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16. Abstract

This report contains detailed recommendations for desirable revisions in the AASHTO Specifications design approach for slender columns, and introduces second order analysis computer codes for electronic computation to improve design of slender concrete bridge piers and bents. The computer codes contain designer-oriented input provisions so that automatic application of load factors and simplified reinforcement distribution patterns can be selected. Separate verification showed the programs to be accurate for a wide range of pier configurations and loading. Input guides and several examples are included. The programs were developed to cover a wide range of variables indicated from a survey of highway departments. This survey is also summarized in this report.

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by

R. W. Poston
M. Diaz
J. E. Breen
and
J. M. Roesset

Research Report 254-2F

Research Project 3-5-79-254

Design of Slender Nonprismatic or Hollow Bridge Piers or Columns

Conducted for

TEXAS

State Department of Highways and Public Transportation

In Cooperation with the
U.S. Department of Transportation
Federal Highway Administration

by

CENTER FOR TRANSPORTATION RESEARCH
BUREAU OF ENGINEERING RESEARCH
THE UNIVERSITY OF TEXAS AT AUSTIN

April 1983

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

There was no invention or discovery conceived or first actually reduced to practice in the course of or under this contract, including any art, method, process, machine, manufacture, design or composition of matter, or any new and useful improvement thereof, or any variety of plant which is or may be patentable under the patent laws of the United States of America or any foreign country.

P R E F A C E

In this report specific design procedures and details concerning the use of several computer programs developed to assist in the design of slender concrete bridge piers are outlined. The detailed verification of the program results through comparison with a wide range of experimental results is contained in a companion report on this project.

The overall project objective was to develop design-oriented procedures for slender reinforced concrete bridge piers with special attention to tapered and hollow bridge piers. In the initial phase of the study a comprehensive survey was carried out which documented the general ranges of important bridge pier parameters. Both the experimental and analytical phases of the study were aimed at providing designer-oriented procedures to simplify design of slender bridge piers for the ranges of variables reported in the survey.

The work was sponsored by the Texas State Department of Highways and Public Transportation and the Federal Highway Administration. It was administered by the Center for Transportation Research at The University of Texas at Austin. Close liaison with the State Department of Highways and Public Transportation has been maintained through Mr. Dave McDonnold, the contact representative during the project. Mr. Gary Johnson has provided similar liaison for the Federal Highway Administration.

The Project was conducted in the Phil M. Ferguson Structural Engineering Laboratory located at the Balcones Research Center of The University of Texas at Austin. The authors are particularly indebted to Mr. John Sladek, who developed the initial state of the art survey document and helped with the overall study.

S U M M A R Y

This report contains detailed recommendations for desirable revisions in the AASHTO Specifications design approach for slender columns, and introduces second order analysis computer codes for electronic computation to improve design of slender concrete bridge piers and bents. The computer codes were developed to treat solid and hollow cross sections and to include both prismatic and nonprismatic piers and bents. The computer codes contain designer-oriented input provisions so that automatic application of load factors and simplified reinforcement distribution patterns can be selected. Separate verification showed the programs to be accurate for a wide range of pier configurations and loading. Input guides and several examples are included. The programs were developed to cover a wide range of variables indicated from a survey of highway departments. This survey is also summarized in this report.

I M P L E M E N T A T I O N

The computer programs reported in detail in this report were developed to carry out the requirements of AASHTO Specification Article 1.5.34(A)(1) which specifies the requirements for a general analysis for slenderness effects in compression members. The programs developed can handle a wide range of pier cross sections and geometries. Designer-oriented input guides allow simplified input for square column, rectangular column, circular column, oval column, and hollow cross sections. The piers can have varying geometry along their height. The programs have straightforward input for constant sections, linearly tapering or flaring sections. Other irregular geometries can be specified. The program PIER handles the case of a single isolated pier with variable end conditions, while program FPIER treats the case of a pier bent with up to two bays and three stories. The design programs were separately verified by comparison with comprehensive test results in the companion report. Acceptance of these programs by designers should make it easier for them to use a general analysis for slender compression members, leading to safer and more economical pier design. Comparative design studies between solid and cellular tall piers for bridges in the 200 ft span range have indicated that provisions of the cellular pier with the same approximate stiffness as a solid pier can result in substantial dead load reductions resulting in important foundation cost savings as well as savings in pier material costs. Use of these programs will make it easier to design such cellular sections and utilize these possible savings. In addition, the use of the programs should greatly simplify analysis of tapered and flared piers which may be desirable for economic or aesthetic reasons.

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NOTATION

A_{st}	= total area of longitudinal reinforcement
$a_{11}, a_{12}, a_{13}, a_{21}, a_{22}, a_{23}, a_{31}, a_{32}, a_{33}$	= flexibility parameters of a section
b	= width of compression face
$b_{11}, b_{12}, b_{13}, b_{21}, b_{22}, b_{23}, b_{31}, b_{32}, b_{33}$	= flexibility parameters of a section
c	= distance from extreme compression fiber to neutral axis
d	= distance from extreme compression fiber to centroid of tension reinforcement
D	= diameter
DFX	= change in force in the x-direction
DFZ	= change in force in the z-direction
e	= eccentricity of axial load from neutral axis of a compression member
E_c	= concrete modulus
E_s	= steel modulus
E_t	= tangent modulus
f_{bb}	= flexibility of end B of member forces at end B
f_c	= concrete stress
f	= flexibility matrix of a section
f'_c	= compressive strength of concrete (determined with 6 in. x 12 in. cylinders)
f''_c	= $k \cdot f'_c$
f_s	= steel stress
f_t	= tensile stress in concrete

f'_t = tensile strength of concrete

f_y = yield point of steel

ΔF = force matrix

ΔF_g = incremental forces in global axis

G = shear modulus of elasticity

h = overall thickness of a member

h' = height of column in Reduction Factor Method

h_s = height of story, center to center of floors or between regions under consideration as in a bridge pier

H = lateral load

H_u = total factored lateral force acting on a story (segment)

J = polar moment of inertia

k = effective length factor; or capacity reduction factor

k_u = c/d at ultimate conditions

K = stiffness matrix of a section

k' = 57 = coefficient to calculate correction in steel strain

K_{AA} = stiffness of end A of member for displacements at end A

K_{AB} = stiffness of end A of member for displacements at end B

K_{BA} = stiffness of end B of member for displacements at end A

K_{BB} = stiffness of end B of member for displacements at end B

$\Delta \ell$ = length of a segment

ℓu = unsupported length of columns

L = length of member

M = moment about plastic centroid

ΔM_y = incremental moment about y-axis

ΔM_{yb} = incremental moment about y-axis of end B of member

ΔM_z = incremental moment about the z-axis

ΔM_{zb} = incremental moment about the z-axis of end B of member

N = axial load

ΔN = incremental axial load

P = axial load

P_o = pure axial load strength

P_u = factored axial load at given eccentricity

Q = stability index = $(\Sigma P_u \Delta_1/(H_u h_g))$

r = radius of gyration of a cross section of a compression member

R = capacity reduction factor used in conjunction with the R-method (ACI 318-63) or; rotation matrix

R^T = transpose of rotation matrix

s = spacing between cracks

t = time under sustained load, days, or t = wall thickness

T = transformation matrix

T^T = transpose of transformation matrix

u = displacement in the x-direction

Δu = incremental displacement in the x-direction

$\Delta u'$ = incremental axial strain

U_L = local displacements

ΔU = incremental displacement matrix

ΔU_g = incremental displacements in global axes

v = displacement in the y-direction

Δv = incremental displacement in the y-direction

$\Delta v''$ = incremental curvature about the z-axis

w = displacement in the z-direction

w_c = unit weight of concrete

w_m = mean crack width

Δw = incremental displacement in the z-direction

$\Delta w''$ = incremental curvature about the y-axis

ΔX_b = incremental force in the direction of the x-axis at end B of member

y = distance to differential element in the y-direction of section

ΔY_b = incremental force in the direction of the y-axis at end B of member

z = distance to differential element in the z-direction of section; or shear span

ΔZ_b = incremental force in the direction of the z-axis at end B of member

Δ_1 = elastically computed first order lateral deflection due to total factored lateral force within a story (region) (neglecting $P\Delta$ effects) at top of story (region) relative to bottom of story (region)

Δ_2 = second order deflection lateral deflection

$$\Delta_2 = \frac{\Delta_1}{1 - \frac{\Sigma P_u \Delta_1}{H_u h_s}}$$

ϵ = strain

ϵ_c = concrete compressive strain

ϵ_m = mean steel strain

ϵ_o = concrete strain corresponding to maximum stress, usually $2f'_c/E_c$

ϵ_{ot} = 0.0001

ϵ_s = steel strain

ϵ_t = concrete tensile strain

ϵ_u = ultimate strain

$\Delta\epsilon$ = incremental strain

$\Delta\epsilon_o$ = increment in axial strain

ϕ = rotation

ϕ' = curvature

$\Delta\phi_x$ = incremental rotation about the x-axis

$\Delta\phi_y$ = incremental rotation about the y-axis

$\Delta\phi_z$ = incremental rotation about the z-axis

$\Delta\phi'_y$ = increment in rotational strain about the y-axis

$\Delta\phi'_z$ = increment in rotational strain about the z-axis

ρ = A_{st}/bh = steel percentage ratio

ρ''_h = volumetric ratio of hoop reinforcement to confined concrete

σ = stress

θ_x = angle measured from the x-axis

θ_z = angle measured from the z-axis

ν = Poisson's ratio

CHAPTER 1

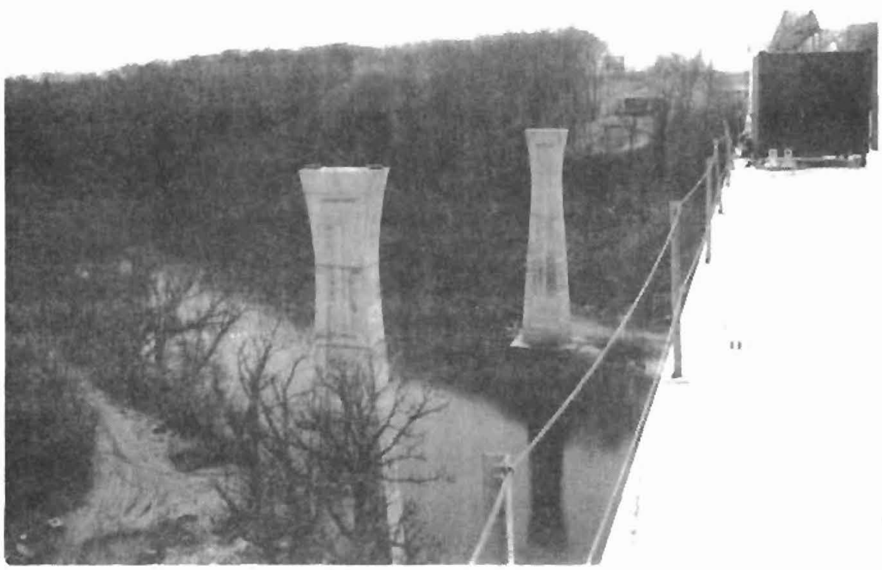
INTRODUCTION

1.1 Bridge Pier Design Trends

Current trends in bridge construction indicate increasing use of slender reinforced concrete compression members as bridge piers. This is primarily due to growing utilization of higher compressive strength concretes and higher yield point reinforcing steels along with a shift to more material-efficient factored load design procedures. In addition, increased sensitivity to bridge aesthetics has increased utilization of flared and tapered bridge piers such as those shown in Fig. 1.1. These slender, graceful piers have become particularly popular where access conditions for pier construction are restricted, as in canyons, heavily developed urban areas, and in deep reservoirs. Throughout the United States, virtually all bridge piers are being constructed of reinforced concrete. Responses in a recent survey detailed in Chapter 2 indicated very occasional use of prestressed concrete and of structural steel. The latter was used in a few railroad trestle applications but almost all piers reported were of reinforced concrete.

