      CALL TIME(1,0,ITIME)
      COSTMY=COST(0)
      WRITE (6,200) ITIME,COSTMY
      GO TO 20
C     SHIFTING SECOND BLOCK INTO FIRST BLOCK POSITION
   10 NBLK=NBLK+1
      DO 4 I=1,NBAND
      DO 4 J=1,2
      RL(J,I)=RR(J,I)
      RR(J,I)=0.0
    4 CONTINUE
      DO 5 I=1,NS3
      AL(I)=AR(I)
      AR(I)=0.0
    5 CONTINUE
C     NEXT STIFFNESS BLOCK IS READ INTO CORE AND CORRESPONDING LOAD
C     VECTOR IS FORMED
      IF(NUMBLK-NBLK) 20,28,20
   20 NB=NBLK+1
      INFO(1)=LPOINT(NB,1)
      CALL POINT(FDUB1,INFO,1,&280)
      J=1
      J1=LEN
      DO 25 I=1,NUMREC
      CALL READ(AR(J),LEN,0,LNUM,FDUB1,&250)
      J=J+LEN/4
```

```
      IF(I.EQ.(NUMREC-1)) LEN=(NS3-J+1)*4
   25 CONTINUE
      LEN=J1
      DO 27 I=1,NBAND
      I1=(NBLK*NBAND)+I
      DO 27 J=1,2
      IF(I1.LE.NEQNS) GO TO 26
      RR(J,I)=0.0
      GO TO 27
   26 RR(J,I)=R2(J,I1)
   27 CONTINUE
      IF(NBLK.LT.1) GO TO 10
      IF(NBLK.NE.1.OR.ISOLVE.GT.1) GO TO 34
      CALL TIME(1,0,ITIME)
      WRITE (6,210) ITIME
      GO TO 34
   28 DO 30 I=1,NS3
      AR(I)=0.0
   30 CONTINUE
      DO 32 I=1,NBAND
      DO 32 K=1,2
      RR(K,I)=0.0
   32 CONTINUE
C     COMPLETE REDUCTION OF FIRST BLOCK IN CORE
   34 DO 70 I=1,NBAND
      IF(NBLK.LT.NUMBLK) GO TO 36
      NEQ=(NBLK-1)*NBAND+I
      IF(NEQ.GT.NEQNS) GO TO 75
   36 IF(TS(1,I).LE.0.0) GO TO 240
      DO 38 J=1,2
      RT(J,I)=RT(J,I)/TS(1,I)
   38 CONTINUE
      DO 60 J=2,NBAND
      IF(TS(J,I)) 40,60,40
   40 C=TS(J,I)/TS(1,I)
      K=I+J-1
      L=0
      DO 50 N=J,NBAND
      L=L+1
   50 TS(L,K)=TS(L,K)-C*TS(N,I)
      DO 55 N=1,2
      RT(N,K)=RT(N,K)-TS(J,I)*RT(N,I)
   55 CONTINUE
      TS(J,I)=C
   60 CONTINUE
   70 CONTINUE
      IF(NBLK.NE.1.OR.ISOLVE.GT.1) GO TO 75
      CALL TIME(1,0,ITIME)
      WRITE (6,215) ITIME
C     REDUCED BLOCK IS WRITTEN OUT ON DISC
   75 CALL NOTE(FDUB2,INFO,&260)
      LPOINT(NBLK,2)=INFO(2)
      J=1
      J1=LEN
      DO 80 I=1,NUMREC
      CALL WRITE(AL(J),LEN,0,LNUM,FDUB2,&270)
      J=J+LEN/4
      IF(I.EQ.(NUMREC-1)) LEN=(NS3-J+1)*4
   80 CONTINUE
      LEN=J1
      DO 90 I=1,NBAND
      I1=(NBLK-1)*NBAND+I
      DO 90 J=1,2
      IF(I1.GT.NEQNS) GO TO 90
      R2(J,I1)=RL(J,I)
   90 CONTINUE
      IF(NBLK.EQ.NUMBLK) GO TO 100
      IF(NBLK.NE.1.OR.ISOLVE.GT.1) GO TO 10
      CALL TIME(1,0,ITIME)
      WRITE (6,220) ITIME
      GO TO 10
C     FINAL STEP IN EQUATION SOLUTION INVOLVING BACKSUBSTITUTION OF
C     REDUCED EQUATIONS AND LOAD ARRAY
  100 DO 95 I=1,NBAND
      DO 95 J=1,2
      RP(J,I)=0.0
   95 CONTINUE
      NBLK=NUMBLK
      IF(ISOLVE.GT.1) GO TO 110
      CALL TIME(1,0,ITIME)
      WRITE (6,225) ITIME
  110 INFO(1)=LPOINT(NBLK,2)
      CALL POINT(FDUB2,INFO,1,&280)
      J=1
      J1=LEN
      DO 120 I=1,NUMREC
      CALL READ(AL(J),LEN,0,LNUM,FDUB2,&290)
      J=J+LEN/4
      IF(I.EQ.(NUMREC-1)) LEN=(NS3-J+1)*4
  120 CONTINUE
      LEN=J1
      DO 130 I=1,NBAND
      I1=(NBLK-1)*NBAND+I
      DO 130 J=1,2
      IF(I1.LE.NEQNS) GO TO 125
      RL(J,I)=0.0
      GO TO 130
  125 RL(J,I)=R2(J,I1)
  130 CONTINUE
      DO 150 I=1,NBAND
      J=NBAND+1-I
      JD=(NBLK-1)*NBAND+J
      DO 140 K=2,NBAND
      L=J+K-1
      DO 140 J1=1,2
  140 RT(J1,J)=RT(J1,J)-TS(K,J)*RT(J1,L)
      N=NBAND+J
      DO 150 J1=1,2
      RT(J1,N)=RT(J1,J)
      IF(JD.GT.NEQNS) GO TO 150
      D2(J1,JD)=RT(J1,J)
  150 CONTINUE
      NBLK=NBLK-1
      IF(NBLK) 110,160,110
C     BACK SUBSTITUTION COMPLETED
  160 IF(ISOLVE.GT.1) RETURN
      CALL TIME(1,0,ITIME)
      COSTEY=COST(0)
      WRITE (6,230) ITIME,COSTEY
C     FORMAT STATEMENTS
```