In very large bridge piers the dead load of the piers becomes substantial, increasing the foundation size significantly in many long span bridges. In order to reduce material quantities, many successful projects have utilized slender piers of hollow or cellular cross section to provide adequate stiffness and strength while reducing dead load. Savings can be quite substantial, depending upon the geometric configuration of the cross section. When a hollow cross section exceeds a width-to-depth ratio of around 3, the cost of extra formwork probably exceeds the material savings, but the

(a) In a busy highway exchange



(b) As a part of a river crossing

Fig. 1.1 Tapered bridge piers

reduction in pier dead load can still produce appreciable savings in foundation cost. Modern construction techniques such as slip-forming or segmental precasting are well-suited to hollow or cellular pier construction. Use of these methods substantially reduces formwork costs and makes hollow pier construction attractive.

In bridge pier design more attention must be given to the large movements which are possible due to nonload effects such as temperature changes than is usually true in building design. In continuous bridge systems there is a growing trend to minimize bearing problems by using more monolithic construction and allowing the large restraint forces that would be set up to be minimized by using more slender and, thus, restraint-relieving, piers.

Due to all of these factors, bridge designers are being more frequently challenged with the design of slender piers of more involved geometry.

1.2 Slender Pier Design Problems

Designers are facing increasing difficulty with bridge pier design as slender nonprismatic or hollow piers become more commonplace. Recent revisions to the AASHTO Specifications [5] base the design of slender concrete compression members on the column slenderness design procedures of the ACI Building Code [1]. These procedures require either a second order analysis considering realistic column stiffness values reflecting the influence of cracking, reinforcement, and time effects, or an approximate evaluation of slenderness effects using the "moment magnification" procedure. Initial application of these provisions to bridge structures upon adoption of the revisions to the AASHTO Specifications caused considerable confusion and frustration became many of the important concepts for application of the moment magnification procedure, such as effective length factors, were contained in the ACI Building Code Commentary rather than in the Code itself. The AASHTO provisions only incorporated the Code values and little guidance was given concerning the Commentary provisions. Subsequent familiarity with the Commentary provisions provided some general guidance for design of slender columns of prismatic cross section when no substantial percentage of voids is present in the cross section.

Even with the substantial explanatory material concerning the application of the moment magnification procedures contained in the ACI Building Code Commentary and background literature [14,26], the practical application of these procedures in tall pier design is extremely difficult and possibly inappropriate in some cases. The types of problems that exist in application of the slenderness provisions to design of tall piers are:

(1) The second order analysis and the more approximate moment magnification procedures are analysis methods and are time-consuming when used in design without suitable design aids or computer codes.

(2) The important empirical equations for column cross section stiffness were based on tests of solid prismatic column or pier specimens. Their validity and applicability for use with tapered or hollow sections has not been verified.

(3) While some analytical procedures are available which can extend the basic thrust, moment, curvature relationships to irregular sections, the general limits and assumptions for such extensions need to be verified experimentally for a range of cross sections, void ratios and configurations, and reinforcement percentages and arrangements typical of tall pier construction.

(4) All basic parameters in the procedures were set based on consideration of typical building values, not bridge parameters.

(5) The AASHTO provisions are based on the ACI Building Code provisions which focus on strength and design to provide proper moment capacity. These provisions do not clearly focus on lateral

displacement, although they consider such displacements in the moment magnification. In typical multistory building construction, the story drifts are small and the magnitude of the lateral displacements is not a primary criterion. In contrast, the design of slender bridge piers must consider the tip displacement and the interactive effects on bearing and joint design. Elliot [27] and Leonhardt [28] have both pointed out the critical importance of assessing and controlling deflections of bridge supports in their evaluation of bridge damage resulting from the San Fernando earthquake. Application of the current AASHTO moment magnification procedures does not clearly alert the designer when a deflection problem exists.

The background for the ACI Building Code provisions for slender column design which form the basis for current AASHTO Specifications is summarized in Ref. 14. The extensive experimental and analytical studies which formed the basis for these provisions considered only solid, prismatic columns with concrete cover, bar arrangements, and cross section shapes typical of those found in buildings. Studies completed since the adoption of these procedures have indicated that the procedures are well-suited for analysis for the conditions envisioned, but need verification in many other cases. The specific case of hollow wall box or cylindrical piers is far removed from the cross sections previously checked. The procedures should be evaluated for these type cross sections.

Recent studies [29,30,31] have indicated that design of slender compression members can be accomplished more efficiently by the use of second order analysis procedures than by use of the approximate moment magnification procedures. The basis for these procedures is very similar to the current provisions but they are considerably more efficient to implement on digital computers when design programs are developed. These procedures have the further advantage that deflection values can be obtained as well as strength values and the design required to meet strength and/or deformation limits.

The 1976 and 1977 Regional Meetings of the AASHTO Bridge Committee indicated concern over design procedures for the design of nonprismatic bridge piers, related to the definition of unsupported length, the effect of cracking and reinforcement on relative stiffness, the effects of fixity, and the establishment of a suitable effective length factor for nonprismatic columns. Concern was also expressed over the lack of guidance in the Specifications regarding the stiffness of individual elements of hollow columns, the required amount of ties to be used in hollow columns, and the minimum amount of reinforcement to be supplied.

Discussions indicated that the primary hope for solution of these concerns was the development of a comprehensive computer program to assist the design process. The State of Washington undertook development of a column computer program, but reported to AASHTO that it was not operational and that there were reservations about the program which needed investigation by qualified persons and the program required verification. Examination of the program guide "Deflection and Buckling of Prismatic and Nonprismatic Columns," as circulated in the 1977 AASHTO Bridge Subcommittee Regional Meeting Agenda, indicated it to be a generalized elastic buckling solution based on iterative recursive procedures. It gave no guidance for inputting important parameters such as concrete stiffness, effect of cracking, effect of duration of loads, effect of reinforcement, or realistic effect of voids. This program could not be considered an answer to the needs indicated above.

1.3 Objectives of the Research

The primary objectives of this study were:

(1) Develop design-oriented computer codes for slender concrete bridge piers considering the influence of axial loads and variable

moment of inertia on member stiffness, effects of deflections on moments and forces, and the effects of duration of loads. Throughout, the procedures are to consider typical bridge applications, including tapered and hollow bridge piers.

(2) Verify those components of the design method which have not previously been authenticated. These include stiffness of hollow cross sections and the applicability of the basic assumptions to solid and hollow sections subjected to biaxial loadings.

(3) Verification of computer codes by detailed checks with a wide range of experimental results.

(4) Suggestion of improvements in approximate design methods for slender bridge piers under the AASHTO specifications.

Detailed information on satisfaction of Objectives 2 and 3 are presented in a companion report [18] on this project.

1.4 Report Contents

A comprehensive survey of the current state-of-the-art in usage of concrete bridge piers was undertaken. Results of a survey which gathered detailed information representing over 150,000 concrete bridge piers is given in Chapter 2. A discussion of design requirements for slender piers is given in Chapter 3 which includes recommendations for desirable revisions in current AASHTO Specifications. A detailed treatment of the basic fiber model selected as the key element in development of a general second order analysis program is given in Chapter 4. Chapter 5 contains the basic information on three computer codes developed for general analysis of pier bents. Program BIMPHI is of use mostly in a research context for evaluating section moment-curvature relationships. Program PIER is a generalized analysis for single pier bents which can treat hollow sections and piers which have variable cross sections along the column height. Program FPIER is a generalized analysis for multiple pier bents which can handle the same type of variable cross sections as program PIER. Testings, input guides and sample input and output are given in Appendices A through C for these programs. Verification of the programs is given in a concurrent report [18]. Chapter 6 summarizes the conclusions and recommendations from the overall study.

CHAPTER 2

STATE OF ART SURVEY

2.1 Introduction

The initial phase of this study concentrated on gathering data concerning usage and design considerations for concrete bridge piers as well as the views of major users concerning possible trends in bridge piers over the next decade. A comprehensive questionnaire was developed to solicit input on usage and was sent to the bridge engineers of all states and provinces as well as to a number of prominent consulting engineering offices, railroads, and municipalities. A sample questionnaire is included in Appendix D.

The response to the questionnaire was very good. Almost forty organizations returned completed questionnaires and a number submitted supplemental information in the form of typical plans. Detailed information was received from 24 states, 2 provinces, 6 consultants, 2 railroads, 2 cities, the Federal Highway Administration, and 1 foreign bridge ministry. The comprehensiveness of the study can be seen from the number of piers covered. The respondents reported that during the period 1960-1980 the trends indicated were determined from surveys of 155,000 actual pier bents that were designed and constructed. They further estimated that during the next decade they would construct 60,000 additional pier bents. In general, anticipated usage in most variables mirrors usage in the past two decades. Actual usage results will be the basis for all figures unless otherwise indicated.

2.2 Overall Geometrical Configuration

As can be seen from Fig. 2.1, the predominant type of bridge pier usage is in multiple pier bents, with isolated or single piers

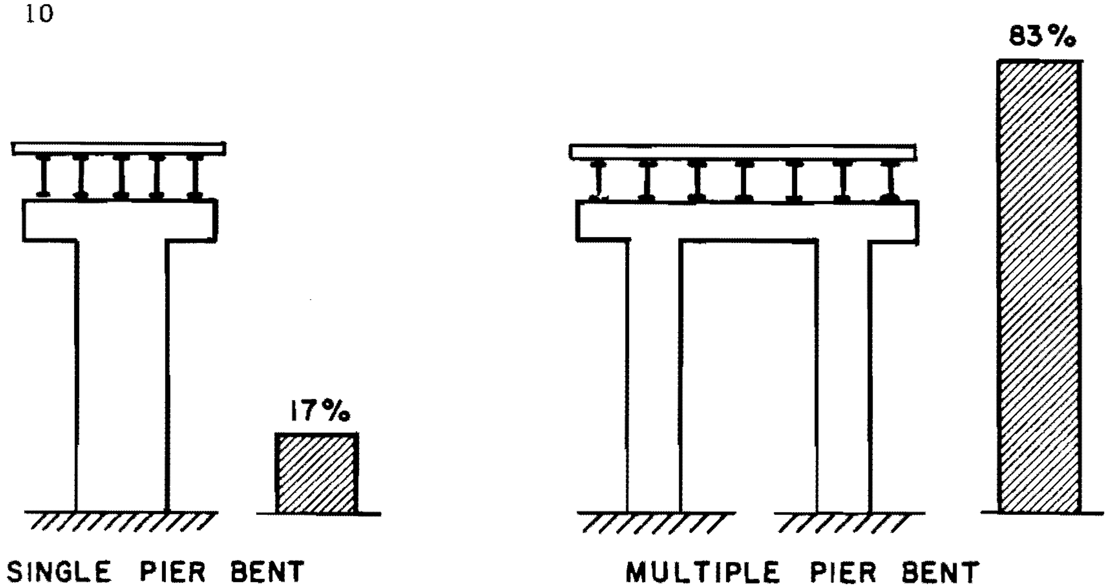


Fig. 2.1 Bent types

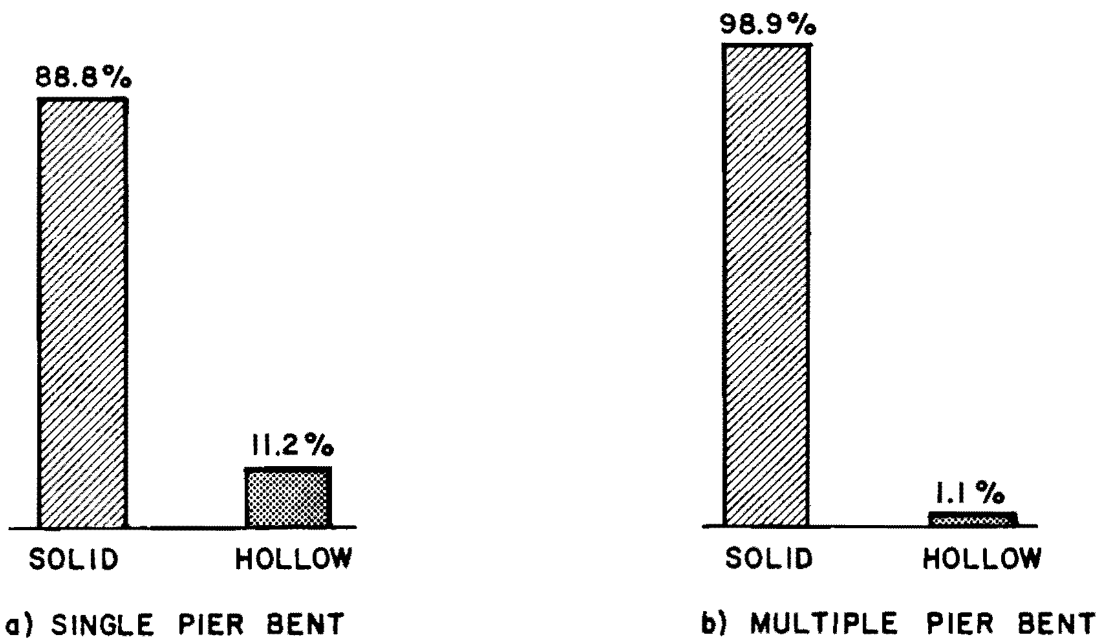


Fig. 2.2 Cross section solidity

occurring in about 1 out of 6 cases. Respondents anticipated a somewhat higher usage of single piers in the future with a prediction of use in about 1 out of 5 cases. Similarly, there is an overwhelming use of solid cross sections. Figure 2.2 indicates that cross sections in multiple pier bents are predominantly solid although there is a much higher percentage of use of hollow cross sections in single pier bents. Figure 2.3 indicates that solid cross sections dominate in lower height applications and that usage of hollow cross sections increases with pier height. The survey indicated that 81% of the piers built from 1960 to 1980 with a height greater than 100 ft were of hollow cross sections. Thus, analysis procedures for slender piers should definitely consider hollow cross sections even though relatively few such cross sections are found in shorter piers.

Seven different states reported the use of hollow piers. Numerous uses of such piers have been reported in foreign applications and this use will be touched on in later discussion. The general distribution of pier cross-section type is shown in Fig. 2.4 for both single pier bents and for multiple pier bents. Again, it can be seen that the hollow cross sections occur most often in single pier bent applications.

A wide variety of pier vertical configurations was reported in the survey. The types of piers are shown in Fig. 2.5 for single pier bents and in Fig. 2.6 for multiple pier bents. In each figure the percentage distribution of all single or multiple pier bents in a given height range are shown. For example, Fig. 2.5 indicates that 82.3% of all single bent piers of less than 30 ft elevation are type A which is a constant dimension shaft along its entire height. Similarly only 3.5% of single bent piers of less than 30 ft elevation are type D which is a stepped pier. Note that the sum of all the percentages for the bars with L < 30' for types A through I total 100%. Thus the bar charts give an indication of usage of various vertical configurations for four different height

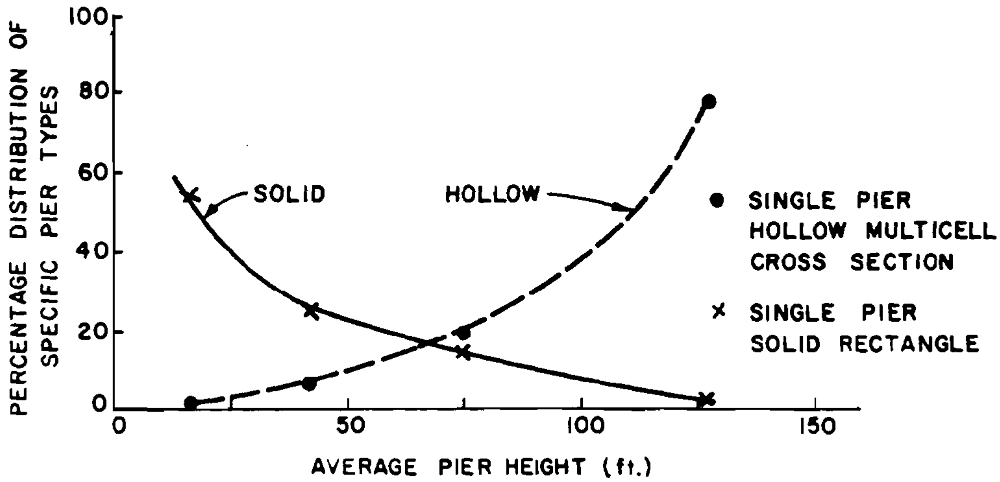


Fig. 2.3 Solidity-height relationship

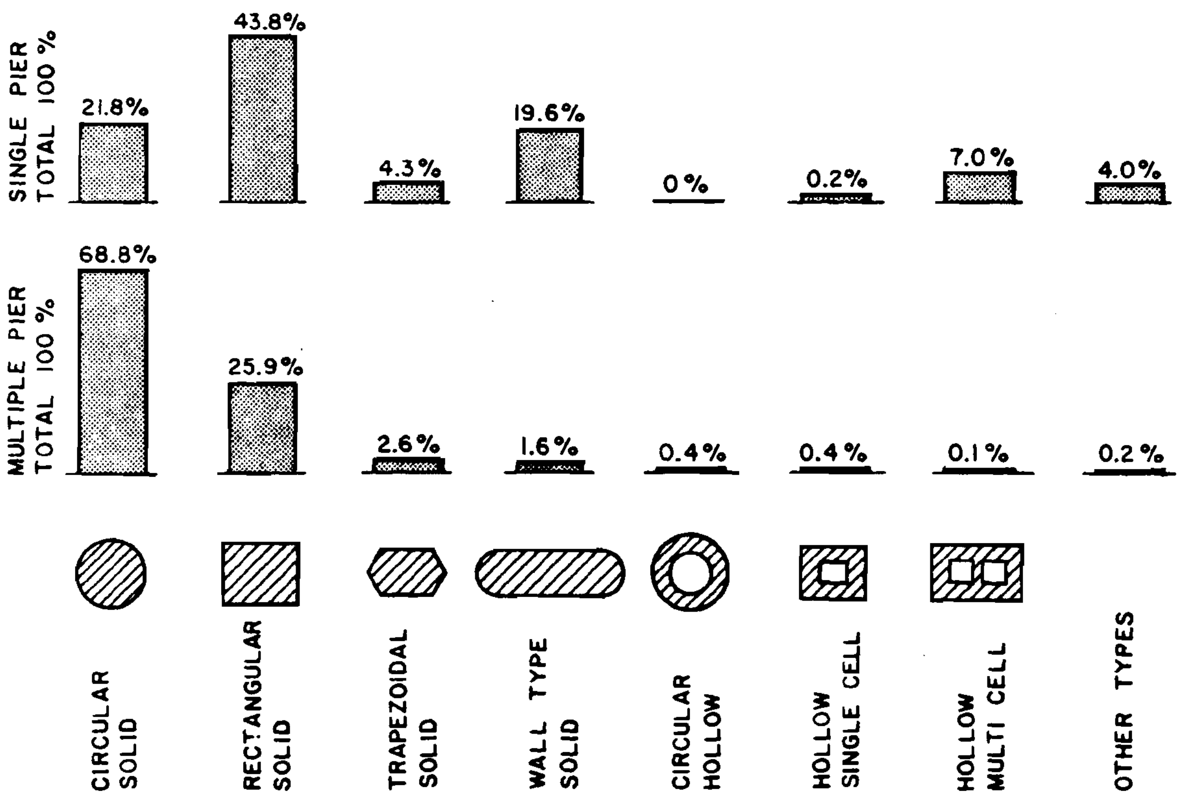
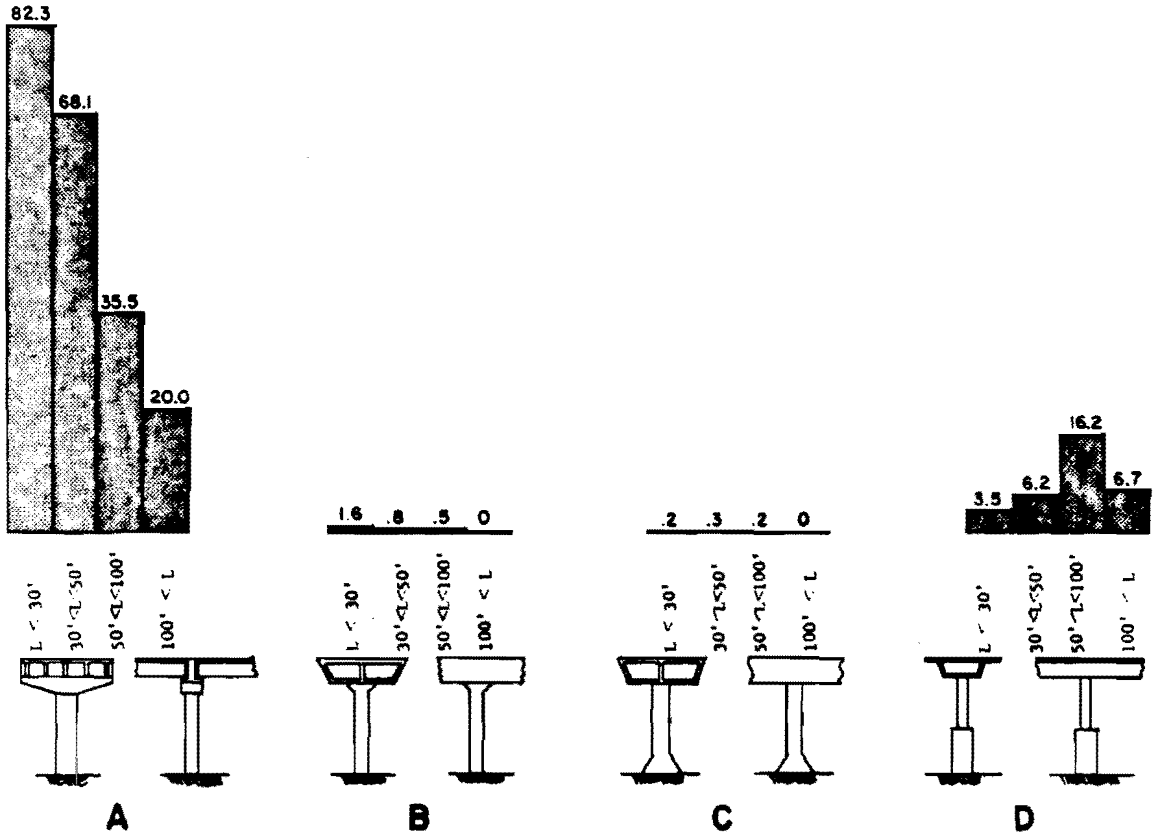