```
$SIGNON TAIL RIBBON=CA PRINT=TN
JOB-TYPE=BATCH, PRIO=NORMAL, CLASS=INTERNAL/TEACHING,RESEARCH
**LAST SIGNON WAS: 12:47:46
 USER "TAIL" SIGNED ON AT  14:13:37 ON THU JUN 30/77
$LIST FILE52(1319,1721)
      200  FORMAT(//' ****** OUTPUT FROM SUBROUTINE SOLVE ******',//,' CPU TI
          1ME AT INITIAL STAGE IN SUBROUTINE SOLVE =',I8,
          2' PROGRAM COST =',F7.2)
      210  FORMAT(' CPU TIME AT COMPLETION OF BLOCK MANIPULATION OF BLOCK NUM
          1BER 1 =',I8)
      215  FORMAT(' CPU TIME AT COMPLETION OF GAUSSIAN ELIMINATION OF BLOCK N
          1UMBER 1 =',I8)
      220  FORMAT(' CPU TIME AFTER REDUCED EQUATIONS OF BLOCK NUMBER 1 HAVE B
          1EEN WRITTEN OUT ON DISC =',I8)
      225  FORMAT(' CPU TIME AT BEGINNING OF BACKSUBSTITUTION PROCESS =',I8)
      230  FORMAT(' CPU TIME AT END OF SOLUTION PROCESS =',I8,
          1' PROGRAM COST =',F7.2)
           RETURN
      240  WRITE (6,245) I,TS(1,I),NBLK
      245  FORMAT(///' *** DIAGONAL TERM TS(',I3,' ,1)=',E14.5,
          1' IN BLOCK NO.',I3,' ***'///)
           STOP
      250  WRITE (6,252)
      252  FORMAT(////' *** ERROR IN FIRST READ SYSTEM SUBROUTINE IN SOLVE,
          1AS NEXT STIFFNESS BLOCK IS READ INTO CORE ***')
           STOP
      260  WRITE (6,262)
      262  FORMAT(////' *** ERROR IN FIRST NOTE SYSTEM SUBROUTINE IN SOLVE')
           STOP
      270  WRITE (6,272)
      272  FORMAT(////' *** ERROR IN WRITE SYSTEM SUBROUTINE IN SOLVE AS REDU
          1CED BLOCK OF EQUATIONS IS WRITTEN OUT ONTO DISC')
           STOP
      280  WRITE (6,282)
      282  FORMAT(///' ***ERROR IN SECOND NOTE SYSTEM SUBROUTINE IN SOLVE')
           STOP
      290  WRITE (6,292)
      292  FORMAT(////' ERROR IN SECOND READ SYSTEM SUBROUTINE IN SOLVE AS BL
          1OCK OF REDUCED EQUATIONS IS READ INTO CORE, PRECEDING BACKSUBSTITU
          2TION')
           STOP
           END
           SUBROUTINE MODIFY (NS1,NS2,NS3,TS,AL,AR)
C*********************************************************************
C           THIS SUBROUTINE MODIFIES A GIVEN STIFFNESS BLOCK,WHERE APPLICABLE
C          FOR SPECIFIED BOUNDARY CONDITIONS
C*********************************************************************
C
           COMMON/BLOCK1/NEQNS,NBAND,NBLK,NUMINC,ICOUNT
           COMMON/BLOCK8/NDISPL,NODED(30),KODED(30),VALUED(30),D1(750)
           COMMON/BLCK11/RTOT(750),REOTOT(750),DTOT(750),R2(2,750),D2(2,750)
           DIMENSION TS(NS1,NS2),AL(NS3),AR(NS3)
C          DETERMINE WHETHER ANY STIFFNESS ELEMENTS IN BLOCK ARE AFFECTED BY
C          BOUNDARY CONDITIONS
           NCOL1=1
           NCOL2=NBAND
           NCOL3=NBAND*2
           DO 50 I=1,NDISPL
           II=NODED(I)
           JJ=1
           CALL LOCATE(II,JJ,NROW,NCOL)
           NR=NROW+KODED(I)-1
           IF(NR.LT.NCOL1.OR.NR.GT.NCOL3) GO TO 50
C          APPROPRIATE ELEMENT CHANGES ARE NOW MADE.
           IF(NR.GT.NCOL2) GO TO 10
           DO 5 J=2,NBAND
           TS(J,NR)=0.0
    5      CONTINUE
           DO 6 J=2,NBAND
           J1=(NR+1-J)
           IF(J1.LT.NCOL1) GO TO 7
           TS(J,J1)=0.0
    6      CONTINUE
    7      TS(1,NR)=1.0
           I1=(NBLK-1)*NBAND+NR
           DO 8 J=1,2
           R2(J,I1)=VALUED(I)
    8      CONTINUE
           REOTOT(I1)=VALUED(I)
           RTOT(I1)=VALUED(I)
           GO TO 50
   10      DO 20 J=2,NBAND
           J1=(NR+1-J)
           IF(J1.GT.NCOL2) GO TO 20
           TS(J,J1)=0.0
   20      CONTINUE
   50      CONTINUE
           RETURN
           END
           SUBROUTINE OUTPUT(NCI)
C*********************************************************************
C           THIS SUBROUTINE PRINTS OUT ALL RELEVANT COMPUTED BEAM BEHAVIOURAL
C          INFORMATION AT THE COMPLETION OF THE LAST LOAD INCREMENT
C*********************************************************************
C
           COMMON/BLOCK1/NEQNS,NBAND,NBLK,NUMINC,ICOUNT
           COMMON/BLOCK2/NEL,NELT,NELCHK,NELD,INDCEL(185),INELSZ(185),
          1INDOWL(185),WIDTHC(185),CANGLE(185),CALTER(185),
          2INELTY(215),ELTHN(215),NODEEL(215,4),NDREF(215),DPSNOD(215)
           COMMON/BLOCK3/NELSH,INMESH(185),INSZMS(185),NDIRNS,PERSFL(185,4),
          1ANGLE(185,4),SHSTF(185,3,3),SHESH(185,1)
           COMMON/BLOCK4/NR,NREO,NODES(190,2),INNDN(190),INFTY(190),
          1RALTER(190),BAREA(190),CAREA(190),RSTF(190),TSGPRE(190),TEPRE(190)
           COMMON/BLOCK5/NNODES,ICNODE(220),X(220),Y(220),Z(220)
           COMMON/BLCK11/RTOT(750),REOTOT(750),DTOT(750),R2(2,750),D2(2,750)
           COMMON/BLCK12/SIGCON(185,3),ECON(185,3),SIGMS(185,4),EMS(185,4),
          1TSGCON(185,3),TECON(185,3),TSGCC(185),TSGCT(185),TSGAGG(185),
          2TECC(185),TECT(185),TSGMS(185,4),TEMS(185,4),SIGREO(190),
          3EREO(190),TSGREO(190),TEREO(190)
           COMMON/BLCK13/NTMESH(185,4),NDOWEL(185),NCRACK(185),NTAGG(185),
          1NTREO(190),NTCREO(190)
           COMMON/BLCK14/IDEFLN,ICON3,ICONPR,IMESH,IREO,ILOAD,ITLOAD,ISTIF,
          1MAXIT,NTCON,NTPEO,NTPRE
           COMMON/BLCK16/PNELT,PNEL(10),PNELSH,PSH(10),PNREO,PNR(10),
          1PDTOT,PDT(10)
           INTEGER*4 CALTER,RALTER,PNELT,PNEL,PNELSH,PSH,PNREO,PNR,
          1PDTOT,PDT
           WRITE (6,100) NUMINC
           WRITE (7,100) NUMINC
           WRITE (6,105) NCI
```

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C     NODAL DISPLACEMENT PRINTOUT
      IF(IDEFLN.EQ.0) GO TO 25
      WRITE (6,115)
      WRITE (7,115)
      J=1
      DO 20 I=1,NNODES
      NDT=0
      DO 5 K=1,PDTOT
      IF(PDT(K).NE.I) GO TO 5
      NDT=1
      GO TO 8
    5 CONTINUE
    8 IF(ICNODE(I)) 10,12,16
   10 WRITE (6,120) I,DTOT(J)
      IF(NDT.EQ.1) WRITE (7,120) I,DTOT(J)
      J=J+1
      GO TO 20
   12 J1=J+2
      NODENO=I
      CALL ELLOC(NODENO,NL)
      IF(INDCEL(NL).EQ.0) GO TO 14
      WRITE (6,125) I,(DTOT(K),K=J,J1)
      IF(NDT.EQ.1) WRITE (7,125) I,(DTOT(K),K=J,J1)
      J=J+3
      GO TO 20
   14 WRITE (6,127) I,(DTOT(K),K=J,J1)
      IF(NDT.EQ.1) WRITE (7,127) I,(DTOT(K),K=J,J1)
      J=J+3
      GO TO 20
   16 ICC=ICNODE(I)-2
      IF(ICC) 19,18,17
   17 J1=J+1
      WRITE (6,128) I,(DTOT(K),K=J,J1)
      IF(NDT.EQ.1) WRITE (7,128) I,(DTOT(K),K=J,J1)
      J=J+2
      GO TO 20
   18 J1=J+3
      WRITE (6,129) I,(DTOT(K),K=J,J1)
      IF(NDT.EQ.1) WRITE (7,129) I,(DTOT(K),K=J,J1)
      J=J+4
      GO TO 20
   19 J1=J+4
      WRITE (6,130) I,(DTOT(K),K=J,J1)
      IF(NDT.EQ.1) WRITE (7,130) I,(DTOT(K),K=J,J1)
      J=J+5
   20 CONTINUE
C     CENTROIDAL CONCRETE STRESSES PRINTOUT FOR EACH ELEMENT
   25 IF(ICON3.EQ.0) GO TO 35
      WRITE (6,135)
      WRITE (7,135)
      DO 30 I=1,NELT
      WRITE (6,140) I,(TSGCON(I,K),K=1,3),TSGAGG(I)
      DO 28 J=1,PNELT
      IF(I.NE.PNEL(J)) GO TO 28
      WRITE (7,140) I,(TSGCON(I,K),K=1,3),TSGAGG(I)
      GO TO 30
   28 CONTINUE
   30 CONTINUE
C     CENTROIDAL PRINCIPAL COMPRESSIVE AND TENSILE STRESSES AND
C     STRAINS ARE PRINTED OUT FOR EACH ELEMENT
   35 IF(ICOMPR.EQ.0) GO TO 50
      WRITE (6,145)
      DO 45 I=1,NELT
      IF(NCRACK(I).EQ.1) GO TO 40
      WRITE (6,150) I,TSGCC(I),TECC(I),TSGCT(I),TECT(I)
      GO TO 45
   40 WRITE (6,155) I,TSGCC(I),TECC(I),TSGCT(I),TECT(I)
   45 CONTINUE
C     PRINTOUT OF STEEL MESH STRESSES
   50 IF(IMESH.EQ.0) GO TO 60
      WRITE (6,160)
      WRITE (7,160)
      DO 55 I=1,NELT
      IF(INMESH(I).EQ.0) GO TO 55
      DO 51 J=1,NDIRMS
      WRITE (6,165) I,J,TSGMS(I,J),TEMS(I,J)
   51 CONTINUE
      DO 53 J=1,PNELSM
      IF(I.NE.PSM(J)) GO TO 53
      DO 52 K=1,NDIRMS
      WRITE (7,165) I,K,TSGMS(I,K),TEMS(I,K)
   52 CONTINUE
      GO TO 55
   53 CONTINUE
   55 CONTINUE
C     REINFORCEMENT ELEMENT STRESS PRINTOUT
   60 IF(IREO.EQ.0) GO TO 75
      WRITE (6,168)
      WRITE (7,168)
      DO 70 I=1,NREO
      INR=0
      DO 61 J=1,PNREO
      IF(I.NE.PNR(J)) GO TO 61
      INR=1
      GO TO 62
   61 CONTINUE
   62 IF(INRTY(I)) 63,64,66
   63 WRITE (6,170) I,TSGREO(I),TEREO(I)
      IF(INR.EQ.0) GO TO 70
      WRITE (7,170) I,TSGREO(I),TEREO(I)
      GO TO 70
   64 WRITE (6,175) I,TSGREO(I),TEREO(I)
      IF(INR.EQ.0) GO TO 70
      WRITE (7,175) I,TSGREO(I),TEREO(I)
      GO TO 70
   66 WRITE (6,180) I,TSGREO(I),TEREO(I)
      IF(INR.EQ.0) GO TO 70
      WRITE (7,180) I,TSGREO(I),TEREO(I)
   70 CONTINUE
C     LOAD INCREMENT PRINTOUT
   75 IF(ILOAD.EQ.0) GO TO 85
      CALL LOAD(MAXINC)
      WRITE (6,185)
      DO 80 I=1,NEQNS
      IF(R2(1,I).GT.-1.0E-03.AND.R2(1,I).LT.1.0E-03) GO TO 80
      NEQ=I
      CALL NODLOC(NEQ,NN)
      II=NN
      JJ=1
      NBLK=1
```

```
      CALL LOCATE(II,JJ,NROW,NCOL)
      MM=I-NROW+1
      WRITE (6,190) NN,MM,R2(1,I)
   80 CONTINUE
C     TOTAL LOAD PRINTOUT
   85 IF(ITLOAD.EQ.0) RETURN
      WRITE (6,195)
      DO 90 I=1,NEQNS
      IF(RTOT(I).GT.-1.0E-03.AND.RTOT(I).LT.1.0E-03) GO TO 90
      NEQ=I
      CALL NODLOC(NEQ,NN)
      II=NN
      JJ=1
      NBLK=1
      CALL LOCATE(II,JJ,NROW,NCOL)
      MM=I-NROW+1
      WRITE (6,200) NN,MM,RTOT(I)
   90 CONTINUE
C     TOTAL REINFORCEMENT RESTRAINT VECTOR PRINTOUT
      WRITE (6,210)
      WRITE (7,210)
      DO 95 I=1,NEQNS
      IF(REOTOT(I).EQ.0.0) GO TO 95
      NEQ=I
      CALL NODLOC(NEQ,NN)
      II=NN
      JJ=1
      NBLK=1
      CALL LOCATE(II,JJ,NROW,NCOL)
      MM=I-NROW+1
      WRITE (6,220) NN,MM,REOTOT(I)
      WRITE (7,220) NN,MM,REOTOT(I)
   95 CONTINUE
      RETURN
C**** FORMAT STATEMENTS
  100 FORMAT(////' *******************************************************
     1******',/,' *  LOAD INCREMENT NUMBER ',I2,' STRESS-DEFORMATION PRIN
     2TOUT  *',/,' *******************************************************
     3******'//)
  105 FORMAT(' THE NUMBER OF CONCRETE ELEMENTS THAT CRACKED IN THIS LOAD
     1 INCREMENT =',I5)
  115 FORMAT(//' THE NODAL DISPLACEMENTS ARE AS FOLLOWS',/,
     1'   NODE              X                     Y                     Z
     2         THETAY                THETAZ')
  120 FORMAT(I7,3X,E14.5)
  125 FORMAT(I7,3X,E14.5,5X,E14.5,43X,E14.5)
  127 FORMAT(I7,3X,E14.5,24X,E14.5,5X,E14.5)
  128 FORMAT(I7,22X,2(E14.5,5X))
  129 FORMAT(I7,3X,3(E14.5,5X),19X,E14.5)
  130 FORMAT(I7,3X,5(E14.5,5X))
  135 FORMAT(//' CENTROIDAL CONCRETE STRESSES TSGCON(NEL,3) AND AGGREGAT
     1E INTERLOCK STRESS TSGAGG(NEL) ARE AS BELOW',/,' ELEMENT NO.
     2SIGMAX              SIGMAY(Z)             SIGMAXY(XZ)          SIGMA-AGG')
  140 FORMAT(I7,5X,4(E14.5,4X))
  145 FORMAT(//' PRINCIPAL COMPRESSIVE AND TENSILE STRESSES AND STRAINS
     1FOR EACH ELEMENT CENTROID ARE AS BELOW',/,
     2' ELEMENT NO.      PRINCIPAL             PRINCIPAL            PRINCIPAL
     3        PRINCIPAL              ELEMENT',/,'                   COMPRESSIVE
     4     COMPRESSIVE            TENSILE               TENSILE              CRACKE
     5D ?',/,'                       STRESS                STRAIN               STRE
     6SS                   STRAIN')
  150 FORMAT(I7,5X,4(E14.5,4X),7X,' NO')
  155 FORMAT(I7,5X,4(E14.5,4X),6X,' YES')
  160 FORMAT(//' STEEL MESH STRESSES ARE AS BELOW.',/,' ELEMENT    MESH',
     1/,'    NO.    DIRECTION'/)
  165 FORMAT(I6,6X,I2,4X,' STRESS=',E14.5,'  STRAIN=',E14.5)
  168 FORMAT(//' REINFORCEMENT ELEMENT STRESSES AND STRAINS ARE AS BELOW
     1;',/,' REINFORCEMENT   REINFORCEMENT          STRESS               STRA
     2IN',/,'     NUMBER            TYPE')
  170 FORMAT(I8,8X,' CONVENTIONAL',5X,E14.5,4X,E14.5)
  175 FORMAT(I8,8X,' PRESTRESSED',5X,E14.5,4X,E14.5)
  180 FORMAT(I8,11X,' BONDED',7X,E14.5,4X,E14.5)
  185 FORMAT(//' THE LOAD INCREMENT VECTOR IS AS BELOW;')
  190 FORMAT('    RINC(',I3,' ,',I1,' ) =',E14.5)
  195 FORMAT(//' THE TOTAL LOAD VECTOR IS AS BELOW;')
  200 FORMAT('    RTOTAL(',I3,' ,',I1,' ) =',E14.5)
  210 FORMAT(//' TOTAL REINFORCEMENT FORCE RESTRAINT VECTOR;')
  220 FORMAT('   RESTRAINT FORCE AT NODE',I3,' IN DIRECTION',
     1I2,' =',E14.5)
      RETURN
      END
      SUBROUTINE ELLOC(NODENO,NL)
C**********************************************************************
C     THIS SUBROUTINE LOCATES THE ELEMENT NUMBER NL THAT CONTAINS THE
C     INTERIOR NODE NUMBER NODENO.
C**********************************************************************
C
      COMMON/BLOCK2/NEL,NELT,NELCHK,NELD,INDCEL(185),INELSZ(185),
     1INDOWL(185),WIDTHC(185),CANGLE(185),CALTER(185),
     2INELTY(215),ELTHN(215),NODEEL(215,4),NDREP(215),DPMMOD(215)
      INTEGER*4 CALTEP
      NL=1
      DO 5 I=1,NELT
      DO 5 J=1,4
      IF(NODEEL(I,J).NE.NODENO) GO TO 5
      NL=I
      GO TO 20
    5 CONTINUE
      WRITE (6,10) NODENO
   10 FORMAT(////' *** THE ELEMENT WHOSE NODE NUMBER IS',I4,
     1' COULD NOT BE FOUND ***'////)
   20 RETURN
      END
       SUBROUTINE LOAD(NAXINC)
C**********************************************************************
C      THIS SUBROUTINE CALCULATES THE FULL LOAD INCREMENT FOR LOAD
C     INCREMENT NUMINC AND THE HALF LOAD INCREMENT FOR THE FOLLOWING
C     INCREMENT
C**********************************************************************
C
       COMMON/BLOCK1/NEQNS,NBAND,NBLK,NUMINC,ICOUNT
       COMMON/BLOCK7/NLDTY,NINCRT(6),NLOADS(6),NODER(40,6),
      1NODER(40,6),VALUER(40,6)
       COMMON/BLCK11/RTOT(750),REOTOT(750),DTOT(750),R2(2,750),D2(2,750)
       DO 2 J=1,NEQNS
       DO 2 I=1,2
```

```
      R2(I,J)=0.0
    2 CONTINUE
      IF(NUMINC.GT.1) GO TO 4
      MAXINC=0
C     CALCULATION OF TOTAL NUMBER OF LOAD INCREMENTS.
      DO 3 I=1,NLDTY
      MAXINC=MAXINC+NINCRT(I)
    3 CONTINUE
C     DETERMINATION OF LOAD INCREMENT TYPE
    4 I=0
      IC=0
    5 I=I+1
      IC=IC+NINCRT(I)
      IF(NUMINC.GT.IC) GO TO 5
C     FORMATION OF FULL LOAD INCREMENT VECTOR TO BE STORED IN FIRST
C     COLUMN OF R2(2,NEQNS)
      NL=NLOADS(I)
      DO 8 J=1,NL
      II=NODER(J,I)
      JJ=1
      NBLK=1
      CALL LOCATE(II,JJ,NROW,NCOL)
      K=NROW+KODER(J,I)-1
      R2(1,K)=VALUER(J,I)
    8 CONTINUE
C     FORMATION OF HALF LOAD INCREMENT OF NEXT LOAD ITERATION AND
C     STORAGE IN SECOND COLUMN OF R2(2,NEQNS)
      IF(NUMINC.EQ.1) GO TO 11
      IF(NUMINC.LT.MAXINC) GO TO 10
C     FOLLOWING HALF LOAD INCREMENT IS SET TO ZERO IF CURRENT LOAD
C     INCREMENT IS THE LAST INCREMENT.
      DO 9 J=1,NEQNS
      R2(2,J)=0.0
    9 CONTINUE
      RETURN
   10 IF((NUMINC+1).GT.IC) GO TO 15
   11 DO 12 J=1,NEQNS
      R2(2,J)=.5*R2(1,J)
   12 CONTINUE
      RETURN
   15 J=I+1
      NL=NLOADS(J)
      DO 18 K=1,NL
      II=NODER(K,J)
      JJ=1
      NBLK=1
      CALL LOCATE(II,JJ,NROW,NCOL)
      L=NROW+KODER(K,J)-1
      R2(2,L)=VALUER(K,J)*.5
   18 CONTINUE
      RETURN
      END
```

APPENDIX E

SUBROUTINE LOGIC NOTES

APPENDIX E

SUBROUTINE LOGIC NOTES

The purpose of this appendix is to clarify any subroutine logic development that might not be sufficiently illustrated by the numerous comment cards inserted throughout the program. The comment card heading in each subroutine states the routine's principal function. Those routines warranting comment are listed in alphabetical order.

Subroutine DOWEL

To detect the failure of a dowel mechanism across a concrete crack, the dowel displacement of the adjacent crack surfaces at the concrete element's centroid is calculated and compared to the critical dowel displacement value DF.

Subroutine FORCE

When a bi-linear stress-strain curve is assumed for the conventional reinforcement, considerable difficulty is experienced in modelling the reinforcement's behaviour beyond yielding. Inevitably, an excessive number of modified Newton-Rapson iterations is required to produce convergence, especially when the second linear segment of the stress-strain curve is almost perfectly plastic. To eliminate this particular cause of time consuming and costly iteration, a yielding conventional reinforcement bar is modelled in the total stress condition as a bar of zero stiffness with externally applied nodal forces at each bar node representing the influence of the bar of the remainder of the structure. In the incremental stress condition, the bar's stiffness corresponds to the slope of the strain-hardening portion of the stress-strain curve. For the range of strain-hardening usually encountered, this approach simulates reinforcement yielding accurately.

Subroutine HORIZ

Before the stiffness terms of a horizontal rectangular element are added into the in-core total stiffness matrix, the component (3x3) element stiffness blocks must be transformed as the local element axes, as shown in Figure A-1, do not correspond to the global axis orientation. The sign of four stiffness terms is changed as a result of the transformation process.

Subroutine NDCONC

The formulations used in this subroutine are given in detail in Sections 3.4.2.1 and 3.4.2.6. In deriving the stiffness of a cracked concrete element, the constitutive matrix is formulated for axes in the crack and orthogonal to crack directions. Consequently, transformation of the constitutive matrix must be undertaken.

Subroutine SHRMOD

The shear rigidity of a cracked concrete element in a direction parallel to the crack direction is the sum of the aggregate interlock and dowel stiffnesses. The empirical derivations of the two contributing stiffnesses are given in Sections 3.4.2.4 and 3.4.2.5. In formulation of the dowel stiffness, the magnitude is not permitted to exceed twenty percent of the aggregate interlock value.

Subroutine WARP

The warping resistance of "thick" diaphragms is greatly overestimated by classical thin plate theory, as detailed in Section 3.4.2.8. However, the warping stiffness formulation used in this subroutine is the classical form given in 3.4.2.7, with real behaviour reflected by the insertion of an equivalent diaphragm thickness.