Fig. 2.4 Cross section types

ranges. The category "other" represents piers which in almost all cases had linear variations in both width and depth. These piers were almost always larger at the base than at the top.

Figure 2.5 indicates that a wide variety of vertical configurations are in use for single pier bents. For pier heights above 50 ft only a small minority of piers have constant dimension shafts. For the moderate height piers (50' < L < 100') almost 38% are types F or H having linearly varying cross sections while another 16% are stepped. This trend is even stronger for the tallest piers (L > 100'). Over 70% of the tallest piers have variable cross section dimensions (types F, H and others) while only 20% are constant section. A relatively small number (6.7%) are stepped. Relatively few single pier bents have flared profiles or taper inwards towards the bottom (types B, C and G). These results indicate that comprehensive design programs for tall single bent piers must be able to treat vertical variations in cross section for linearly tapering, flared, and stepped cross sections. This is possibly even more important than handling prismatic constant cross section piers since these types tend to be most widely used in lower elevation applications where slenderness effects are less pronounced. This reinforces the bridge designer complaint that the present approximate procedures in AASHTO Specifications for slenderness effects need to be able to handle varying cross sections. This variation is much less frequent in building applications for which the present AASHTO approximate design methods were originally developed. This was a main reason for the general development of program PIER which will be reported in detail in Chapter 5.

Figure 2.6 indicates that the shorter ranges of multiple pier bents (L < 30' and 30' < L < 50') have a much higher percentage of constant cross section shafts than the single pier bents did. Above L = 50 ft, the stepped shaft configuration (type D) plays a much more important role. These represent stepping of the shaft at a lateral


82.3
68.1
35.5
20.0
1.6
.8
.5
0
.2
.3
.2
0
3.5
6.2
16.2
6.7
A
B
C
D

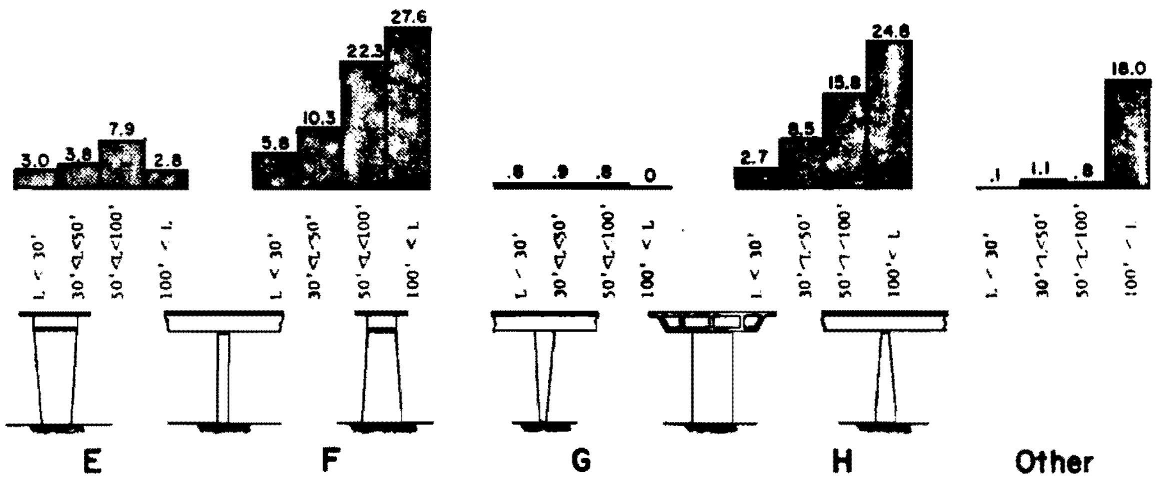
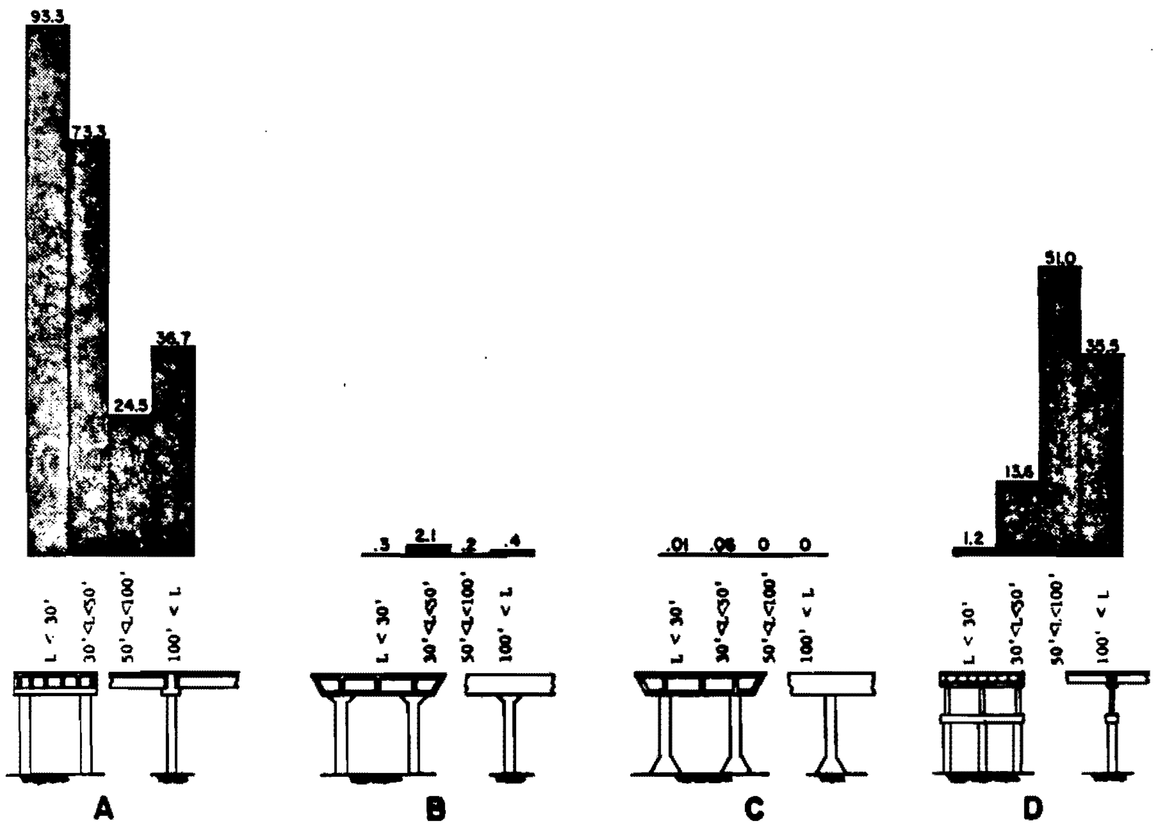