APPENDIX F

INPUT DATA FILE EXAMPLE

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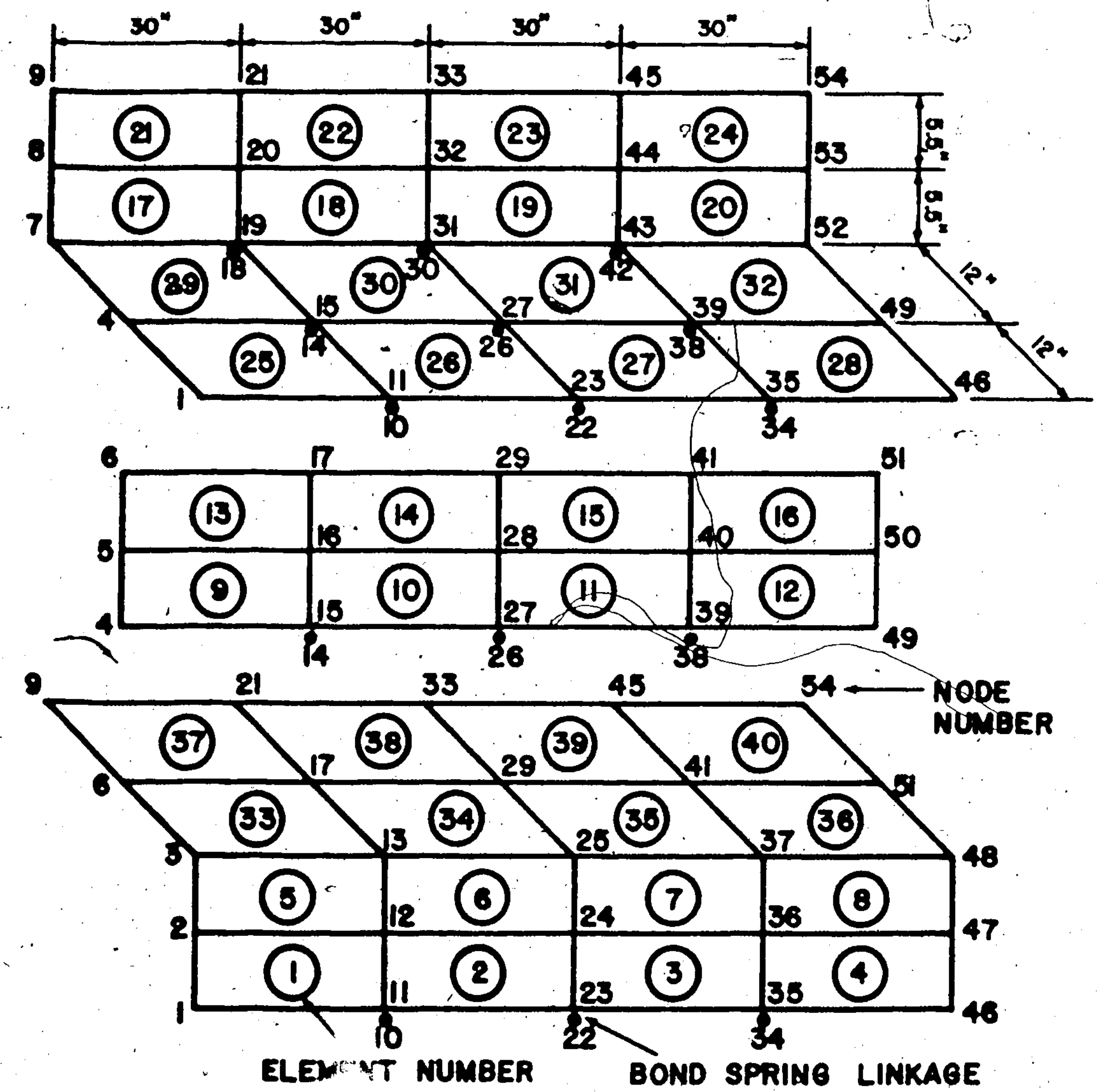
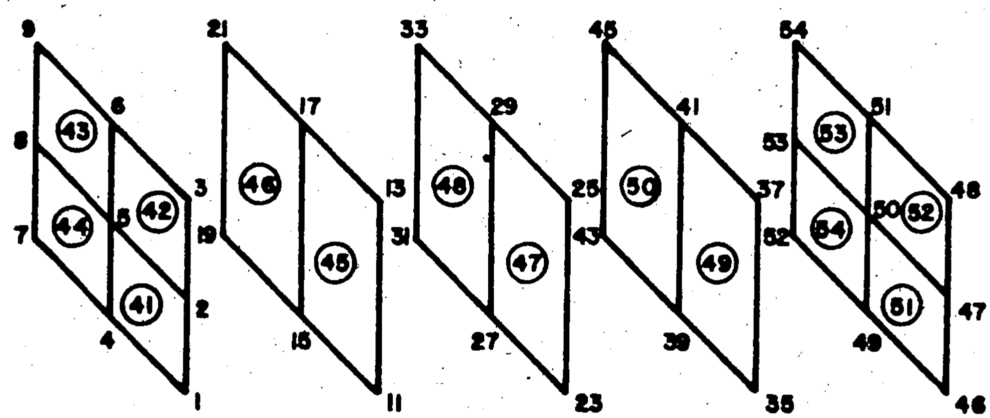
 1, 120.0, 11.0,  0.0,
3.0E06, 2.95E07, 26.5E06, 1.95E06,  2.95E07,
-5000.0, 400.0, -5000.0, 75700.0, 277083.0, 75700.0,
 .00185, .00985, -.003, .045, .045, .045, .0012, .0005,
.2, .18, .19, .2,
```

```
8.0, .001, .001, 20.0, 50.0, 8, 4.0, 1.2,
15, 1, 3, 4,
 2,
1, 6,
 1, 1, 16465.0,
 4, 1, 32930.0,
 7, 1, 16465.0,
46, 1, -16465.0,
49, 1, -32930.0,
52, 1, -16465.0,
10, 3,
24, 2, -5000.0,
28, 2, -5000.0,
32, 2, -5000.0,
6,
 5, 1, 0.0,
 5, 2, 0.0,
 5, 3, 0.0,
27, 3, 0.0,
29, 3, 0.0,
50, 2, 0.0,
4,
26, 27, 34, 35,
2,
2, 6,
10,
1, 2, 3, 13, 14, 15, 25, 26, 27, 37,
1,
28,
1, 1, 1, 1, 1, 1, 1,  0,
```



(a) Exploded view of concrete web and flange elements



(b) Diaphragm elements

FIG. F-1 BEAM MESH FOR INPUT DATA FILE EXAMPLE

APPENDIX G

OUTPUT EXAMPLE

****

****

* ECHO CHECK OF DATA INPUT *

TOTAL NUMBER OF EQUATIONS = 210
BAND WIDTH = 62
NUMBER OF BYTES IN EACH RECORD = 15376
NUMBER OF FINITE ELEMENTS = 40
NUMBER OF FINITE ELEMENTS CHECKED FOR CRACKING = 40

ELEMENT NUMBER INDICATOR	INCLINATION OF ELEMENT INDICATOR	ELEMENT SIZE INDICATOR	ELEMENT TYPE	ELEMENT THICKNESS	ELEMENT NODE NUMBERS				DOWEL INDICATOR	STEEL MESH INDICATOR	MESH SIZE INDICATOR	DOWEL WIDTH
1	1	1	1	2.0000	1	2	12	11	1	1	1	6.000
2	1	1	2	2.0000	11	12	24	23	1	1	1	6.000
3	1	1	1	2.0000	23	24	36	35	1	1	1	6.000
4	1	1	2	2.0000	35	36	47	46	1	1	1	6.000
5	1	1	2	2.0000	2	3	13	12	0	0	1	0.0
6	1	1	1	2.0000	12	13	25	24	0	1	1	0.0
7	1	1	2	2.0000	24	25	37	36	0	1	1	0.0
8	1	1	1	2.0000	36	37	48	47	0	1	1	0.0
9	1	1	1	2.0000	4	5	16	15	1	1	1	12.000
10	1	1	2	2.0000	15	16	28	27	1	1	1	12.000
11	1	1	1	2.0000	27	28	40	39	1	1	1	12.000
12	1	1	2	2.0000	39	40	50	49	1	1	1	12.000
13	1	1	2	2.0000	5	6	17	16	0	1	1	0.0
14	1	1	1	2.0000	16	17	29	28	0	1	1	0.0
15	1	1	2	2.0000	28	29	41	40	0	1	1	0.0
16	1	1	1	2.0000	40	41	51	50	0	1	1	0.0
17	1	1	1	2.0000	7	8	20	19	1	1	1	6.000
18	1	1	2	2.0000	19	20	32	31	1	1	1	6.000

19	1	1	1	2.0000	31	32	44	43	1	1	1	6.000
20	1	1	2	2.0000	43	44	53	52	1	1	1	6.000
[illegible]	1	1	2	2.0000	8	9	21	20	0	1	1	0.0
[illegible]	1	1	1	2.0000	20	21	33	32	0	1	1	0.0
[illegible]	1	1	2	2.0000	32	33	45	44	0	1	1	0.0
[illegible]	1	1	1	2.0000	44	45	54	53	0	1	1	0.0
[illegible]	0	2	1	2.0000	4	1	11	15	0	1	1	0.0
[illegible]	0	2	2	2.0000	15	11	23	27	0	1	1	0.0
[illegible]	0	2	1	2.0000	27	23	35	39	0	1	1	0.0
[illegible]	0	2	2	2.0000	39	35	46	49	0	1	1	0.0
[illegible]	0	2	2	2.0000	7	4	15	19	0	1	1	0.0
[illegible]	0	2	1	2.0000	19	15	27	31	0	1	1	0.0
31	0	2	2	2.0000	31	27	39	43	0	1	1	0.0
32	0	2	1	2.0000	43	39	49	52	0	1	1	0.0
33	0	2	1	2.0000	6	3	13	17	0	1	1	0.0
34	0	2	2	2.0000	17	13	25	29	0	1	1	0.0
35	0	2	1	2.0000	29	25	37	41	0	1	1	0.0
36	0	2	2	2.0000	41	37	48	51	0	1	1	0.0
37	0	2	2	2.0000	9	6	17	21	0	1	1	0.0
38	0	2	1	2.0000	21	17	29	33	0	1	1	0.0
39	0	2	2	2.0000	33	29	41	45	0	1	1	0.0
40	0	2	1	2.0000	45	41	51	54	0	1	1	0.0

DATA FOR INITIAL CONCRETE SHRINKAGE STRESSES IS AS BELOW
NUMBER OF ELEMENTS IN EACH FLANGE= 8
LONGITUDIONAL SHRINKAGE STRESS IN TOP FLANGE= 0.1000E+03
TRANSVERSE SHRINKAGE STRESS IN TOP FLANGE= 0.0
LONGITUDIONAL SHRINKAGE STRESS IN BOTTOM FLANGE= 0.2000E+03
TRANSVERSE SHRINKAGE STRESS IN BOTTOM FLANGE= 0.0
LONGITUDIONAL SHRINKAGE STRESS IN WEBS= 0.0
TRANSVERSE SHRINKAGE STRESS IN WEBS= 0.0
LONGITUDIONAL SHRINKAGE STRAIN IN TOP FLANGE= 0.3300E-04
TRANSVERSE SHRINKAGE STRAIN IN TOP FLANGE= 0.0
LONGITUDIONAL SHRINKAGE STRAIN IN BOTTOM FLANGE= 0.6600E-04
TRANSVERSE SHRINKAGE STRAIN IN BOTTOM FLANGE= 0.0
LONGITUDIONAL SHRINKAGE STRAIN IN WEBS= 0.0
TRANSVERSE SHRINKAGE STRAIN IN WEBS= 0.0

FLANGE ELEMENT NUMBERS ARE AS BELOW ; CONSECUTIVE ROWS OF TOP AND BOTTOM FLANGE ELEMENTS.

NLTP(1 , 8)= 33 34 35 36 37 38 39 40
NLBH(1 , 8)= 25 26 27 28 29 30 31 32

NUMBER OF ELEMENTS REINFORCED WITH A STEEL MESH= 40

ELEMENT NUMBER	NUMBER OF DIRECTIONS OF REINFORCEMENT	DIRN. NUMBER	STEEL PERCENTAGE	STEEL ANGLE
1	1	1	2.5000	90.0
2	1	1	2.5000	90.0
3	1	1	2.5000	90.0
4	1	1	2.5000	90.0
5	1	1	2.5000	90.0
6	1	1	2.5000	90.0
7	1	1	2.5000	90.0
8	1	1	2.5000	90.0
9	1	1	2.5000	90.0

10	1	1	2.5000	90.0
11	1	1	2.5000	90.0
12	1	1	2.5000	90.0
13	1	1	2.5000	90.0
14	1	1	2.5000	90.0
15	1	1	2.5000	90.0
16	1	1	2.5000	90.0
17	1	1	2.5000	90.0
18	1	1	2.5000	90.0
19	1	1	2.5000	90.0
20	1	1	2.5000	90.0
21	1	1	2.5000	90.0
22	1	1	2.5000	90.0
23	1	1	2.5000	90.0
24	1	1	2.5000	90.0
25	1	1	2.5000	90.0
26	1	1	2.5000	90.0
27	1	1	2.5000	90.0
28	1	1	2.5000	90.0
29	1	1	2.5000	90.0
30	1	1	2.5000	90.0
31	1	1	2.5000	90.0
32	1	1	2.5000	90.0
33	1	1	2.5000	90.0
34	1	1	2.5000	90.0
35	1	1	2.5000	90.0
36	1	1	2.5000	90.0
37	1	1	2.5000	90.0
38	1	1	2.5000	90.0
39	1	1	2.5000	90.0
40	1	1	2.5000	90.0

DIAPHRAGM INPUT DATA IS AS BELOW

TOTAL NUMBER OF DIAPHRAGM ELEMENTS IN COMPLETE BEAM =16

DATA FOR ACTUAL AND PSEUDO DIAPHRAGMS IS AS BELOW

ELEMENT NO.	ELEMENT TYPE	ELEMENT THICKNESS	ELEMENT NODE NUMBERS				REFERENCE NO.	SHEAR MODULUS
41	3	25.50	4	1	2	5	4	0.3000E+07
42	3	25.50	5	2	3	6	4	0.3000E+07
43	3	25.50	8	5	6	9	4	0.3000E+07
44	3	25.50	7	4	5	8	4	0.3000E+07
45	3	25.50	49	46	47	50	4	0.3000E+07
46	3	25.50	50	47	48	51	4	0.3000E+07
47	3	25.50	53	50	51	54	4	0.3000E+07
48	3	25.50	52	49	50	53	4	0.3000E+07
49	4	1.00	15	11	13	17	5	0.5700E+06
50	4	1.00	19	15	17	21	5	0.5700E+06
51	4	1.00	27	23	25	29	5	0.5700E+06
52	4	1.00	31	27	29	33	5	0.5700E+06
53	4	1.00	39	35	37	41	5	0.5700E+06
54	4	1.00	43	39	41	45	5	0.5700E+06
55	5	8.50	7	1	3	9	6	0.3000E+07
56	5	8.50	52	46	48	54	6	0.3000E+07

NUMBER OF SINGLE REINFORCING BARS AND BOND SLIP LINKAGES = 45

ELEMENT NUMBER	FIRST NODE	SECOND NODE	REO. TYPE CODE	DIRECTION INDICATOR	AREA	INITIAL PRESTRESS	INITIAL PRESTRAIN
1	34	46	-1	-1	0.1100		
2	38	49	-1	-1	0.2200		
3	42	52	-1	-1	0.1100		
4	22	34	-1	-1	0.1100		
5	26	38	-1	-1	0.2200		
6	30	42	-1	-1	0.1100		
7	10	22	-1	-1	0.1100		
8	14	26	-1	-1	0.2200		
9	18	30	-1	-1	0.1100		
10	1	10	-1	-1	0.1100		
11	4	14	-1	-1	0.2200		
12	7	18	-1	-1	0.1100		
13	37	48	-1	-1	0.1100		
14	41	51	-1	-1	0.2200		
15	45	54	-1	-1	0.1100		
16	25	37	-1	-1	0.1100		
17	29	41	-1	-1	0.2200		
18	33	45	-1	-1	0.1100		
19	13	25	-1	-1	0.1100		
20	17	29	-1	-1	0.2200		
21	21	33	-1	-1	0.1100		
22	3	13	-1	-1	0.1100		
23	6	17	-1	-1	0.2200		
24	9	21	-1	-1	0.1100		
25	34	46	0	-1	0.0990	0.16667E+06	0.66670E-02
26	38	49	0	-1	0.1980	0.16667E+06	0.66670E-02
27	42	52	0	-1	0.0990	0.16667E+06	0.66670E-02
28	22	34	0	-1	0.0990	0.16667E+06	0.66670E-02
29	26	38	0	-1	0.1980	0.16667E+06	0.66670E-02
30	30	42	0	-1	0.0990	0.16667E+06	0.66670E-02
31	10	22	0	-1	0.0990	0.16667E+06	0.66670E-02
32	14	26	0	-1	0.1980	0.16667E+06	0.66670E-02
33	18	30	0	-1	0.0990	0.16667E+06	0.66670E-02
34	1	10	0	-1	0.0990	0.16667E+06	0.66670E-02
35	4	14	0	-1	0.1980	0.16667E+06	0.66670E-02
36	7	18	0	-1	0.0990	0.16667E+06	0.66670E-02
37	34	35	1		125.4000		
38	38	39	1		215.4000		
39	42	43	1		125.4000		
40	22	23	1		125.4000		
41	26	27	1		215.4000		
42	30	11	1		125.4000		
43	10	11	1		125.4000		
44	14	15	1		215.4000		
45	18	19	1		125.4000		

NUMBER OF FINITE ELEMENT NODES = 54

NODE NUMBER	NODE TYPE INDICATOR	X COORDINATE	Y COORDINATE	Z COORDINATE
1	1	0.0	0.0	24.00

2	2	0.0	5.50	24.00
3	1	0.0	11.00	24.00
4	1	0.0	0.0	12.00
5	2	0.0	5.50	12.00
6	1	0.0	11.00	12.00
7	1	0.0	0.0	0.0
8	2	0.0	5.50	0.0
9	1	0.0	11.00	0.0
10	-1	30.00	0.0	24.00
11	1	30.00	0.0	24.00
12	0	30.00	5.50	24.00
13	1	30.00	11.00	24.00
14	-1	30.00	0.0	12.00
15	1	30.00	0.0	12.00
16	0	30.00	5.50	12.00
17	1	30.00	11.00	12.00
18	-1	30.00	0.0	0.0
19	1	30.00	0.0	0.0
20	0	30.00	5.50	0.0
21	1	30.00	11.00	0.0
22	-1	60.00	0.0	24.00
23	1	60.00	0.0	24.00
24	0	60.00	5.50	24.00
25	1	60.00	11.00	24.00
26	-1	60.00	0.0	12.00
27	1	60.00	0.0	12.00
28	0	60.00	5.50	12.00
29	1	60.00	11.00	12.00
30	-1	60.00	0.0	0.0
31	1	60.00	0.0	0.0
32	0	60.00	5.50	0.0
33	1	60.00	11.00	0.0
34	-1	90.00	0.0	24.00
35	1	90.00	0.0	24.00
36	0	90.00	5.50	24.00
37	1	90.00	11.00	24.00
38	-1	90.00	0.0	12.00
39	1	90.00	0.0	12.00
40	0	90.00	5.50	12.00
41	1	90.00	11.00	12.00
42	-1	90.00	0.0	0.0
43	1	90.00	0.0	0.0
44	0	90.00	5.50	0.0
45	1	90.00	11.00	0.0
46	1	120.00	0.0	24.00
47	2	120.00	5.50	24.00
48	1	120.00	11.00	24.00
49	1	120.00	0.0	12.00
50	2	120.00	5.50	12.00
51	1	120.00	11.00	12.00
52	1	120.00	0.0	0.0
53	2	120.00	5.50	0.0
54	1	120.00	11.00	0.0

THE STRENGTH AND DEVIATION PARAMETERS ARE AS BELOW

INITIAL YOUNG|S MODULUS FOR CONCRETE = 0.30000E+07
INITIAL YOUNG|S MODULUS FOR CONVENTIONAL REINFORCEMENT = 0.29500E+08

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INITIAL YOUNG|S MODULUS FOR PRESTRESSING STEEL =    0.26500E+08
INITIAL BOND SLIP LINKAGE STIFFNESS =    0.19500E+07 LBS/CU. INCH
INITIAL STEEL MESH MODULUS =    0.29500E+08

COMPRESSIVE CONCRETE STRENGTH =   -0.50000E+04 LBS./SQ.INCH
TENSILE CONCRETE STRENGTH =    0.40000E+03 LBS./SQ.INCH
ULTIMATE AGGREGATE INTERLOCK STRENGTH =  -0.50000E+04 LBS./SQ.IN.
ULTIMATE TENSILE STRENGTH OF CONVENTIONAL REINFORCEMENT =    0.75700E+05 LBS./SQ.INCH
ULTIMATE TENSILE STRENGTH OF PRESTRESSING STEEL =    0.27708E+06 LBS./SQ.INCH
ULTIMATE TENSILE STRENGTH OF STEEL MESH BARS =    0.75700E+05 LBS/SQ.INCH

YIELD STRAIN FOR CONVENTIONAL REINFORCEMENT =   0.18500E-02
YIELD STRAIN FOR PRESTRESS REINFORCEMENT=   0.98500E-02
MAXIMUM CONCRETE COMPRESSIVE STRAIN =-.003000
MAXIMUM CONVENTIONAL REINFORCEMENT TENSILE STRAIN =0.045000
MAXIMUM PRESTRESSING STEEL TENSILE STRAIN =0.045000
MAXIMUM STEEL MESH BAR TENSILE STRAIN =0.045000
MAXIMUM BOND LINKAGE SLIP =0.001200 INCHES
DOWEL FAILURE DISPLACEMENT =0.000500 INCHES

POISSON RATIO FOR CONCRETE IN BIAXIAL COMPRESSION =.20000
POISSON RATIO FOR CONCRETE IN BIAXIAL TENSION =.18000
POISSON RATIO FOR CONCRETE IN TENSION-COMPRESSION =.19000
UNIAXIAL POISSON RATIO FOR CONCRETE =.20000
MAXIMUM ALLOWABLE PERCENTAGE DEVIATION OF CONCRETE =   8.000
MAXIMUM ALLOWABLE PERCENTAGE DEVIATION OF MAIN DIAGONAL
TERMS OF CONCRETE CONSTITUTIVE MATRIX = 0.00100
MAXIMUM ALLOWABLE PEPCENTAGE DEVIATION OF REINFORCEMENT STIFFNESS
BEFORE CHANGE MUST BE MADE =  0.00100
MAXIMUM ALLOWABLE PERCENTAGE DEVIATION OF PRESTRESS BARS =  20.000
MAXIMUM ALLOWABLE PERCENTAGE DEVIATION FOR BOND SLIP LINKAGE =  50.000
CRITICAL NUMBER OF MATERIAL DEVIATIONS TO INVOKE ITERATIVE METHOD =  8
AVERAGE CRACK SPACING = 4.000 INCHES
RELAXATION FACTOR = 1.200

MAXIMUM NUMBER OF ITERATIONS BEFORE EXECUTION IS TERMINATED= 15
CONCRETE DEVIATION WEIGHTING = 1
CONVENTIONAL REINFORCEMENT DEVIATION WEIGHTING = 3
PRESTRESS STRAND DEVIATION WEIGHTING = 4


LOADING INFORMATION FOLLOWS

NUMBER OF DIFFERENT LOADING TYPES TO BE SUPERIMPOSED ON MODEL =  2
 LOAD TYPE NUMBER      NO. OF INCREMENTS    NO. OF NODAL LOADS
        1                     1                     6
 NODE OF APPLICATION    LOAD DIRECTION CODE    LOAD MAGNITUDE
        1                     1                 0.164650E+05
        4                     1                 0.329300E+05
        7                     1                 0.164650E+05
       46                     1                -0.164650E+05
       49                     1                -0.329300E+05
       52                     1                -0.164650E+05
 LOAD TYPE NUMBER      NO. OF INCREMENTS    NO. OF NODAL LOADS
        2                    10                     3
 NODE OF APPLICATION    LOAD DIRECTION CODE    LOAD MAGNITUDE
       24                     2                -0.500000E+04
       28                     2                -0.500000E+04
       32                     2                -0.500000E+04
```

```
IMPOSED BOUNDARY CONDITIONS ARE AS BELOW

NUMBER OF IMPOSED BOUNDARY CONDITIONS =    6

  NODE OF APPLICATION     DIRECTION OF     IMPOSED
 OF IMPOSED BOUNDARY    APPLICATION CODE    VALUE
     CONDITIONS
          5                    1            0.0
          5                    2            0.0
          5                    3            0.0
         27                    3            0.0
         29                    3            0.0
         50                    2            0.0


SCREEN PRINTOUT DATA IS AS BELOW

THE ELEMENT NUMBERS OF THE 4 CONCRETE ELEMENTS WHOSE (STRESS,STRAIN) STATE
WILL BE DISPLAYED, ARE BELOW
 26  27  34  35

THE 2 STEEL MESH ELEMENT NUMBERS WHOSE (STRESS,STRAIN) STATES WILL BE DISPLAYED, ARE BELOW
  2   6

THE 10 REINFORCEMENT ELEMENTS WHOSE (STRESS,STRAIN) STATES WILL BE DISPLAYED, ARE BELOW
  1   2   3  13  14  15  25  26  27  37

THE 1 NODES WHOSE DISPLACEMENTS WILL BE DISPLAYED ARE BELOW
 28


PRINTOUT FORMAT CONTROL VARIABLES ARE AS BELOW

DEFLECTIONS CONTROL VARIABLE = 1
CENTROIDAL CONCRETE STRESSES CONTROL VARIABLE = 1
PRINCIPAL CONCRETE STRESSES CONTROL VARIABLE= 1
STEEL MESH STRESSES CONTROL VARIABLE = 1
REINFORCEMENT ELEMENT STRESSES CONTROL VARIABLE = 1
LOAD INCREMENT CONTROL VARIABLE = 1
TOTAL LOAD CONTROL VARIABLE = 1
STIFFNESS ASSEMBLAGE CONTROL VARIABLE = 0


                                                *** END OF INPUT ECHO CHECK ***
*****************************************************************************************************************


AFTER DATA IS READ IN, CPU TIME =     1616  PROGRAMME COST =    0.66

NBYTES = 15376 LEN = 15376 NUMREC = 1
```

```
STRUCTURE STIFFNESS MATRIX HAS BEEN FULLY ASSEMBLED AND WRITTEN ON -FILE1


AFTER TOTAL STIFFNESS MATRIX HAS BEEN ASSEMBLED AND CERTAIN MATRICES PRESET TO ZERO,
CPU TIME =    3470  PROGRAMME COST =   1.17



********************************
*  LOAD INCREMENT NUMBER  1.  *
********************************


****** OUTPUT FROM SUBROUTINE SOLVE ******

CPU TIME AT INITIAL STAGE IN SUBROUTINE SOLVE =     3479 PROGRAM COST =   1.18
CPU TIME AT COMPLETION OF BLOCK MANIPULATION OF BLOCK NUMBER 1 =     3504
CPU TIME AT COMPLETION OF GAUSSIAN ELIMINATION OF BLOCK NUMBER 1 =     3981
CPU TIME AFTER REDUCED EQUATIONS OF BLOCK NUMBER 1 HAVE BEEN WRITTEN OUT ON DISC =    4004
CPU TIME AT BEGINNING OF BACKSUBSTITUTION PROCESS =     5263
CPU TIME AT END OF SOLUTION PROCESS =     5477 PROGRAM COST =   1.73

BEFORE ROUTINE STRESS IS CALLED IN FIRST LOAD INCREMENT ,
CPU TIME =    5480  PROGRAMME COST =   1.74

AFTER ROUTINE STRESS HAS BEEN CALLED AND BEFORE ROUTINE KUTTA IS CALLED IN THE FIRST LOAD INCREMENT,
CPU TIME =    5536  PROGRAMME COST =   1.75

AFTER ROUTINE KUTTA HAS BEEN CALLED IN FIRST LOAD INCREMENT,
CPU TIME =    7227  PROGRAMME COST =   2.22

BEFORE BEAM IS CHECKED FOR FURTHER CRACKING,
CPU TIME =    9209  PROGRAMME COST =   2.78



**************************************************************
*  LOAD INCREMENT NUMBER  1 STRESS-DEFORMATION PRINTOUT  *
**************************************************************


THE NUMBER OF CONCRETE ELEMENTS THAT CRACKED IN THIS LOAD INCREMENT =    0
```