Fig. 2.5 Single pier bent vertical configuration



93.3
73.3
24.5
36.7
.3
2.1
.2
.4
.01
.08
0
0
1.2
13.6
51.0
35.5
L < 30'
30'<L<50'
50'<L<100'
100' < L
A
B
C
D

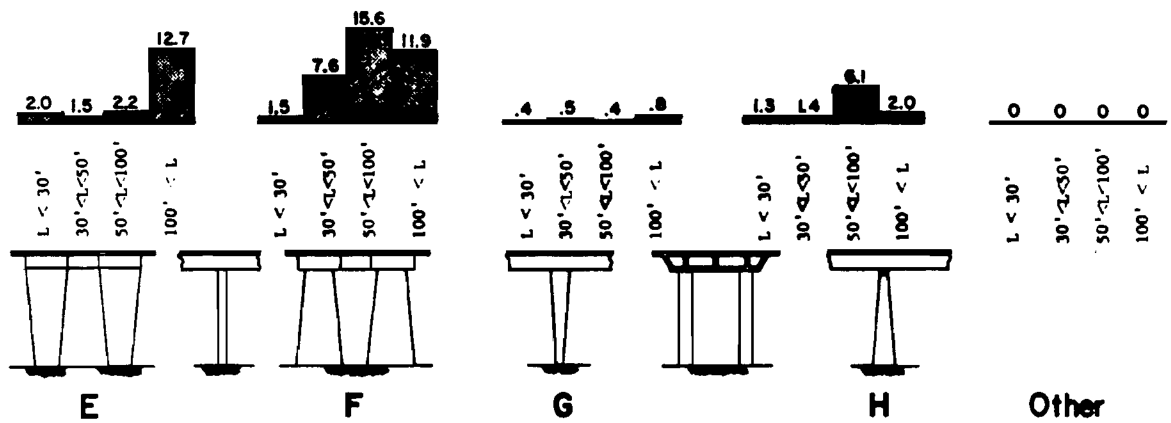


Fig. 2.6 Multiple bent pier vertical configurations

bracing beam level in almost all cases. Note that type D is used in a majority of cases for 50 ft < L < 100 ft and about 35% of the cases when L > 100 ft. In the tallest multiple pier bents relatively few (2%) have variations in depth (type H) as compared to the wide usage of this type in tall single pier bents (25%). When variations of cross section are used in multiple pier bents they tend to be in the width (types E and F). However, the survey results clearly indicate that if a comprehensive analysis program for multiple pier bents is limited to constant cross section members or stepped size members with the steps only at the level of interconnecting bracing beams, it would only be able to handle 52% of the bents with 50 ft < L < 100 ft and 27% of the tallest bents with L > 100 ft. Thus, even for multiple pier bents the general analysis program should be able to handle variable cross section. This is why the program PIER which can treat variable sections was chosen for expansion into program FPIER as detailed in Chapter 5.

2.3 Pier Slenderness

All of the piers reported were classified according to their overall height into four general ranges: 0-30 ft, 30-50 ft, 50-100 ft, and over 100 ft. General height distributions for all piers and then separately for solid and hollow piers are shown in Fig. 2.7. As mentioned previously the vast majority of piers fall into the below 30 ft range. However, there are almost 10% of all piers in the over 50 ft ranges and these involve over 70% of hollow pier applications. Approximately 15% of the piers in the over 50 ft height ranges are hollow with the predominant number being in the over 100 ft range where over 80% of the piers are hollow.

However, pier height is not in itself as meaningful a parameter as pier slenderness ratio, defined herein as the pier height, L, divided by the radius of gyration of the cross section, r. The L/r ratio is an indication of a basic structural parameter which

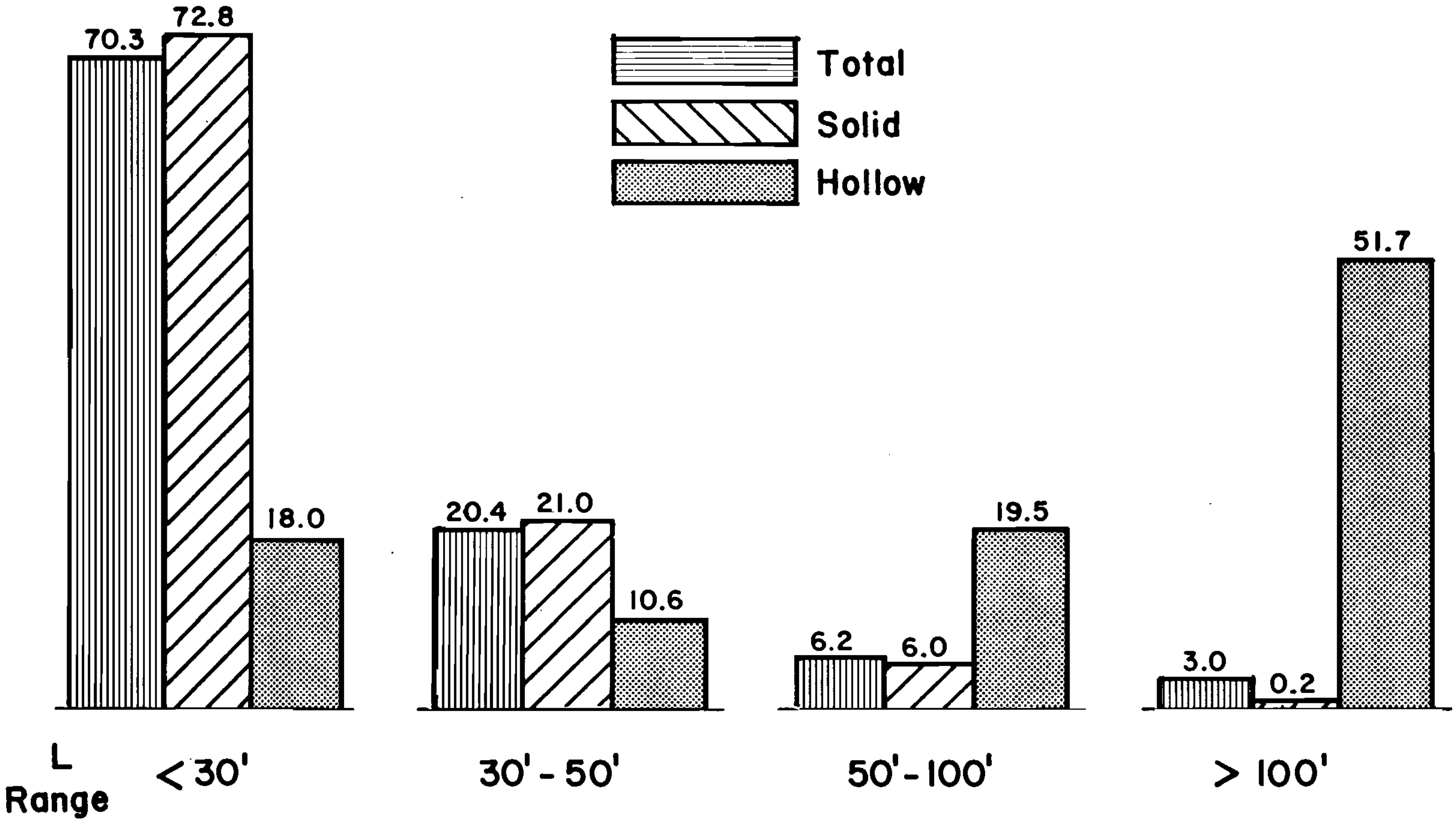


Fig. 2.7 Pier height distribution

governs the secondary moment characteristics and pier instability. The actual ratio of interest is $k\ell_u/r$ where k is the effective length factor which depends on degree of restraint to both lateral displacement and rotation at the ends and ℓ_u is the unsupported height. It was felt that it was asking too much of respondents to the survey to indicate such precision in slenderness ratio and so only gross L/r was asked for. In most American bridge applications little positive restraint against translation parallel to the bridge axis is provided at the top. In addition, little rotational restraint is provided at the top of piers and the base is far from perfect fixity. In these conditions k can range from 1.5 to 3.0. Thus, the L/r values must be looked at critically. The approximate analysis procedure for slenderness of AASHTO Spec. Art. 1.5.34(B) is for ranges of 22 to 100. Below $k\ell_u/r$ of 22 (say in the present case L/r of about 10) there is no need to check slenderness as secondary P-Δ moments will not be significant. Above $k\ell_u/r$ of 100 (say in the present case L/r of about 40) the moment magnification approximate calculation procedure is not verified and a comprehensive second order analysis must be used.

The distribution of the ratio L/r reported by the respondents is shown in Figs. 2.8 and 2.9 for single and multiple pier bents respectively. Fig. 2.8 shows several interesting aspects of single pier bent slenderness. In single piers the proportioning of the more slender hollow columns seems to be quite conservative. Note that 91% of these columns fall in the L/r range of 20 or less. Since it has been shown in Fig. 2.7 that hollow piers are frequently used in quite tall applications, this means that such sections are sized quite conservatively to actually reduce slenderness ratios. The movement of material towards the outer fibers in hollow sections naturally increases r and makes such sections more efficient. Note that in single pier bent applications about one third of all cases would seem to fall above the $k\ell_u/r$ = L/r = 40 level which represents