THE NODAL DISPLACEMENTS ARE AS FOLLOWS

NODE	X	Y	Z	THETAY	THETAZ
1	0.99938E-02	0.36988E-04	0.52881E-04	0.34184E-03	0.18010E-02
2	-0.39375E-03	0.43202E-04	0.65843E-05		0.17471E-02
3	-0.98948E-02	0.46680E-04	-0.24967E-04	-0.76227E-04	0.18175E-02
4	0.10573E-01	-0.69680E-06	-0.28306E-05	-0.60900E-07	0.18023E-02
5	0.0	0.0	0.0		0.17733E-02
6	-0.96088E-02	0.35095E-05	0.28372E-05	0.56352E-07	0.18213E-02
7	0.99945E-02	0.49371E-04	-0.58537E-04	-0.34193E-03	0.18014E-02
8	-0.39427E-03	0.55577E-04	-0.65797E-05		0.17474E-02
9	-0.98959E-02	0.59086E-04	0.30637E-04	0.76335E-04	0.18177E-02
10	0.10539E-02				
11	0.10534E-02	0.40308E-01	0.54888E-03	0.24908E-05	0.81834E-03
12	-0.36414E-02	0.40456E-01			0.87636E-03

13	-0.85336E-02	0.40489E-01	-0.47548E-04	0.22171E-05	0.89192E-03
14	0.12249E-02				
15	0.12241E-02	0.40244E-01	-0.13830E-05	0.26399E-08	0.82518E-03
16	-0.35383E-02	0.40411E-01			0.87417E-03
17	-0.85261E-02	0.40430E-01	0.15497E-05	0.39597E-08	0.91443E-03
18	0.10536E-02				
19	0.10531E-02	0.40322E-01	-0.55124E-03	-0.26189E-05	0.81827E-03
20	-0.36413E-02	0.40470E-01			0.87605E-03
21	-0.85335E-02	0.40500E-01	0.50210E-04	-0.20950E-05	0.89201E-03
22	-0.73006E-02				
23	-0.73007E-02	0.53396E-01	0.57705E-03	-0.79025E-09	-0.14912E-06
24	-0.73005E-02	0.53583E-01			-0.13679E-07
25	-0.73004E-02	0.53632E-01	-0.55690E-04	0.43934E-07	-0.10656E-06
26	-0.73018E-02				
27	-0.73018E-02	0.53327E-01	0.0	0.12085E-07	-0.24671E-06
28	-0.73009E-02	0.53529E-01			-0.99830E-07
29	-0.73007E-02	0.53569E-01	0.0	-0.19418E-09	-0.24052E-06
30	-0.73013E-02				
31	-0.73014E-02	0.53403E-01	-0.57727E-03	0.58092E-07	-0.25796E-06
32	-0.73008E-02	0.53590E-01			-0.70402E-07
33	-0.73004E-02	0.53639E-01	0.55906E-04	-0.63056E-07	-0.27254E-06
34	-0.15652E-01				
35	-0.15651E-01	0.40314E-01	0.54813E-03	-0.24564E-05	-0.81791E-03
36	-0.10958E-01	0.40462E-01			-0.87564E-03
37	-0.60674E-02	0.40492E-01	-0.48373E-04	-0.21764E-05	-0.89175E-03
38	-0.15824E-01				
39	-0.15824E-01	0.40246E-01	-0.16613E-05	0.12005E-07	-0.82461E-03
40	-0.11070E-01	0.40412E-01			-0.87361E-03
41	-0.60742E-02	0.40430E-01	0.10850E-05	0.14522E-07	-0.91486E-03
42	-0.15652E-01				
43	-0.15651E-01	0.40324E-01	-0.55108E-03	0.25770E-05	-0.81783E-03
44	-0.10959E-01	0.40472E-01			-0.87535E-03
45	-0.60682E-02	0.40502E-01	0.50150E-04	0.21081E-05	-0.89168E-03
46	-0.24593E-01	0.38219E-04	0.52772E-04	-0.34225E-03	-0.18020E-02
47	-0.14202E-01	0.44403E-04	0.58753E-05		-0.17476E-02
48	-0.46988E-02	0.47884E-04	-0.26217E-04	0.76380E-04	-0.18183E-02
49	-0.25167E-01	-0.71870E-06	-0.30635E-05	0.59516E-07	-0.18029E-02
50	-0.14586E-01	0.0	-0.71066E-06		-0.17735E-02
51	-0.49823E-02	0.35534E-05	0.16418E-05	-0.31025E-07	-0.18218E-02
52	-0.24594E-01	0.48498E-04	-0.58892E-04	0.34238E-03	-0.18022E-02
53	-0.14203E-01	0.54676E-04	-0.72968E-05		-0.17477E-02
54	-0.46985E-02	0.58163E-04	0.29493E-04	-0.76436E-04	-0.18185E-02