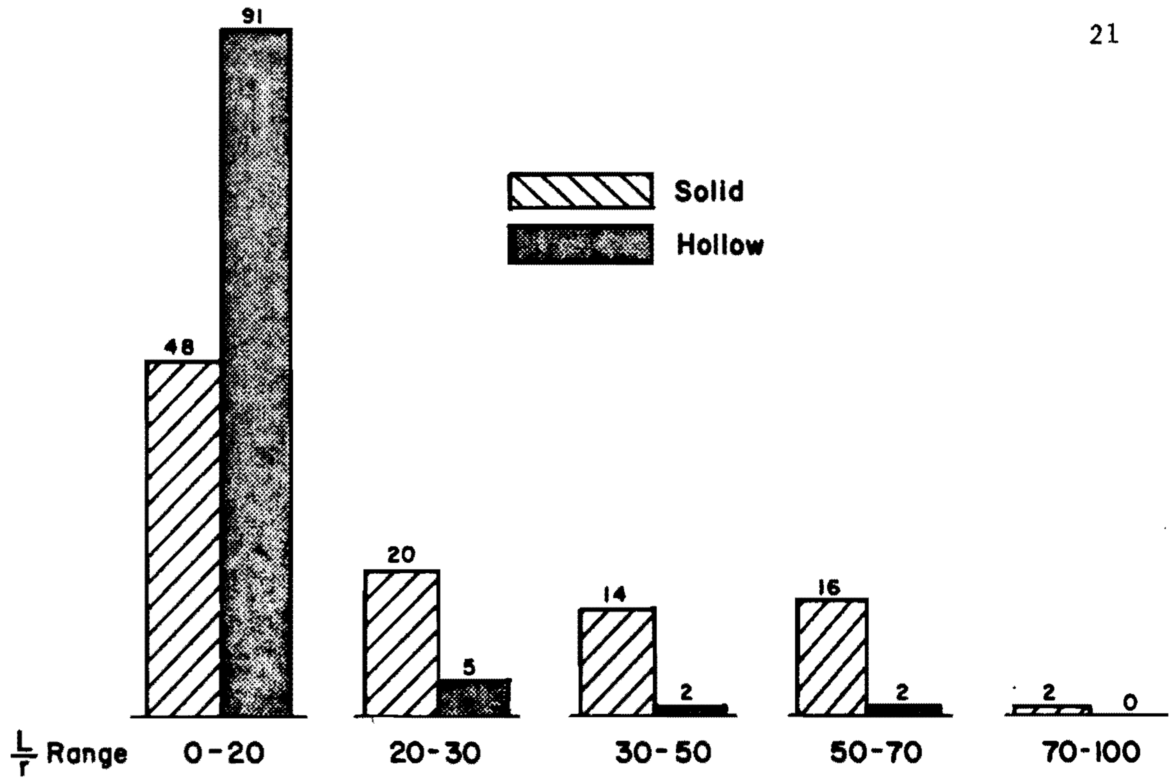


Fig. 2.8 L/r ratio parallel to bridge axis, single pier bents

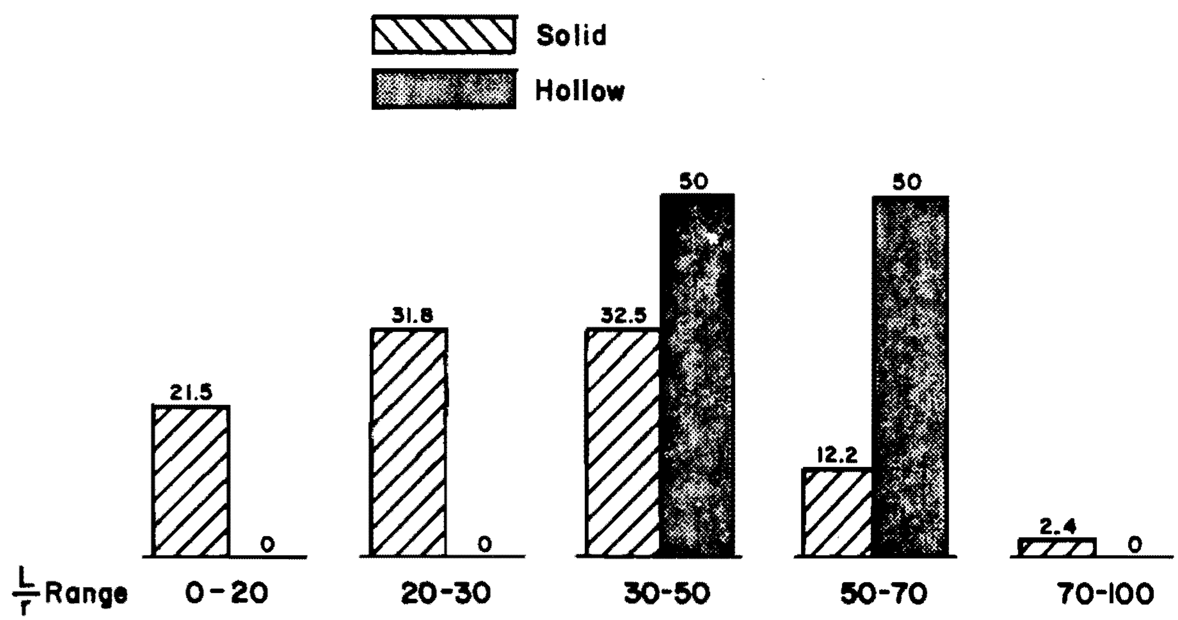


Fig. 2.9 L/r ratio parallel to bridge axis, multiple pier bents

a rough upper bond for the moment magnification method. This indicates the need for a general analysis procedure for a large number of cases. Figure 2.9 is harder to interpret, since the large number of stepped columns or multiple level multiple pier bents makes ℓ_u very difficult to relate to L. Since many of these cases are multiple levels, the L/r ratios would be divided by 2 or 3 to give L_u/r ratios. This probably explains the greatly higher slenderness apparent for multiple pier bents when compared to single pier bents. Again, the high slenderness ratios, even if reduced appreciably for k and ℓ_u corrections, indicate the need for a general analysis.

The ratios perpendicular to the bridge axis differed appreciably and were generally of lower slenderness. Figure 2.10 compares the overall totals (both single pier bents and multiple pier bents) for L/r ratios perpendicular and parallel to the bridge axis. It can be seen that lower ratios of L/r perpendicular to the bridge axis show more frequency of occurrence in almost all cases.

Figure 2.11 illustrates the same variables except that these are the anticipated future usages in the 1980-1990 decade as seen by the respondents. Note that there is a decided shift towards higher slenderness ratios ($L/r > 30$) when compared to the present usage shown in Fig. 2.10. This is one of the areas of the survey where the respondents clearly indicated an anticipation of a change in practice. This reinforces the need for development of a general analysis program.

2.4 Pier-Superstructure Connections

Several questions probed the usage and trends of various connection types. The survey asked respondents to categorize usages of various types of connections with respect to variables such as: pier type (single pier bents vs. multiple pier bents); pier height (less vs. greater than 50'); span length (less vs. greater than 100'); and continuity (simply supported vs. continuous spans). Only the latter variable seemed to cause any significant difference in connection type.

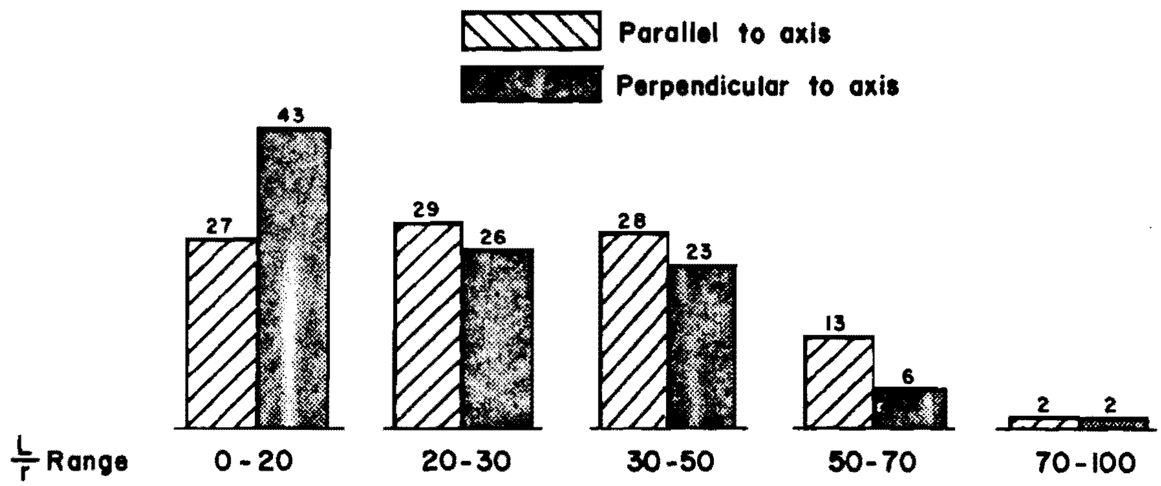


Fig. 2.10 Reported 1960-80 usage of slenderness ratios, L/r, all bridges

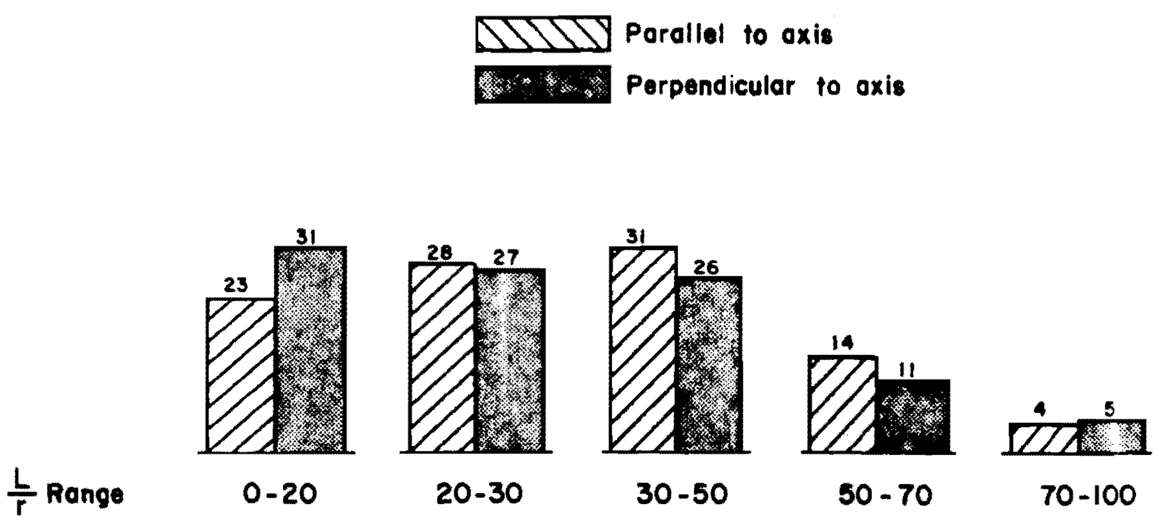


Fig. 2.11 Anticipated 1980-1990 usage of slenderness ratios, L/r, all bridges

The distribution of connections reported is shown in Fig. 2.12. Current usage seems to favor elastomeric bearing pads, sliding bearings, and steel shoes for simply supported span applications. In continuous spans the use of steel shoes increases slightly but it appears that monolithic joints and pot bearings replace elastomeric pads and sliding bearings in many cases. There were a number of comments which indicated that this is a rapidly changing area and that current trends are towards increased use of more sliding Teflon or Fabrika bearings in place of stainless steel bearings in spans less than 100 ft. Similarly, a trend towards higher use of pot bearings for longer span bridges was predicted.

The relatively low percentage of monolithic joints in continuous spans (15%) indicates U.S. practice is not following a number of European countries where wide use of monolithic pier and superstructure construction is being used.

2.5 Bent/Foundation Types

Respondents were asked to estimate the degree of usage by their organization of the bent-type foundation classification shown in Fig. 2.13. Estimates were made on the basis of pier height, L, in four categories of height. For each type bent foundation it was possible to indicate whether the bent was either carried on a spread footing or raft-type foundation or was carried on piles or piers. In all four height categories approximately 1/3 of the piers were carried on spread footing or raft foundations while approximately 2/3 of the piers were carried on piles or pier foundations. This indicates a need for analysis procedures which can model very different types of foundation stiffness and restraint.

The results shown in Fig. 2.13 show the percentage distribution of pier bent/foundation types by pier height classes. In any pier height classification (i.e., 30' to 50') the overall total of all bent-foundation types was considered as 100%. Thus the bars in

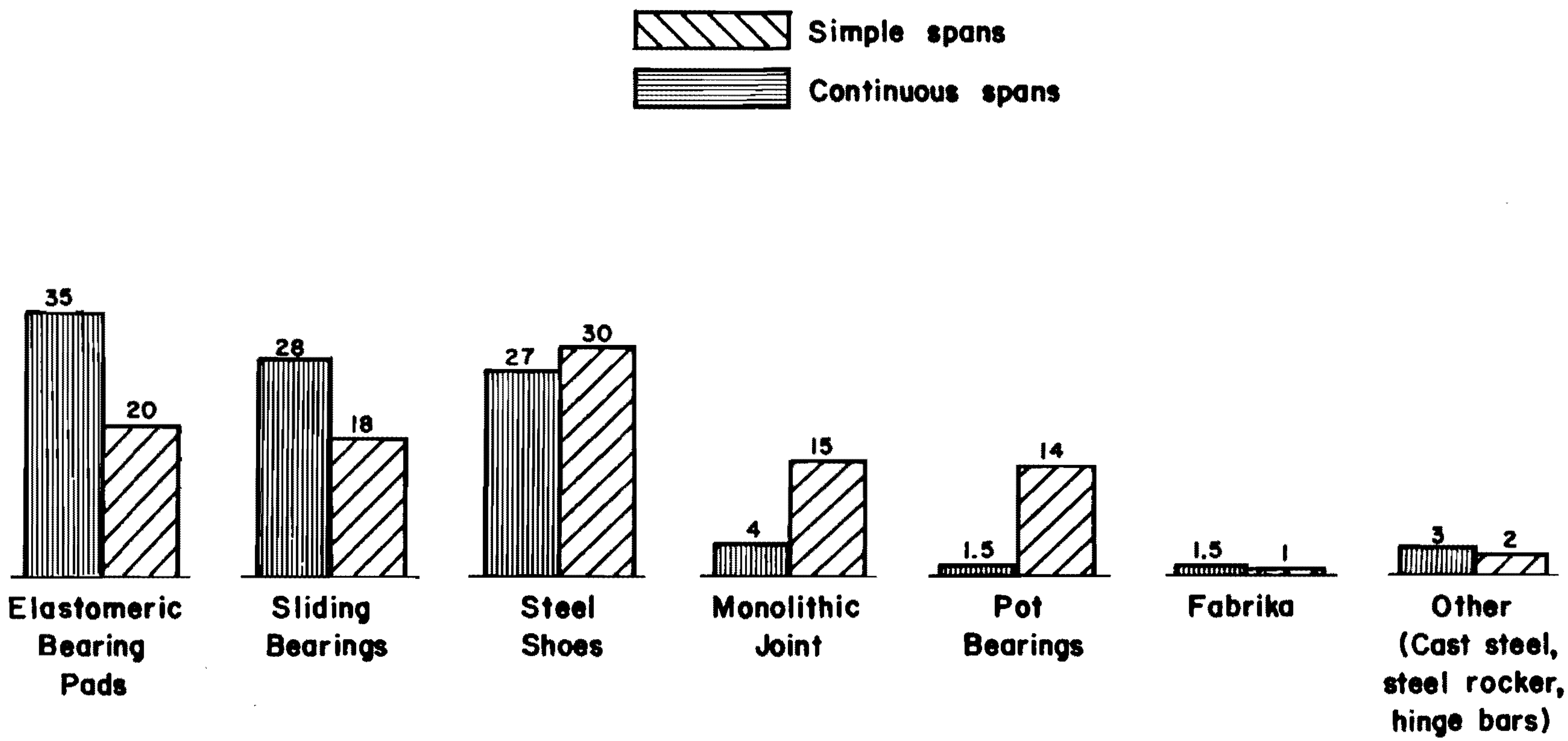


Fig. 2.12 Pier-superstructure connections

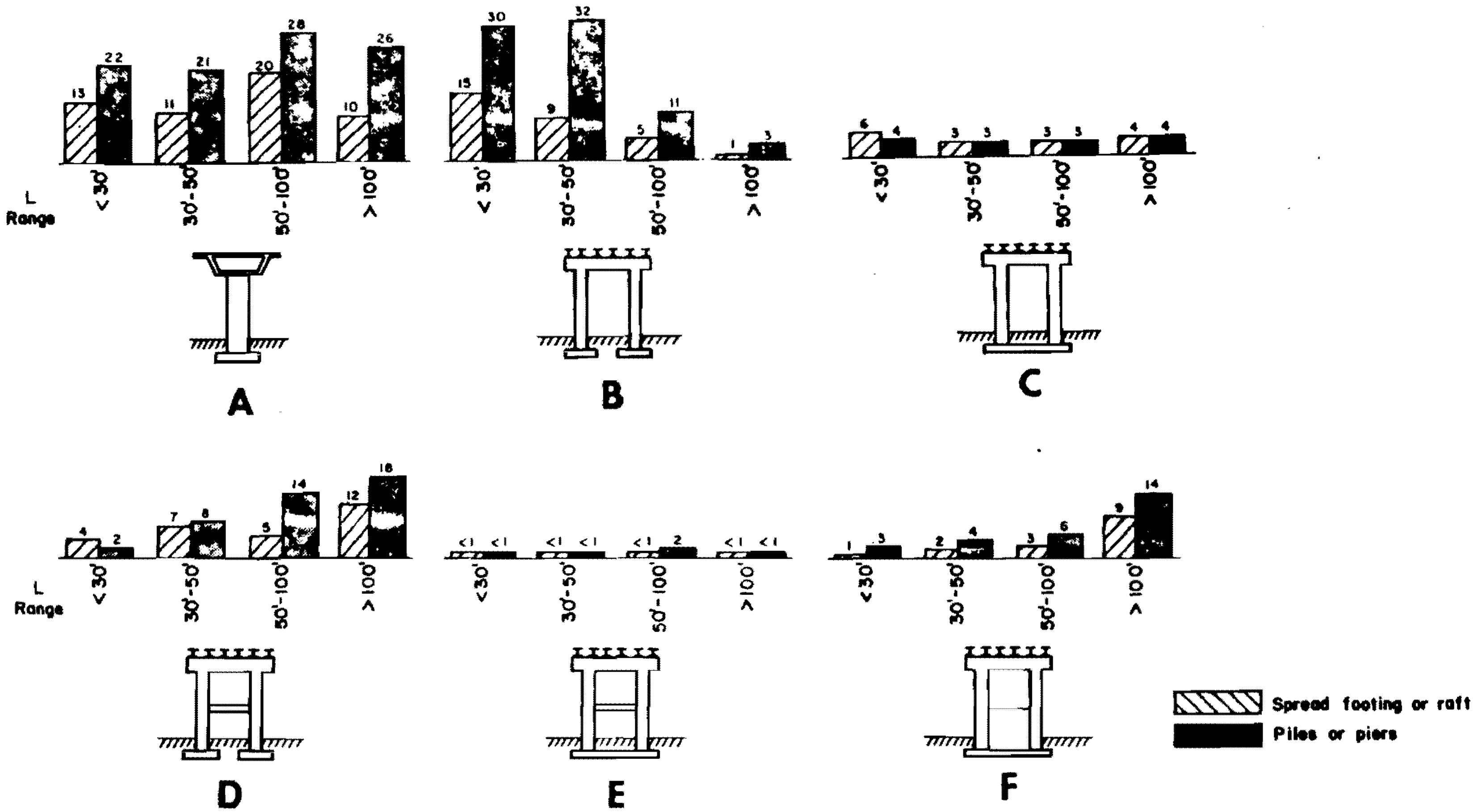


Fig. 2.13 Bent/foundation types

each height category total 100% when types A through F are summed for both spread footing and pile foundations. Type A seems to be the overall widest used type and is used in all height ranges but dominates in higher piers. Type B is very widely used in the lower pier height ranges. Types D and F are found in higher height ranges but type E seldom occurs.

Respondents from the Tokyo Metropolitan Expressway Commission indicated a surprisingly large number of very different bent configurations in their congested urban construction areas. These are shown in Fig. 2.14 as types G through L. These very special configurations provide a good deal of challenge to analysis program developers. The nonsymmetry of types G, K and L would place them outside the range of the programs developed in this study.

2.6 Load/Effects

Respondents were asked to indicate the relative importance of the following load/effect categories on tall concrete bridge pier design:

Structure dead load
Horizontally curved structure dead load
Unbalanced line load
Vehicle braking loads
Live load centrifugal forces--curved bridges
Temperature variations
Shrinkage and creep
Foundation settlements and lateral movements
Foundation rotational stiffness
Pier stiffness at various load levels
Deflection limits
Wind perpendicular to superstructure
Wind parallel to superstructure
Wind on line load
Ice/snow loads
Stream ice/current loads
Seismic loads
Out of plumbness due to construction tolerances
Construction forces
P-Δ moment, slenderness effects
Biaxial bending effects