CENTROIDAL CONCRETE STRESSES TSGCON(NEL,3) AND AGGREGATE INTERLOCK STRESS TSGAGG(NEL) ARE AS BELOW

ELEMENT NO.	SIGMAX	SIGMAY(Z)	SIGMAXY(XZ)	SIGMA-AGG
1	-0.59586E+03	-0.27457E+02	-0.33231E+02	0.0
2	-0.61559E+03	-0.21985E+02	0.12743E+02	0.0
3	-0.61537E+03	-0.22100E+02	-0.12694E+02	0.0
4	-0.59573E+03	-0.27369E+02	0.33153E+02	0.0
5	-0.94145E+02	-0.13564E+02	0.49406E+02	0.0
6	-0.12288E+03	-0.47805E+01	-0.86866E+01	0.0
7	-0.12280E+03	-0.45861E+01	0.87226E+01	0.0
8	-0.93808E+02	-0.14799E+02	-0.49397E+02	0.0
9	-0.63013E+03	-0.30825E+02	-0.62066E+02	0.0
10	-0.63075E+03	-0.23391E+02	0.56014E+01	0.0
11	-0.63058E+03	-0.23441E+02	-0.55446E+01	0.0
12	-0.62885E+03	-0.30892E+02	0.62752E+02	0.0
13	-0.12118E+03	-0.14744E+02	0.24660E+02	0.0
14	-0.12901E+03	-0.30465E+01	-0.20764E+02	0.0

15	-0.12896E+03	-0.29597E+01	0.20965E+02	0.0
16	-0.11978E+03	-0.14993E+02	-0.25182E+02	0.0
17	-0.59588E+03	-0.27410E+02	-0.33272E+02	0.0
18	-0.61563E+03	-0.22014E+02	0.12572E+02	0.0
19	-0.61534E+03	-0.22100E+02	-0.12574E+02	0.0
20	-0.59577E+03	-0.27319E+02	0.33188E+02	0.0
21	-0.94248E+02	-0.14766E+02	0.49351E+02	0.0
22	-0.12292E+03	-0.47364E+01	-0.88374E+01	0.0
23	-0.12287E+03	-0.47249E+01	0.88439E+01	0.0
24	-0.93772E+02	-0.14859E+02	-0.49361E+02	0.0
25	-0.60765E+03	-0.50152E+02	-0.28936E+02	0.0
26	-0.66161E+03	-0.37948E+02	-0.82854E+01	0.0
27	-0.66131E+03	-0.37903E+02	0.84904E+01	0.0
28	-0.60740E+03	-0.50250E+02	0.28780E+02	0.0
29	-0.60769E+03	-0.50128E+02	0.29046E+02	0.0
30	-0.66161E+03	-0.37861E+02	0.84423E+01	0.0
31	-0.66127E+03	-0.37808E+02	-0.85779E+01	0.0
32	-0.60744E+03	-0.50229E+02	-0.28845E+02	0.0
33	0.20897E+03	0.12772E+02	-0.16055E+02	0.0
34	0.22374E+03	0.56247E+01	-0.59046E+00	0.0
35	0.22382E+03	0.55886E+01	0.55573E+00	0.0
36	0.20967E+03	0.12796E+02	0.15859E+02	0.0
37	0.20901E+03	0.12744E+02	0.15983E+02	0.0
38	0.22373E+03	0.55304E+01	0.46257E+00	0.0
39	0.22378E+03	0.54986E+01	-0.43837E+00	0.0
40	0.20970E+03	0.12769E+02	-0.15776E+02	0.0

PRINCIPAL COMPRESSIVE AND TENSILE STRESSES AND STRAINS FOR EACH ELEMENT CENTROID ARE AS BELOW

ELEMENT NO.	PRINCIPAL COMPRESSIVE STRESS	PRINCIPAL COMPRESSIVE STRAIN	PRINCIPAL TENSILE STRESS	PRINCIPAL TENSILE STRAIN	ELEMENT CRACKED ?
1	-0.59780E+03	-0.19580E-03	-0.25521E+02	0.31346E-04	NO
2	-0.61586E+03	-0.20194E-03	-0.21711E+02	0.33820E-04	NO
3	-0.61564E+03	-0.20187E-03	-0.21828E+02	0.33767E-04	NO
4	-0.59765E+03	-0.19576E-03	-0.25442E+02	0.31363E-04	NO
5	-0.11761E+03	-0.39714E-04	0.98972E+01	0.10747E-04	NO
6	-0.12352E+03	-0.40796E-04	-0.41450E+01	0.68528E-05	NO
7	-0.12344E+03	-0.40783E-04	-0.39460E+01	0.69140E-05	NO
8	-0.11755E+03	-0.39639E-04	0.89475E+01	0.10428E-04	NO
9	-0.63649E+03	-0.20856E-03	-0.24464E+02	0.34278E-04	NO
10	-0.63080E+03	-0.20675E-03	-0.23340E+02	0.34273E-04	NO
11	-0.63063E+03	-0.20669E-03	-0.23390E+02	0.34245E-04	NO
12	-0.63537E+03	-0.20819E-03	-0.24378E+02	0.34232E-04	NO
13	-0.12661E+03	-0.41488E-04	-0.93080E+01	0.53381E-05	NO
14	-0.13234E+03	-0.44010E-04	0.28807E+00	0.84778E-05	NO
15	-0.13236E+03	-0.44023E-04	0.43718E+00	0.85283E-05	NO
16	-0.12552E+03	-0.41129E-04	-0.92556E+01	0.52829E-05	NO
17	-0.59782E+03	-0.19581E-03	-0.25469E+02	0.31365E-04	NO
18	-0.61590E+03	-0.20195E-03	-0.21748E+02	0.33810E-04	NO
19	-0.61561E+03	-0.20186E-03	-0.21834E+02	0.33763E-04	NO
20	-0.59770E+03	-0.19577E-03	-0.25388E+02	0.31384E-04	NO
21	-0.11787E+03	-0.39737E-04	0.88555E+01	0.10417E-04	NO
22	-0.12357E+03	-0.40819E-04	-0.40791E+01	0.68785E-05	NO
23	-0.12353E+03	-0.40804E-04	-0.40665E+01	0.68796E-05	NO
24	-0.11751E+03	-0.39619E-04	0.88773E+01	0.10401E-04	NO
25	-0.60915E+03	-0.19895E-03	-0.48654E+02	0.37690E-04	NO
26	-0.66172E+03	-0.21679E-03	-0.37838E+02	0.44833E-04	NO
27	-0.66143E+03	-0.21670E-03	-0.37787E+02	0.44830E-04	NO
28	-0.60889E+03	-0.19886E-03	-0.48767E+02	0.37635E-04	NO

29	-0.60919E+03	-0.19897E-03	-0.48619E+02	0.37704E-04	NO
30	-0.66173E+03	-0.21680E-03	-0.37747E+02	0.44864E-04	NO
31	-0.66139E+03	-0.21669E-03	-0.37690E+02	0.44860E-04	NO
32	-0.60893E+03	-0.19887E-03	-0.48740E+02	0.37646E-04	NO
33	0.11467E+02	-0.28392E-05	0.21027E+03	0.69115E-04	NO
34	0.56231E+01	-0.55505E-05	0.22375E+03	0.73912E-04	NO
35	0.55872E+01	-0.55671E-05	0.22382E+03	0.73939E-04	NO
36	0.11526E+02	-0.28532E-05	0.21094E+03	0.69333E-04	NO
37	0.11451E+02	-0.28460E-05	0.21030E+03	0.69125E-04	NO
38	0.55295E+01	-0.55805E-05	0.22373E+03	0.73910E-04	NO
39	0.54977E+01	-0.55943E-05	0.22378E+03	0.73930E-04	NO
40	0.11513E+02	-0.28627E-05	0.21095E+03	0.69337E-04	NO

STEEL MESH STRESSES ARE AS BELOW.

ELEMENT NO.	MESH DIRECTION				
1	1	STRESS=	0.90204E+03	STRAIN=	0.30578E-04
2	1	STRESS=	0.99449E+03	STRAIN=	0.33712E-04
3	1	STRESS=	0.99294E+03	STRAIN=	0.33659E-04
4	1	STRESS=	0.90265E+03	STRAIN=	0.30598E-04
6	1	STRESS=	0.19467E+03	STRAIN=	0.65991E-05
7	1	STRESS=	0.19643E+03	STRAIN=	0.66585E-05
8	1	STRESS=	0.30367E+02	STRAIN=	0.10294E-05
9	1	STRESS=	0.93677E+03	STRAIN=	0.31755E-04
10	1	STRESS=	0.10105E+04	STRAIN=	0.34253E-04
11	1	STRESS=	0.10096E+04	STRAIN=	0.34225E-04
12	1	STRESS=	0.93360E+03	STRAIN=	0.31647E-04
13	1	STRESS=	0.93460E+02	STRAIN=	0.31682E-05
14	1	STRESS=	0.21116E+03	STRAIN=	0.71581E-05
15	1	STRESS=	0.21193E+03	STRAIN=	0.71840E-05
16	1	STRESS=	0.88280E+02	STRAIN=	0.29925E-05
17	1	STRESS=	0.90253E+03	STRAIN=	0.30594E-04
18	1	STRESS=	0.99429E+03	STRAIN=	0.33705E-04
19	1	STRESS=	0.99288E+03	STRAIN=	0.33657E-04
20	1	STRESS=	0.90322E+03	STRAIN=	0.30617E-04
21	1	STRESS=	0.31507E+02	STRAIN=	0.10680E-05
22	1	STRESS=	0.19518E+03	STRAIN=	0.66161E-05
23	1	STRESS=	0.19520E+03	STRAIN=	0.66168E-05
24	1	STRESS=	0.29704E+02	STRAIN=	0.10069E-05
25	1	STRESS=	0.10954E+04	STRAIN=	0.37131E-04
26	1	STRESS=	0.13213E+04	STRAIN=	0.44791E-04
27	1	STRESS=	0.13212E+04	STRAIN=	0.44787E-04
28	1	STRESS=	0.10939E+04	STRAIN=	0.37082E-04
29	1	STRESS=	0.10957E+04	STRAIN=	0.37141E-04
30	1	STRESS=	0.13222E+04	STRAIN=	0.44821E-04
31	1	STRESS=	0.13221E+04	STRAIN=	0.44816E-04
32	1	STRESS=	0.10942E+04	STRAIN=	0.37091E-04
33	1	STRESS=	-0.67279E+02	STRAIN=	-0.22807E-05
34	1	STRESS=	-0.16372E+03	STRAIN=	-0.55498E-05
35	1	STRESS=	-0.16421E+03	STRAIN=	-0.55665E-05
36	1	STRESS=	-0.68295E+02	STRAIN=	-0.23151E-05
37	1	STRESS=	-0.67631E+02	STRAIN=	-0.22926E-05
38	1	STRESS=	-0.16461E+03	STRAIN=	-0.55800E-05
39	1	STRESS=	-0.16502E+03	STRAIN=	-0.55939E-05
40	1	STRESS=	-0.68601E+02	STRAIN=	-0.23255E-05

REINFORCEMENT ELEMENT STRESSES AND STRAINS ARE AS BELOW;

REINFORCEMENT NUMBER	REINFORCEMENT TYPE	STRESS	STRAIN
1	CONVENTIONAL	-0.87923E+04	-0.29804E-03
2	CONVENTIONAL	-0.91870E+04	-0.31142E-03
3	CONVENTIONAL	-0.87933E+04	-0.29808E-03
4	CONVENTIONAL	-0.82117E+04	-0.27836E-03
5	CONVENTIONAL	-0.83807E+04	-0.28409E-03
6	CONVENTIONAL	-0.82111E+04	-0.27834E-03
7	CONVENTIONAL	-0.82153E+04	-0.27848E-03
8	CONVENTIONAL	-0.83845E+04	-0.28422E-03
9	CONVENTIONAL	-0.82156E+04	-0.27850E-03
10	CONVENTIONAL	-0.87909E+04	-0.29800E-03
11	CONVENTIONAL	-0.91927E+04	-0.31162E-03
12	CONVENTIONAL	-0.87919E+04	-0.29803E-03
13	CONVENTIONAL	0.13458E+04	0.45620E-04
14	CONVENTIONAL	0.10738E+04	0.36398E-04
15	CONVENTIONAL	0.13468E+04	0.45655E-04
16	CONVENTIONAL	0.12125E+04	0.41102E-04
17	CONVENTIONAL	0.12061E+04	0.40883E-04
18	CONVENTIONAL	0.12117E+04	0.41074E-04
19	CONVENTIONAL	0.12127E+04	0.41107E-04
20	CONVENTIONAL	0.12049E+04	0.40846E-04
21	CONVENTIONAL	0.12126E+04	0.41104E-04
22	CONVENTIONAL	0.13385E+04	0.45373E-04
23	CONVENTIONAL	0.10647E+04	0.36092E-04
24	CONVENTIONAL	0.13396E+04	0.45412E-04
25	PRESTRESSED	0.15877E+06	0.63690E-02
26	PRESTRESSED	0.15841E+06	0.63556E-02
27	PRESTRESSED	0.15877E+06	0.63689E-02
28	PRESTRESSED	0.15929E+06	0.63886E-02
29	PRESTRESSED	0.15914E+06	0.63829E-02
30	PRESTRESSED	0.15929E+06	0.63887E-02
31	PRESTRESSED	0.15929E+06	0.63885E-02
32	PRESTRESSED	0.15913E+06	0.63828E-02
33	PRESTRESSED	0.15929E+06	0.63885E-02
34	PRESTRESSED	0.15877E+06	0.63690E-02
35	PRESTRESSED	0.15841E+06	0.63554E-02
36	PRESTRESSED	0.15877E+06	0.63690E-02
37	BONDED	0.78424E+00	0.40233E-06
38	BONDED	0.13647E+01	0.70035E-06
39	BONDED	0.81326E+00	0.41723E-06
40	BONDED	0.10170E+00	0.52154E-07
41	BONDED	0.11623E+00	0.59605E-07
42	BONDED	0.14528E+00	0.74506E-07
43	BONDED	0.90346E+00	0.46357E-06
44	BONDED	0.14797E+01	0.75949E-06
45	BONDED	0.90436E+00	0.46403E-06

THE LOAD INCREMENT VECTOR IS AS BELOW;
RINC(1 ,1) = 0.16465E+05
RINC(4 ,1) = 0.32930E+05
RINC(7 ,1) = 0.16465E+05
RINC(46 ,1) = -0.16465E+05
RINC(49 ,1) = -0.32930E+06
RINC(52 ,1) = -0.16465E+05

THE TOTAL LOAD VECTOR IS AS BELOW;
RTOTAL(1 ,1) = 0.16465E+05

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RTOTAL(  4 ,1 ) =   0.32930E+05
RTOTAL(  7 ,1 ) =   0.16465E+05
RTOTAL( 46 ,1 ) =  -0.16465E+05
RTOTAL( 49 ,1 ) =  -0.32930E+05
RTOTAL( 52 ,1 ) =  -0.16465E+05

TOTAL REINFORCEMENT FORCE RESTRAINT VECTOR;


******************************
* LOAD INCREMENT NUMBER  2  *
******************************

BEFORE BEAM IS CHECKED FOR FURTHER CRACKING,
CPU TIME =    12804  PROGRAMME COST =    3.90
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*  LOAD INCREMENT NUMBER  5  *
******************************

BEFORE BEAM IS CHECKED FOR FURTHER CRACKING,
CPU TIME =    42853  PROGRAMME COST =  12.66

******** CONCRETE ELEMENT NUMBER  1* HAS JUST CRACKED ********

******** CONCRETE ELEMENT NUMBER  15 HAS JUST CRACKED ********

CONVENTIONAL REINFORCEMENT ELEMENT NUMBER  4 HAS JUST YIELDED
 STRESS=   0.63238E+05  STRAIN=   0.21437E-02

CONVENTIONAL REINFORCEMENT ELEMENT NUMBER  5 HAS JUST YIELDED
 STRESS=   0.62074E+05  STRAIN=   0.21042E-02

CONVENTIONAL REINFORCEMENT ELEMENT NUMBER  6 HAS JUST YIELDED
 STRESS=   0.63226E+05  STRAIN=   0.21433E-02

CONVENTIONAL REINFORCEMENT ELEMENT NUMBER  7 HAS JUST YIELDED
 STRESS=   0.63315E+05  STRAIN=   0.21463E-02

CONVENTIONAL REINFORCEMENT ELEMENT NUMBER  8 HAS JUST YIELDED
 STRESS=   0.62213E+05  STRAIN=   0.21089E-02

CONVENTIONAL REINFORCEMENT ELEMENT NUMBER  9 HAS JUST YIELDED
 STRESS=   0.63362E+05  STRAIN=   0.21479E-02

BEFORE STIFFNESS ADJUSTMENTS HAVE BEEN MADE,
CPU TIME =    42875  PROGRAMME COST =  12.67

**** ITERATION CYCLE NO.  1 ****

AFTER STIFFNESS ADJUSTMENTS HAVE BEEN MADE,
CPU TIME =    44577  PROGRAMME COST =  13.15

BEFORE BEAM IS CHECKED FOR FURTHER CRACKING,
CPU TIME =    46540  PROGRAMME COST =  13.70
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******** CONCRETE ELEMENT NUMBER   6 HAS JUST CRACKED ********

******** CONCRETE ELEMENT NUMBER   7 HAS JUST CRACKED ********

******** CONCRETE ELEMENT NUMBER  22 HAS JUST CRACKED ********

******** CONCRETE ELEMENT NUMBER  23 HAS JUST CRACKED ********
BEFORE STIFFNESS ADJUSTMENTS HAVE BEEN MADE,
CPU TIME =   46560  PROGRAMME COST =  13.71

**** ITERATION CYCLE NO.  2 ****

AFTER STIFFNESS ADJUSTMENTS HAVE BEEN MADE,
CPU TIME =   48263  PROGRAMME COST =  14.19

BEFORE BEAM IS CHECKED FOR FURTHER CRACKING,
CPU TIME =   50234  PROGRAMME COST =  14.74

**** PRESTRESS ELEMENT NUMBER 28 IS YIELDING ****
STRESS=   0.25533E+06  STRAIN=   0.10013E-01

**** PRESTRESS ELEMENT NUMBER 29 IS YIELDING ****
STRESS=   0.25440E+06  STRAIN=   0.99778E-02

**** PRESTRESS ELEMENT NUMBER 30 IS YIELDING ****
STRESS=   0.25527E+06  STRAIN=   0.10010E-01

**** PRESTRESS ELEMENT NUMBER 31 IS YIELDING ****
STRESS=   0.25608E+06  STRAIN=   0.10041E-01

**** PRESTRESS ELEMENT NUMBER 32 IS YIELDING ****
STRESS=   0.25545E+06  STRAIN=   0.10017E-01

**** PRESTRESS ELEMENT NUMBER 33 IS YIELDING ****
STRESS=   0.25615E+06  STRAIN=   0.10044E-01


*********************************************************
*  LOAD INCREMENT NUMBER  5 STRESS-DEFORMATION PRINTOUT  *
*********************************************************

THE NUMBER OF CONCRETE ELEMENTS THAT CRACKED IN THIS LOAD INCREMENT =     6

THE NODAL DISPLACEMENTS ARE AS FOLLOWS
  NODE         X              Y              Z              THETAY            THETAZ
```

1	-0.63200E-01	-0.13257E-03	-0.74226E-04	-0.14483E-03	-0.12472E-01
2	-0.11541E-03	-0.13452E-03	-0.45251E-05		-0.12167E-01
3	0.66038E-01	-0.10204E-03	0.61611E-04	0.10686E-03	-0.12073E-01
4	-0.61701E-01	-0.49890E-04	0.95676E-04	-0.18705E-05	-0.12527E-01
5	-0.0	-0.0	-0.0		-0.12031E-01
6	0.66156E-01	-0.30815E-04	-0.95559E-04	-0.18245E-05	-0.12065E-01
7	-0.63153E-01	-0.54993E-03	0.26530E-03	0.14086E-03	-0.12472E-01
8	-0.71909E-04	-0.55176E-03	0.45525E-05		-0.12165E-01
9	0.66081E-01	-0.51781E-03	-0.25252E-03	-0.10973E-03	-0.12069E-01
10	-0.61958E-01				
11	-0.62022E-01	-0.37279E+00	0.17057E-03	0.29135E-05	-0.10934E-01
12	-0.27901E-02	-0.37213E+00			-0.12801E-01
13	0.62713E-01	-0.37058E+00	0.84525E-03	-0.42858E-04	-0.12574E-01
14	-0.61021E-01				
15	-0.61096E-01	-0.37201E+00	0.15114E-03	-0.23944E-05	-0.10651E-01
16	-0.32883E-02	-0.37101E+00			-0.12755E-01
17	0.60854E-01	-0.36916E+00	-0.33174E-04	-0.16105E-05	-0.12193E-01
18	-0.61905E-01				
19	-0.61969E-01	-0.37315E+00	0.12135E-03	-0.51241E-05	-0.10934E-01
20	-0.27371E-02	-0.37249E+00			-0.13796E-01
21	0.62768E-01	-0.37110E+00	-0.91114E-03	0.38030E-04	-0.12575E-01
22	0.39262E-01				
23	0.39262E-01	-0.56[illegible]+00	-0.37679E-03	-0.18159E-04	0.24726E-04
24	0.39092E-01	-0.5[illegible]E+00			0.30923E-04
25	0.38939E-01	-0.5[illegible]E+00	0.12548E-02	0.11662E-05	0.27478E-04
26	0.39488E-01				
27	0.39489E-01	-0.56538E+00	-0.0	0.94361E-05	0.38290E-04
28	0.39197E-01	-0.56813E+00			0.46463E-04
29	0.38941E-01	-0.56456E+00	-0.0	0.37551E-06	0.34805E-04
30	0.39396E-01				
31	0.39396E-01	-0.56708E+00	0.37281E-03	0.70815E-05	0.42645E-04
32	0.39166E-01	-0.56934E+00			0.48231E-04
33	0.38922E-01	-0.56636E+00	-0.12530E-02	0.18674E-05	0.43086E-04
34	0.13964E+00				
35	0.13970E+00	-0.37238E+00	-0.10686E-03	-0.78285E-05	0.10926E-01
36	0.80540E-01	-0.37174E+00			0.12779E-01
37	0.15082E-01	-0.37034E+00	0.90720E-03	0.38285E-04	0.12559E-01
38	0.13881E+00				
39	0.13889E+00	-0.37120E+00	-0.13651E-03	-0.21731E-05	0.10621E-01
40	0.81097E-01	-0.37019E+00			0.12738E-01
41	0.16982E-01	-0.36835E+00	0.19305E-04	-0.98916E-06	0.12194E-01
42	0.13970E+00				
43	0.13976E+00	-0.37204E+00	-0.16029E-03	0.47699E-05	0.16930E-01
44	0.80554E-01	-0.37139E+00			0.12783E-01
45	0.15112E-01	-0.36999E+00	-0.86858E-03	-0.41526E-04	0.12561E-01
46	0.14074E+00	-0.51634E-03	-0.25640E-03	0.14073E-03	0.12439E-01
47	0.77759E-01	-0.51771E-03	-0.10758E-04		0.12140E-01
48	0.11750E-01	-0.48285E-03	0.22895E-03	-0.10278E-03	0.12038E-01
49	0.13921E+00	-0.47847E-04	-0.88500E-04	-0.12291E-05	0.12483E-01
50	0.77353E-01	-0.0	-0.63707E-05		0.11992E-01
51	0.11561E-01	-0.31813E-04	0.75732E-04	-0.22012E-05	0.12023E-01
52	0.14078E+00	-0.15781E-03	0.79493E-04	-0.14300E-03	0.12436E-01
53	0.77798E-01	-0.15923E-03	-0.19804E-05		0.12139E-01
54	0.11796E-01	-0.12427E-03	-0.77438E-04	0.98766E-04	0.12035E-01

CENTROIDAL CONCRETE STRESSES TSGCON(NEL,3) AND AGGREGATE INTERLOCK STRESS TSGAGG(NEL) ARE AS BELOW

ELEMENT NO.	SIGMAX	SIGMAY(Z)	SIGMAXY(XZ)	SIGMA-AGG
1	-0.83323E+03	-0.31993E+03	-0.51631E+03	-0.38193E+01
2	0.25875E+02	0.25971E+03	0.81976E+02	0.24299E+02

3	0.26013E+02	0.26144E+03	-0.82468E+02	-0.24369E+02
4	-0.82825E+03	-0.31859E+03	0.51368E+03	0.55641E+01
5	-0.45256E+03	-0.17054E+03	-0.51802E+03	0.0
6	-0.17184E+03	-0.45642E+03	-0.28006E+03	0.65053E+02
7	-0.17053E+03	-0.44783E+03	0.27635E+03	-0.63619E+02
8	-0.45589E+03	-0.16757E+03	0.51356E+03	0.0
9	-0.95736E+03	-0.[illegible]74E+03	-0.63467E+03	-0.27348E+02
10	0.42347E+02	0.[illegible]E+03	0.10704E+03	0.42326E+02
11	0.42539E+02	0.270[illegible]1E+03	-0.10735E+03	-0.43236E+02
12	-0.94940E+03	-0.41610E+03	0.62853E+03	0.31951E+02
13	-0.55775E+03	-0.22312E+03	-0.62595E+03	0.0
14	-0.23728E+03	-0.44094E+03	-0.32346E+03	0.12557E+03
15	-0.23586E+03	-0.43684E+03	0.32098E+03	-0.12979E+03
16	-0.59052E+03	-0.21747E+03	0.63661E+03	0.0
17	-0.83209E+03	-0.31940E+03	-0.51552E+03	-0.36327E+01
18	0.25566E+02	0.25833E+03	0.81267E+02	0.24643E+02
19	0.25743E+02	0.25909E+03	-0.81668E+02	-0.23910E+02
20	-0.82866E+03	-0.31846E+03	0.51371E+03	0.55614E+01
21	-0.45069E+03	-0.16632E+03	-0.51882E+03	0.0
22	-0.16825E+03	-0.44842E+03	-0.27467E+03	0.66154E+02
23	-0.17117E+03	-0.45003E+03	0.27755E+03	-0.64209E+02
24	-0.45530E+03	-0.16810E+03	0.51320E+03	0.0
25	0.30960E+03	0.22543E+02	-0.12131E+03	0.0
26	0.17829E+00	0.27490E+02	0.22138E+01	0.25966E+02
27	0.17055E+00	0.26477E+02	-0.21253E+01	-0.27596E+02
28	0.28948E+03	0.22846E+02	0.12159E+03	0.0
29	0.30989E+03	0.21849E+02	0.12049E+03	0.0
30	0.16913E+00	0.26491E+02	-0.21167E+01	-0.26700E+02
31	0.17364E+00	-0.26650E+02	0.21512E+01	0.26822E+02
32	0.28877E+03	0.23263E+02	-0.12264E+03	0.0
33	-0.34473E+03	0.13771E+02	0.11016E+03	0.0
34	-0.21296E+04	-0.12720E+03	0.10448E+03	0.0
35	-0.21350E+04	-0.12709E+03	-0.10440E+03	0.0
36	-0.35049E+03	0.15503E+02	-0.10384E+03	0.0
37	-0.34479E+03	0.13957E+02	-0.11005E+03	0.0
38	-0.21324E+04	-0.12727E+03	-0.10362E+03	0.0
39	-0.21333E+04	-0.12701E+03	0.10533E+03	0.0
40	-0.34955E+03	0.15448E+02	0.10455E+03	0.0

PRINCIPAL COMPRESSIVE AND TENSILE STRESSES AND STRAINS FOR EACH ELEMENT CENTROID ARE AS BELOW

ELEMENT NO.	PRINCIPAL COMPRESSIVE STRESS	PRINCIPAL COMPRESSIVE STRAIN	PRINCIPAL TENSILE STRESS	PRINCIPAL TENSILE STRAIN	ELEMENT CRACKED ?
1	-0.11532E+04	-0.38072E-03	0.0	0.10692E-02	YES
2	0.28558E+03	0.91387E-04	0.0	0.25158E-02	YES
3	0.28746E+03	0.91986E-04	0.0	0.24919E-02	YES
4	-0.11468E+04	-0.37863E-03	0.0	0.10588E-02	YES
5	-0.84841E+03	-0.29187E-03	0.22532E+03	0.12884E-03	NO
6	-0.62826E+03	-0.20746E-03	0.0	0.50329E-03	YES
7	-0.61836E+03	-0.20419E-03	0.0	0.49263E-03	YES
8	-0.84514E+03	-0.29058E-03	0.22168E+03	0.12742E-03	NO
9	-0.13781E+04	-0.45568E-03	0.0	0.12549E-02	YES
10	0.31289E+03	0.10042E-03	0.0	0.25274E-02	YES
11	0.31345E+03	0.10030E-03	0.0	0.24942E-02	YES
12	-0.13655E+04	-0.45148E-03	0.0	0.12260E-02	YES
13	-0.10384E+04	-0.35602E-03	0.25749E+03	0.15159E-03	NO
14	-0.67822E+03	-0.22392E-03	0.0	0.58763E-03	YES
15	-0.67269E+03	-0.22210E-03	0.0	0.57131E-03	YES
16	-0.10674E+04	-0.36562E-03	0.25937E+03	0.15405E-03	NO

17	-0.11515E+04	-0.38016E-03	0.0	0.10676E-02	YES
18	0.28389E+03	0.90846E-04	0.0	0.25155E-02	YES
19	0.28483E+03	0.91147E-04	0.0	0.24912E-02	YES
20	-0.11471E+04	-0.37872E-03	0.0	0.10583E-02	YES
21	-0.84646E+03	-0.29147E-03	0.22945E+03	0.13009E-03	NO
22	-0.61666E+03	-0.20363E-03	0.0	0.49951E-03	YES
23	-0.62120E+03	-0.20513E-03	0.0	0.49290E-03	YES
24	-0.84467E+03	-0.29040E-03	0.22118E+03	0.12722E-03	NO
25	-0.21856E+02	-0.18609E-04	0.35400E+03	0.12043E-03	NO
26	0.27668E+02	0.88537E-05	0.0	0.33400E-02	YES
27	0.26648E+02	0.85273E-05	0.0	0.33045E-02	YES
28	-0.24272E+02	-0.18484E-04	0.33659E+03	0.11496E-03	NO
29	-0.21904E+02	-0.18580E-04	0.35364E+03	0.12029E-03	NO
30	0.26661E+02	0.85314E-05	0.0	0.33402E-02	YES
31	0.26824E+02	0.85837E-05	0.0	0.33043E-02	YES
32	-0.24714E+02	-0.18669E-04	0.33676E+03	0.11507E-03	NO
33	-0.37588E+03	-0.12655E-03	0.44913E+02	0.44632E-04	NO
34	-0.21351E+04	-0.71509E-03	-0.12176E+03	0.10840E-03	NO
35	-0.21404E+04	-0.71692E-03	-0.12168E+03	0.10878E-03	NO
36	-0.37790E+03	-0.12715E-03	0.42915E+02	0.44151E-04	NO
37	-0.37586E+03	-0.12655E-03	0.45028E+02	0.44670E-04	NO
38	-0.21377E+04	-0.71597E-03	-0.12193E+03	0.10852E-03	NO
39	-0.21388E+04	-0.71637E-03	-0.12149E+03	0.10874E-03	NO
40	-0.37737E+03	-0.12699E-03	0.43274E+02	0.44231E-04	NO

STEEL MESH STRESSES ARE AS BELOW.

ELEMENT NO.	MESH DIRECTION				
1	1	STRESS=	0.19881E+05	STRAIN=	0.67393E-03
2	1	STRESS=	0.67444E+04	STRAIN=	0.22862E-03
3	1	STRESS=	0.66984E+04	STRAIN=	0.22707E-03
4	1	STPESS=	0.19755E+05	STRAIN=	0.66965E-03
6	1	STRESS=	-0.10695E+04	STRAIN=	-0.36255E-04
7	1	STRESS=	-0.10256E+04	STRAIN=	-0.34767E-04
8	1	STRESS=	-0.73981E+03	STRAIN=	-0.25078E-04
9	1	STRESS=	0.23607E+05	STRAIN=	0.80025E-03
10	1	STRESS=	0.73674E+04	STRAIN=	0.24974E-03
11	1	STRESS=	0.72109E+04	STRAIN=	0.24444E-03
12	1	STRESS=	0.23368E+05	STRAIN=	0.79215E-03
13	1	STRESS=	-0.10857E+04	STRAIN=	-0.36803E-04
14	1	STRESS=	-0.10333E+03	STRAIN=	-0.35027E-05
15	1	STRESS=	-0.22757E+03	STRAIN=	-0.77142E-05
16	1	STRESS=	-0.96871E+03	STRAIN=	-0.32838E-04
17	1	STRESS=	0.19844E+05	STRAIN=	0.67266E-03
18	1	STRESS=	0.66599E+04	STRAIN=	0.22576E-03
19	1	STRESS=	0.67133E+04	STRAIN=	0.22757E-03
20	1	STRESS=	0.19751E+05	STRAIN=	0.66953E-03
21	1	STRESS=	-0.73632E+03	STRAIN=	-0.24960E-04
22	1	STRESS=	-0.10434E+04	STRAIN=	-0.35369E-04
23	1	STRESS=	-0.10543E+04	STRAIN=	-0.35740E-04
24	1	STRESS=	-0.74597E+03	STRAIN=	-0.25287E-04
25	1	STRESS=	0.18816E+02	STRAIN=	0.63783E-06
26	1	STPESS=	-0.23766E+03	STRAIN=	-0.80563E-05
27	1	STRESS=	-0.31506E+03	STRAIN=	-0.10680E-04
28	1	STRESS=	0.59464E+02	STRAIN=	0.20157E-05
29	1	STRESS=	0.11470E+02	STRAIN=	0.38882E-06
30	1	STRESS=	-0.28047E+03	STRAIN=	-0.95074E-05
31	1	STRESS=	-0.27941E+03	STRAIN=	-0.94716E-05

32	1	STRESS=	0.64889E+02	STRAIN=	0.21996E-05
33	1	STRESS=	0.96904E+03	STRAIN=	0.32849E-04
34	1	STRESS=	0.31333E+04	STRAIN=	0.10621E-03
35	1	STRESS=	0.31447E+04	STRAIN=	0.10660E-03
36	1	STRESS=	0.99658E+03	STRAIN=	0.33782E-04
37	1	STRESS=	0.97098E+03	STRAIN=	0.32915E-04
38	1	STRESS=	0.31379E+04	STRAIN=	0.10637E-03
39	1	STRESS=	0.31424E+04	STRAIN=	0.10652E-03
40	1	STRESS=	0.99430E+03	STRAIN=	0.33705E-04

REINFORCEMENT ELEMENT STRESSES AND STRAINS ARE AS BELOW;

REINFORCEMENT NUMBER	REINFORCEMENT TYPE	STRESS	STRAIN
1	CONVENTIONAL	0.10869E+04	0.36846E-04
2	CONVENTIONAL	0.39270E+03	0.13312E-04
3	CONVENTIONAL	0.10641E+04	0.36071E-04
4	CONVENTIONAL	0.55241E+05	0.33459E-02
5	CONVENTIONAL	0.55226E+05	0.33108E-02
6	CONVENTIONAL	0.55240E+05	0.33434E-02
7	CONVENTIONAL	0.55253E+05	0.33740E-02
8	CONVENTIONAL	0.55243E+05	0.33503E-02
9	CONVENTIONAL	0.55254E+05	0.33767E-02
10	CONVENTIONAL	0.12207E+04	0.41381E-04
11	CONVENTIONAL	0.66893E+03	0.22675E-04
12	CONVENTIONAL	0.12266E+04	0.41581E-04
13	CONVENTIONAL	-0.32771E+04	-0.11109E-03
14	CONVENTIONAL	-0.53306E+04	-0.18070E-03
15	CONVENTIONAL	-0.32609E+04	-0.11054E-03
16	CONVENTIONAL	-0.23460E+05	-0.79524E-03
17	CONVENTIONAL	-0.21593E+05	-0.73197E-03
18	CONVENTIONAL	-0.23413E+05	-0.79365E-03
19	CONVENTIONAL	-0.23378E+05	-0.79246E-03
20	CONVENTIONAL	-0.21547E+05	-0.73040E-03
21	CONVENTIONAL	-0.23448E+05	-0.79486E-03
22	CONVENTIONAL	-0.32689E+04	-0.11081E-03
23	CONVENTIONAL	-0.52137E+04	-0.17674E-03
24	CONVENTIONAL	-0.32584E+04	-0.11045E-03
25	PRESTRESSED	0.16764E+06	0.67038E-02
26	PRESTRESSED	0.16702E+06	0.66803E-02
27	PRESTRESSED	0.16762E+06	0.67031E-02
28	PRESTRESSED	0.25533E+06	0.10013E-01
29	PRESTRESSED	0.25440E+06	0.99778E-02
30	PRESTRESSED	0.25527E+06	0.10010E-01
31	PRESTRESSED	0.25608E+06	0.10041E-01
32	PRESTRESSED	0.25545E+06	0.10017E-01
33	PRESTRESSED	0.25615E+06	0.10044E-01
34	PRESTRESSED	0.16776E+06	0.67084E-02
35	PRESTRESSED	0.16727E+06	0.66897E-02
36	PRESTRESSED	0.16777E+06	0.67086E-02
37	BONDED	0.11844E+03	0.65029E-04
38	BONDED	0.13655E+03	0.75877E-04
39	BONDED	0.11814E+03	0.64850E-04
40	BONDED	0.68998E+00	0.35390E-06
41	BONDED	0.10457E+01	0.53644E-06
42	BONDED	0.87149E+00	0.44703E-06
43	BONDED	0.11695E+03	0.64146E-04
44	BONDED	0.13662E+03	0.75873E-04
45	BONDED	0.11702E+03	0.64187E-04

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THE LOAD INCREMENT VECTOR IS AS BELOW;
  RINC( 24 ,2 ) =  -0.50000E+04
  RINC( 28 ,2 ) =  -0.50000E+04
  RINC( 32 ,2 ) =  -0.50000E+04

THE TOTAL LOAD VECTOR IS AS BELOW;
  RTOTAL(  1 ,1 ) =   0.16465E+05
  RTOTAL(  4 ,1 ) =   0.32930E+05
  RTOTAL(  7 ,1 ) =   0.16465E+05
  RTOTAL( 24 ,2 ) =  -0.20000E+05
  RTOTAL( 28 ,2 ) =  -0.20000E+05
  RTOTAL( 32 ,2 ) =  -0.20000E+05
  RTOTAL( 46 ,1 ) =  -0.16465E+05
  RTOTAL( 49 ,1 ) =  -0.32930E+05
  RTOTAL( 52 ,1 ) =  -0.16465E+05

TOTAL REINFORCEMENT FORCE RESTRAINT VECTOR;
 RESTRAINT FORCE AT NODE 10 IN DIRECTION 1 =   0.60417E+04
 RESTRAINT FORCE AT NODE 14 IN DIRECTION 1 =   0.12081E+05
 RESTRAINT FORCE AT NODE 18 IN DIRECTION 1 =   0.60418E+04
 RESTRAINT FORCE AT NODE 22 IN DIRECTION 1 =  -0.43750E+00
 RESTRAINT FORCE AT NODE 26 IN DIRECTION 1 =  -0.17031E+01
 RESTRAINT FORCE AT NODE 30 IN DIRECTION 1 =  -0.61328E+00
 RESTRAINT FORCE AT NODE 34 IN DIRECTION 1 =  -0.60413E+04
 RESTRAINT FORCE AT NODE 38 IN DIRECTION 1 =  -0.12080E+05
 RESTRAINT FORCE AT NODE 42 IN DIRECTION 1 =  -0.60412E+04


********************************
*  LOAD INCREMENT NUMBER  6   *
********************************


*** DIAGONAL TERM TS( 24 ,1)=  -0.46834E+07 IN BLOCK NO.  4 ***


20:58:16 56.427 RC=0
```

APPENDIX H

INPUT PREPARATION NOTES

APPENDIX H

The purpose of this appendix is to assist the computer program user in the compilation of the input data file. Units of inches and pounds are used throughout, and if large numbers are to be read in, the E format should be used. The user should be especially conscious of the sign conventions listed in Appendix A. In referring to several input variables in the description below, the symbolic name may be used for the sake of brevity. The alphabetic listing of all symbolic names and their corresponding codes, where applicable, are given in Appendix B. Complete freedom in input data entry is provided through the use of a semi-freefield format, where commas are used to separate adjacent entries, as illustrated in the sample input listing in Appendix F. A sufficient number of columns have been allocated for the entry on all variables of realistic size, but reference to the format statements in subroutine READIN listed in Appendix D will clarify the maximum number of assigned columns to accommodate the full digit length. Following the description of each input card or assemblage of associated cards, the significance and qualifications that pertain to the important variable entries will be commented upon.

The data cards must occur in the following order:

1. <u>Echo Check Card</u>

Order of Entries	Description
1	IWRITE

Comments: If IWRITE = 1, the input data will be printed out so that it can be quickly checked.

2. Number of Finite Elements Card

Order of Entries	Description
1	NELT
2	NELCHK

Comments: (a) If it is recognized that all concrete elements will not crack, the element numbering system should be chosen such that only the first NELCHK number of elements will be checked for cracking.

(b) NELT does not include the number of diaphragm elements.

3. Concrete Finite Elements Description Cards

One card for each concrete finite element in ascending order.

Order of Entries	Description
1	INDCEL (NEL)
2	INELSZ (NEL)
3	INELTY (NEL)
4	ELTHN (NEL)
5	First element node number
6	Second node number
7	Third node number
8	Fourth node number
9	INDOWL (NEL)
10	INMESH (NEL)
11	INSZMS (NEL)
12	WIDTHC (NEL)

Comments: (a) Concrete elements of the same dimensions, thickness, and percentage of reinforcement are assigned the same arbitrary integre number INELSZ (NEL).

(b) Only those web elements that adjoin a tension flange element will develop dowel shear resistance.

(c) Elements that have the same percentages of steel mesh reinforcement are assigned the same arbitrary integre number INSZMS (NEL).

(d) The effective dowel width of the beam is distributed proportionately between the web elements that adjoin the tension flange.

4. Concrete Shrinkage Data Card

Order of Entries	Description
1	NELTOP
2	SIGXT1
3	SIGXT2
4	SIGXB1
5	SIGXB2
6	SIGXS1
7	SIGXS2
8	EXT1
9	EXT2
10	EXB1
11	EXB2
12	EXS1
13	EXS2

5. Alternate Top and Bottom Flange Shrinkage Elements Cards

Alternate cards for top and bottom flange concrete elements that develop shrinkage stresses.

The first card will list ten top flange concrete elements, the second card ten bottom flange concrete elements. This sequence of cards is continued until all flange elements subjected to shrinkage stresses are listed. The last two cards will contain at least one entry, but will not exceed ten entries.

6. Number of Elements with Steel Mesh Card

The only entry on the card is NELSM.

Comments: It is imperative for the proper functioning of the program that all transverse shear or torsion reinforcement be represented by an equivalent steel mesh distributed throughout the element.

7. Number of Layers in Steel Mesh Card

The only entry is NDIRNS.

Comments: If NELSM = 0, omit this card and proceed to card 9.

8. Steel Mesh Description Cards

Two cards for each element containing a steel mesh. The element number is contained on the first card. The second card is as below:

Order of Entries	Description
1	Steel mesh percentages for all layers
2	Corresponding inclinations

Comments: A steel mesh layer is defined as a discrete system of parallel bars.

9. Number of Diaphragms Card

The only entry is NELD.

Comments: (a) NELD is the total sum of actual, equivalent, and warping diaphragm elements.

(b) If NELD = 0, proceed directly to card 11.

10. Diaphragm Description Cards

One card for each diaphragm.

Order of Entries	Description
1	INELTY (NEL)
2	ELTHN (NEL
3	First element node number
4	Second element node number
5	Third element node number
6	Fourth element node number
7	NDREF (NEL)
8	DPMMOD (NEL)

Comments: Identical diaphragm elements have the same NDREF (NEL) integre value.

11. Number of Reinforcement Elements Card

The only entry is NREO.

Comments: NREO is the sum of all prestress and conventional longitudinal reinforcement elements and bond linkages.

12. Reinforcement Type Description Cards

One card for each reinforcement element.

Order of Entries	Description
1	First reinforcement node number
2	Second reinforcement node num[illegible]
3	Reinforcement element type ind[illegible]

Comments: (a) The reinforcement node numbers must be [illegible]mbered in the positive axis direction.

(b) The following individual reinforcement cards must be in the same order.

13. Conventional Longitudinal Reinforcement Card

This card is read if INRTY(NR) = -1.

Order of Entries	Description
1	INRDN (NR)
2	RAREA (NR)

14. Prestress Reinforcement Card

This card is read if INRTY (NR) = 0.

Order of Entries	Description
1	INRDN (NR)
2	RAREA (NR)
3	TSGPRE (NR)
4	TEPRE (NR)

15. Bond Linkage Card

This card is read if INRTY (NR) = 1.

The only entry is CAREA (NR).

16. Number of Finite Element Nodes Card

The only entry is NNODES.

Comments: Bond spring linkage elements adjoin adjacent concrete and reinforcement nodes. However, no bond spring inkages are provided at beam ends.

17. Finite Element Node Description Cards

One card for each node.

Order of Entries	Description
1	ICNODE (I)
2	X (I)
3	Y (I)
4	Z (I)

Comments: (a) Nodes adjoining two non-planar elements must be designated as corner nodes: ie. they possess five degrees of freedom.

(b) Internal node numbers connecting web diaphragm elements are assigned ICNODE (I) = 2: ie. they have four degrees of freedom.

(c) Node adjoining co-planar elements are designated as interior nodes, with three degrees of freedom.

(d) The X, Y, and Z coordinates are global axis coordinates, the direction of the global axes being defined in Appendix A.

18. Strength Moduli Card

Order of Entries	Description
1	CONMOD
2	REOMOD

3	PREMOD
4	SLPMOD
5	SMSMOD

19. Ultimate Strength Card

Order of Entries	Description
1	FC
2	FT
3	FAGG
4	FUR
5	FUP
6	FUMS

20. Ultimate Strain Cards

Order of Entries	Description
1	EEREO
2	EEPRE
3	ECULT
4	ERULT
5	EPULT
6	EMSULT
7	ESLIP
8	DF

21. Poisson Ratios Card

Order of Entries	Description
1	P1
2	P2
3	P3
4	PU

22. Material Deviation Card

Order of Entries	Description
1	CONDEV
2	DEVCON
3	DEVREO
4	PREDEV
5	SLPDEV
6	LDEV
7	AVCSP
8	RELAX

Comments: (a) Entries 1 to 5 specify the maximum allowable deviation percentages in material behaviour that is permitted before corrective measures are taken. Only CONDEV and PREDEV of the five variables mentioned above have a direct influence on the decision to invoke the iterative process. Obviously, if the tolerances expressed by CONDEV and PREDEV are too demanding, the expensive iterative process will be undertaken frequently. Thus, a compromise has to be struck between the maximum allowable deviation that is acceptable and the cost incurred if that maximum level of deviation is to be enforced Values used in this application of the computer model were CONDEV = 8% and

PREDEV = 10%. Severe tolerances expressed by DEVCON and DEVREO do not increase computing cost significantly, and it is recommended that these values be kept small.

(b) IDEV is the critical weighted iteration integre that determines whether the iterative procedure will be invoked. Each important aspect of structural deviation is given a weighting, as declared in the following card. If the sum total of the deviation counters exceeds IDEV, a cycle of the iterative process will be initiated.

(c) RELAX is the relaxation factor employed to improve the rate of convergence in the highly inelastic segments of reinforcement stress-strain curves during the iterative process. The variable will invariably exceed unity, but its optimum value is very much dependent upon the nature of the analysis.

23. Deviation Weighting Card

Order of Entries	Description
1	MAXIT
2	NWTCON
3	NWTREO
4	NWTPRE

Comments: If allowable behavioural deviation is exceeded in a concrete element, a conventional reinforcement or steel mesh element, or a prestress reinforcement element, the deviation integre counter is increased by the addition of NWTCON, NVTREO, or NWTPRE respectively. The deviation integre counter within the program is the variable ID. Thus, when ID exceeds IDEV after the application of a load increment, iteration is commenced until deviation is reduced to the extent that ID is less than or equal to IDEV.

24. Number of Load Types Card

The only entry on the card is NLDTY.

Comments: A load type is one where the relationship of the individual loads within successive load increments remains constant.

25. General Load Information Card

One card for each load type.

Order of Entries	Description
1	NINCRT (I)
2	NLOADS (I)

Comments: (a) For each load type, this card and the cards of item 26 are read in.

(b) The application of prestress forces comprises the first load increment in the study of prestressed concrete beam behaviour.

(c) Load increments can be heavy in the elastic and early post-cracking regions, but should become progressively lighter as behaviour becomes more inelastic. Small load increments are necessary close to failure if the ultimate load conditions are to be estimated within close bounds.

26. Individual Load Description Cards

One card for each nodal load.

Order of Entries	Description
1	NODER (I)
2	KODER (I)
3	VALUER (I)

27. Number of Boundary Conditions Card

The only entry on this card is NDISPL.

28. Individual Boundary Conditions Cards

One card for each boundary condition.

Order of Entries	Description
1	NODED (I)
2	KODED (I)
3	VALUED (I)

29. Number of Concrete Elements Display Card

The only entry on this card is PNELT (integre)

30. Concrete Elements Display Card

All the element numbers of those elements whose centroidal stresses will be displayed, are listed on this card. (integres)

31. Number of Steel Mesh Elements Display Card

The only entry on this card is PNELSM (integre).

32. Steel Mesh Elements Display Card

All steel mesh elements to be displayed have their element numbers listed on this card (integres).

33. Number of Reinforcement Elements Display Card

The only entry on this card is PNREO (integre).

34. Reinforcement Elements Display Card

Similar to card 30, but concerning reinforcement elements.

35. Number of Nodal Deflections Display Card

The only entry on this card is PDTOT.

36. Nodal Deflections Display Card

Similar to card 30, but with respect to nodal deflections.

37. Printout Control Card

All data entries are contained on one card.

Order of Entries	Description
1	IDEFLN
2	ICON3
3	ICONPR
4	IMESH
5	IREO
6	ILOAD
7	ITLOAD
8	ISTIF

The data file is now complete. If too few data cards have been included, the end of file card will be read, and execution will immediately cease. If too many cards have been inserted in the data file, the error will be apparent in the echo check printout of the data.

APPENDIX I

EXPERIMENTAL PROGRAM TEST RESULTS

APPENDIX I

EXPERIMENTAL PROGRAM TEST RESULTS

Note: Units of kips, inches, and degrees are used throughout.

1. Beam Dimensions and Reinforcement Details

Refer to Figures 4.3 and 4.4.

2. Loading Systems

Three different patterns of application of the torque and bending moment loads were employed in the testing of the seven beams, and are shown in Figure I-1.

3. Prestress Levels and Shrinkage Stresses

Refer to Table 4.3 for prestress levels.

Between transfer and testing, considerable concrete shrinkage occurred, stressing the longitudinal conventional steel in compress and surrounding concrete in tension. Only the concrete in the flanges is assumed to develop shrinkage stresses.

	Beams						
Shrinkage Stresses	R1	R2	R3	R4	R5	T1	T2
Tension in Top Flange Concrete	.115	.135	.125	.095	.12	.12	.075
Tension in Bottom Flange Concrete	.203	.23	.216	.117	.2	.25	.25
Compression in Top Reinforcement	13.5	15.75	14.6	11.1	14.	15.	9.
Compression in Bottom Reinforcement	24.0	27.0	25.5	19.9	25.	15.	15.

TABLE I-1 SHRINKAGE STRESSES

4. Test Loading and Deformation Results

In the following tables, the rotation values are in degrees and were measured over a 30 inch central beam length. The deflection measurements are the pure bending vertical displacements of the central cross-section.

The torque and bending moments are those values at the central cross-section. The shear values are those close to the central cross-section.

5. Reinforcement Stresses

Refer to Figures 5.6 to 5.12.

Load Increment No.	Loading						Deformations	
	System	T	P	Bending Moment	Torque	Shear	Central Defln.	Rotation
1		0.0		0.0	0.0		0.0	0.0
2		2.0		116	48		.024	.036
3		4.0		216	96		.048	.068
4		6.0		314	143		.073	.0575
5		8.0		414	192		.1045	.08
6		10.0		510	238		.139	.10
7		10.78		552	259		.182	[illegible]
8		12.0		611	288		[illegible]	.295
9		12.89		656	310		[illegible]	[illegible]
10		14.04		713	337		.4	.4776
11	A	14.96	0.0	759	359	0.0	.45	.585
12		16.06		813	385.4		.54	.667
13		17.0		859	408		.62	.768
14		17.97		908	431		.69	.881
15		19.0		959	456		.86	1.427
16		19.657		991	472		.97	1.306
17		21.1		1062	506			
18		21.65		1090	520			
19		21.05		1060	505			

Note: Linear transducers were removed after load increment No. 15.

TABLE I-2 BEAM R1 TEST RESULTS

Load Increment No.	Loading						Deformations	
	System	T	P	Bending Moment	Torque	Shear	Central Defln.	Rotation
1			0	0.0			0.0	
2			2	66.4			.0167	
3			4	136			.03	
4			6	206			.058	
5		0.0	8	276	0.0		.078	0.0
6			10	346			.097	
7			12	417			.117	
8			13	452			.128	
9			14	488			.14	
10		1.04		555	25		.163	.001
11		1.92		600	46		.18	.0032
12		3.1		657	74		.21	.009
13		4.03		703	97		.24	.017
14		5.07		755	122		.29	.043
15	A	6.0		797	142	0.0	.36	.104
16		7.0		850	168		.45	.2
17		7.9		896	190		.52	.26
18		9.0		949	216		.6	.33
19		10		998	240		.68	.39
20		10.9		1044	262		.77	.46
21		11.3		1064	271		.816	.52
22		12.		1097	287		.91	.61
23		12.19		1107	292		.97	.68
24		12.75		1135	306			
25		13.35		1165	321			
26		13.9		1192	334			
27		14.5		1222	348			
28		14.58		1226	350			
29		15.06		1249	361			
30		15.49	14.0	1271	372			

Note: Transducers were removed after load increment No. 23.

TABLE I-3 BEAM R2 TEST RESULTS

Load Increment No.	Loading						Deformations	
	System	T	P	Bending Moment	Torque	Shear	Central Defln.	Rotation
1			0.0	0.0		0.0	0.0	
2			2	72		1	.015	
3			2	72		1	.016	
4			4	145		2	.03	
5			6	218		3	.043	
6			8	291		4	.054	
7		0.0	10	364	0.0	5	.067	0.0
8			12	437		6	.078	
9			14	510		7	.09	
10			16	583		8	.102	
11			18	656		9	.115	
12			20	729		10	.13	
13			22	802		11	.15	
14		2.04		854	49		.177	0.0
15		4.1		906	99		.205	.007
16		5.8		947	138		.25	.02
17	C	8.0		1003	192		.325	.08
18		9.8		1049	236		.425	.16
19		11.86		1099	285		.52	.246
20		12.8		1123	307		.59	.32
21		13.9		1151	334		.64	.41
22		14.84		1174	356		.69	.4
23		15.8		1199	380		.77	.46
24		17		1229	409		.85	.54
25		17.3		1235	415		.89	.575
26		17.81		1248	428		.93	.6
27		18.2		1258	437		1.0	.72
28		19.2		1283	461			
29		19.6		1293	471			
30		20.14		1306	483			
31		20.76		1322	498			
32		21.3		1336	510			
33		21.65		1344	519			
34		22.2	22	1357	532	11		

Note: Linear transducers were removed after load increment No. 27.

TABLE I-4 BEAM R3 TEST RESULTS

Load Increment No.	Loading						Deformations	
	System	T	P	Bending Moment	Torque	Shear	Central Defln.	Rotation
1			0.0	0.0		0.0	0.0	
2			2	73		1	.01	
3			4	146		2	.023	
4			6	220		3	.036	
5		0.0	8	293	0.0	4	.05	0.0
6			10	366		5	.063	
7			12	440		6	.076	
8			14	513		7	.089	
9			16	587		8	.102	
10			18	660		9	.116	
11			20	733		10	.131	
12		4.04		831	97		.167	
13		6.1		883	147		.197	Similar
14	C	7.9		928	190		.244	to
15		9.84		976	236		.33	R3*
16		12.1		1032	290		.45	
17		13.95		1079	335		.55	
18			22	1152		11	.61	
19			24	1222		12	.6725	
20			26	1297		13	.78	
21			28	1366		14	.90	
22			30	1403		15	.99	
23			31	1443		15.5	1.09	
24			32	1484		16	1.28	
25			33	1511		16.5	1.51	
26			34	1554		17	1.665	
27			35	1591		17.5	2.02	
28		13.95	36	1624	335	18		

* Rotation results for this short torsion loading interval are similar to those corresponding results for beam R3.

Note: Linear transducers were removed after load increment No. 27.

TABLE I-5 BEAM R4 TEST RESULTS

Load Increment No.	Loading						Deformations	
	System	T	P	Bending Moment	Torque	Shear	Central Defln.	Rotation
1		0.0		0.0	0.0		0.0	0.0
2		1.96		49	47		.02	.006
3		3.92		98	94		.043	.009
4		6.15		154	148		.068	.067
5		8		200	192		.085	.0125
6		9		250	240		.103	.0325
7		11		276	265		.113	.036
8		11.9		298	282		.122	.043
9		13		324	311		.131	.063
10		14.2		355	341		.173	.156
11		14.84		371	356		.196	.21
12		15.5		388	372		.217	.25
13		15.9		398	382		.233	.272
14		16.7		418	401		.253	.31
15		17.2		431	414		.273	.34
16	B	17.4	0.0	435	417	0.0	.294	.39
17		18		451	433		.307	.42
18		18.5		463	444		.327	.437
19		19		476	457		.354	.49
20		19.5		487	468		.383	.53
21		19.9		497	477		.4	.57
22		20.5		513	492		.426	.61
23		20.93		523	502		.44	.64
24		21.6		540	519		.47	.69
25		21.82		546	524		.486	.73
26		22.3		558	536		.504	.76
27		22.84		571	548		.53	.8
28		23.3		583	559		.55	.84
29		23.87		597	573		.583	.89
30		24.38		609	585		.617	.97
31		25		626	601		.65	1.06
32		2		640	615		.683	1.167

TABLE I-6 BEAM R5 TEST RESULTS

Beam	Load Increment No.	Loading						Deformations	
		System	T	P	Bending Moment	Torque	Shear	Central Defln.	Rotation
	1		0.0		0.0	0.0		0.0	0.0
	2		1.19		77	28.6		.0225	.018
	3		2.03		118	49		.04	.031
	4		2.95		164	71		.062	.032
	5		4.0		215	97		.085	.008
	6		5.08		269	[illegible]		.109	.02
T1	7	A	6.07	0.0	319	146		.14	.052
	8		6.75		352	[illegible]		.215	.067
	9		8.13		420	95		.316	.13
	10		10.0		511	[illegible]		.565	.38
	11		11.0		563	264		.68	.53
	12		11.3		577	271		.743	.63
	13		9.5		490	229			
	1			0.0	0.0			0.0	
	2			2	72			.029	
	3			4	145			.063	
	4		0.0	6	216	0.0		.1	0.0
	5			7	252			.12	
	6			8	289			.143	
	7			9	322			.166	
	8			10	361			.197	
	9			11	396			.24	
	10		.98		445	24		.32	-
T2	11	A	2		495	48	0.0	.38	-
	12		3		546	73		.49	.1
	13		4		594	96		.68	.035
	14		4.53		620	109		.74	.156
	15		5		647	121		.8	.225
	16		5.45		666	131		.86	.26
	17		6		690	142		.955	.286
	18		6.43		715	154		1.06	.4
	19		6.9		738	166		1.25	.528
	20		7.4		763	178		1.53	.79
	21		7.94		789	191			
	22		8.2		801	196.5			
	23		8.13	11	798	195			

Notes: Linear transducers were removed after load increment 13 for T1, and increment 20 for beam T2

TABLE I-7 BEAMS T1 AND T2 TEST RESULTS

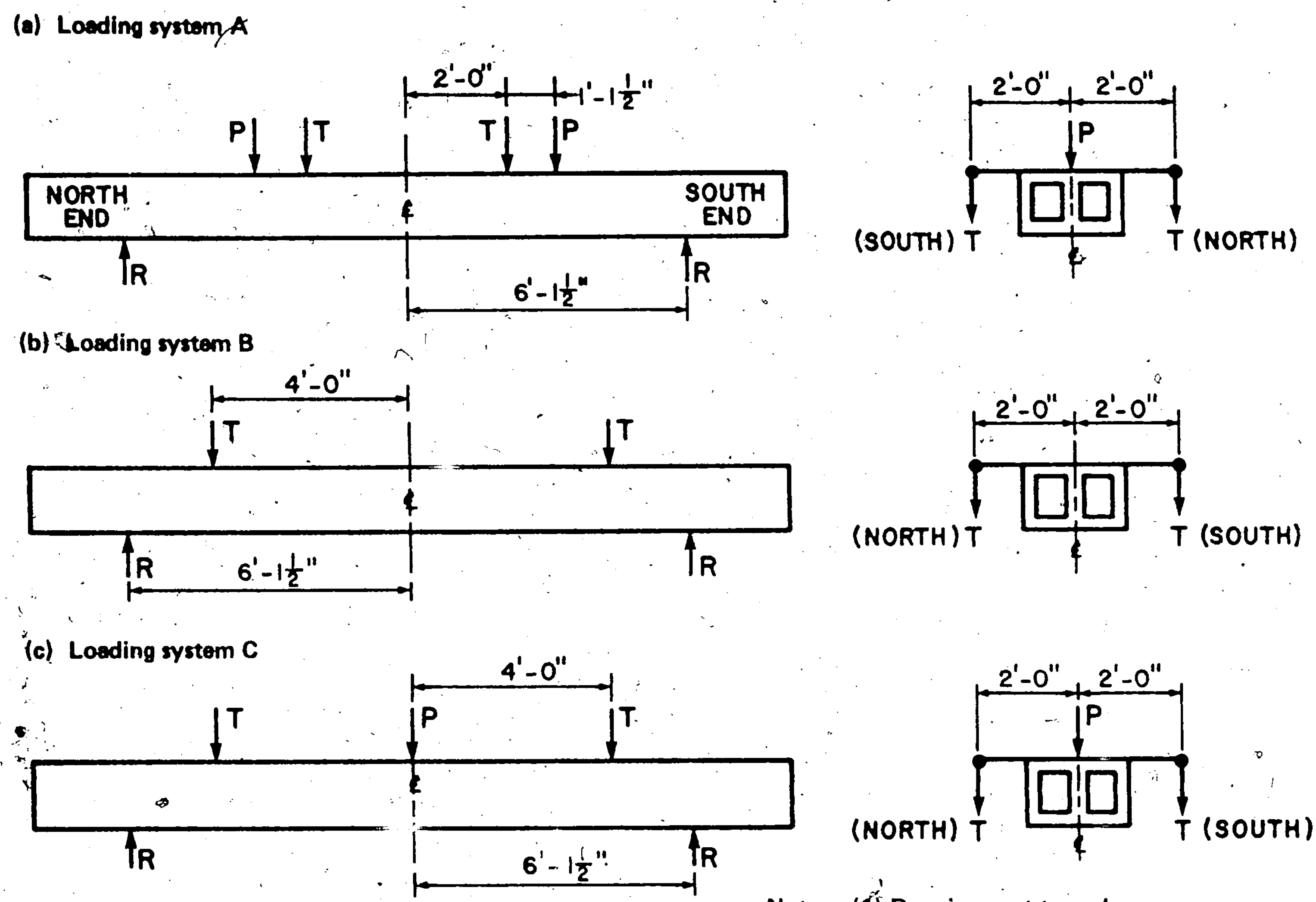

FIG. I-1 LOADING SYSTEMS

Notes : (1) Drawings not to scale
(2) Loading systems symmetrical about centreline
(3) T = torque load P = bending moment load