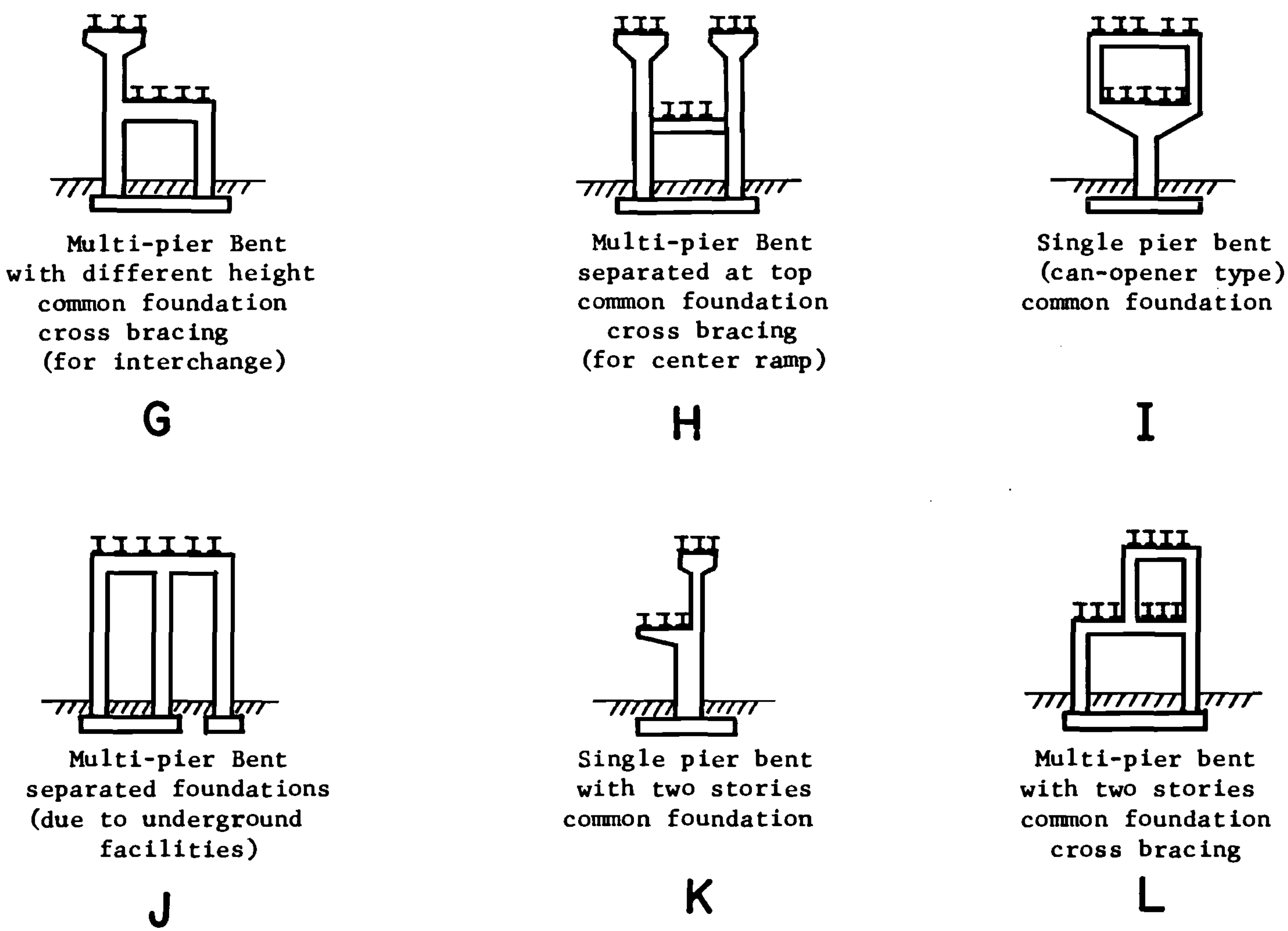


Fig. 2.14 Different bent configurations

From this overall broad list the general consensus of the respondents was that the critical load/effects which tended to govern pier design were dead load, unbalanced live load, wind, seismic load, and/or stream ice and current, depending on geographical location, and biaxial bending effects. Most respondents indicated that the P-Δ moments or slenderness effects were quite important in piers over 50 ft in height but were not generally critical in piers less than 50 ft in height. Forces occurring during construction were indicated to be important for particular types of construction such as precast and cast-in-place cantilever construction. In general, more loads and effects were indicated as critical in taller piers. Several respondents suggested some drift limits should be introduced for very tall piers.

The results indicated that a comprehensive analysis program should have the ability to handle biaxial loading, slenderness (P-Δ) effects, and a wide variety of loading cases. These requirements had much to do with the analytical procedures selected.

2.7 Load Combinations

Respondents were asked to indicate the relative importance of each of the AASHTO Load Factor Design Groups on tall concrete bridge pier design. Most respondents indicated that they always found it necessary to check Load Group I with $\beta_d = 1.0$ and Load Group III with $\beta_d = 1.0$. They indicated that they are always critical groups in design. For increasing span lengths (span length $>$ 100-150') and for increasing pier heights ($L > 50'$) then it was indicated that Load Group II with $\beta_d = 1.0$ and $\beta_d = 0.75$ and Load Group III with $\beta_d = 0.75$ tend to become critical. Respondents from seismic areas indicated Load Group VII is often critical. Most other groups were not usually critical although isolated cases were indicated where particular conditions had required that a given case should be considered.

The survey results indicated that input to a general analysis program should include both load and deformation effects. It also indicated that in many cases it would be desirable to check several Load Groups or combinations. Because of this alternate input modes were selected for the programs. One input mode allows the user to input specific loads or deformations for a single run. The other input mode allows the user to indicate all possible loads and their basic classification (dead, live, wind, etc.). The user can then indicate which Load Groups are to be checked and the program applies the required load factors and selects the combinations for the various groups.

2.8 Prestressing of Pier Bents

A pair of survey questions asked for information regarding usage of prestressed concrete pier bents. Only three of the respondents indicated any use of prestressed concrete (other than prestressed concrete piles below the piers). In all these cases the use of prestressed concrete was indicated to be very limited (less than 1% of all piers even in those organizations). Several respondents indicated they were giving some consideration to prestressing pier bents where they were very tall or had very high transverse loading. Since the response was so low, the analysis programs were developed for nonprestressed concrete only.

2.9 Detailing

A wide variety of questions were asked concerning detailing practices. The information supplied by respondents included both answers to the questions and submission of a wide variety of plan sheets. No clear consensus was possible in most areas since the size, shape, and complexity of the structures was so variable. However, a number of areas indicated trends which can be reported for information, but with a caution that wide variations were generally experienced.

In overall cross section dimensions the minimum width for solid cross section single piers ranged from 1.5 ft to 13 ft. The larger values (greater than 6') corresponded to wall-type piers. Common values were in the range from 2.5 ft to 4.5 ft. Hollow sections were generally in a smaller range (2' to 8'). The minimum widths suggested for piers in multiple bents were in a much tighter range from 1.5 ft to 3 ft. Minimum depths of solid single piers ranged from 1 ft to 4 ft while for hollow single piers it ranged from 1 ft to 8 ft. Again in multiple pier bents a much smaller range of minimum depth from 1.5 ft to 3 ft was reported.

In most questionnaire responses the minimum percentage of vertical reinforcement was suggested as 1%. However, there was very considerable disagreement on the maximum percentage. Figure 2.15 indicates the distribution of opinions. While the largest number favored the traditional limit of 8%, half of the respondents favored 4% or less. Considerable discussion by contractors questions the constructability of compression members with more than 4% reinforcement. It was quite interesting and informative to study the actual designs submitted along with the questionnaires. A wide number of these were checked for actual steel percentages. It was found that while minimum values ranged from 0.7 to 1% there were virtually no cases of maximum values greater than 4%. This suggests that the maximum limit might be reduced to 4% to improve constructability with no serious penalty to actual designs.

Spacing limits for vertical reinforcement also showed wide differences of opinion between respondents. Minimum spacing values used for solid and hollow cross sections ranged from 1-1/2 in. to 6 in. with the majority of respondents favoring 4 in. Several favored expression in terms of bar diameter. A number of respondents favored 3 to 3-1/2 d. Maximum spacing limits showed more difference in opinion with values from 6 in. to 18 in. Almost equal numbers favored 12 in. and 18 in.

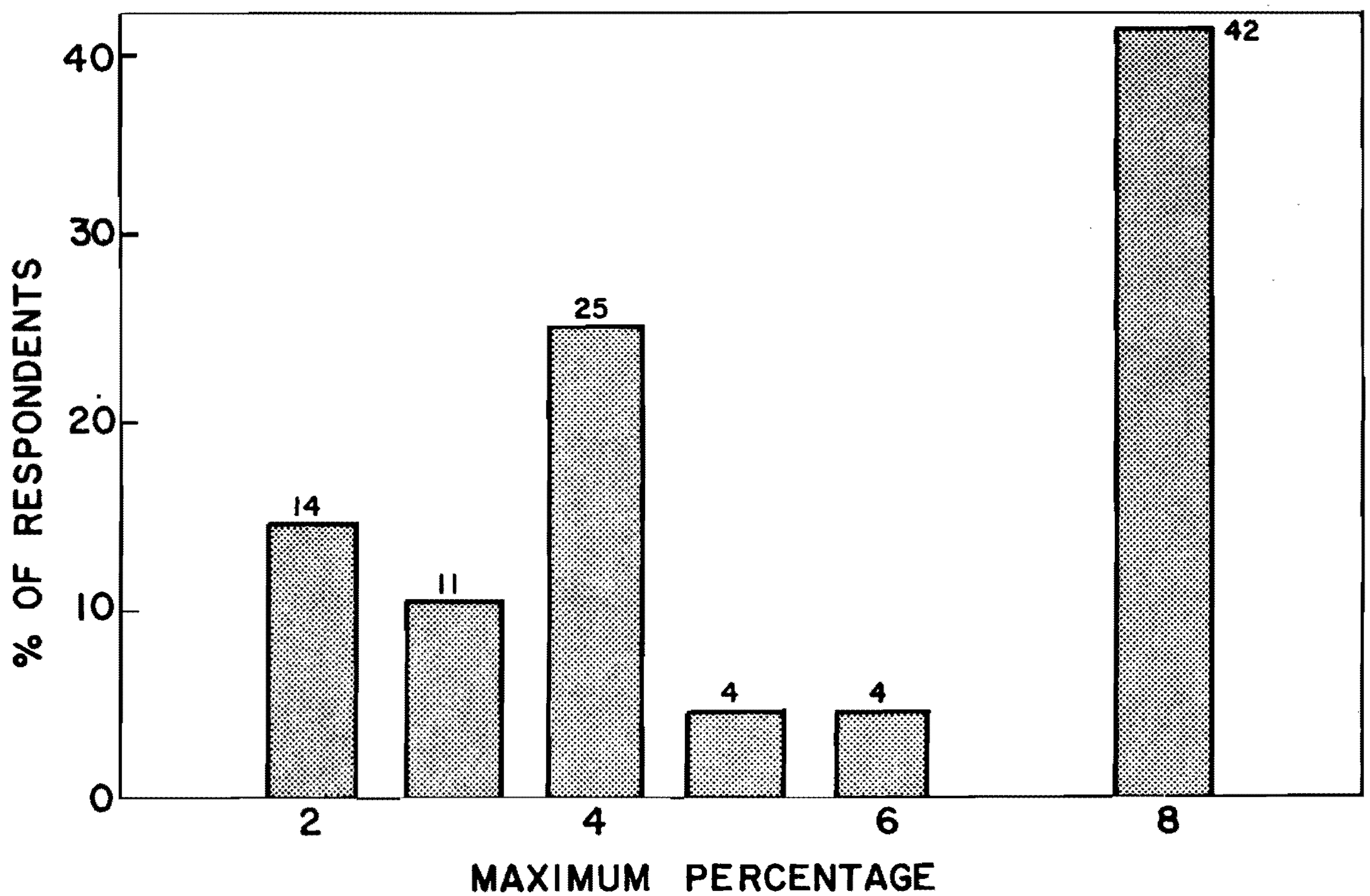


Fig. 2.15 Maximum longitudinal reinforcement limit variations

A wide variation of opinion was given considering the permissible wall thickness parameter for hollow sections ℓ_u/t shown in Fig. 2.16. Opinions on this limit ranged from 4 to 30 with an average of 12. In startling contrast the actual hollow section plans submitted for ten different actual structures in five separate states indicated a very narrow range from 3 to 6. Japanese plans for hollow circular structures indicated actual D/t ranges of 4 to 5. As indicated in Fig. 2.17 from the companion report (Ref. 18), full section action was not found experimentally for sections where ℓ_u/t was > 7.5. Additional data supporting this evidence comes from a study by Proctor [45] and Jobse [46]. Thus this limit should be maintained at 6 to 7 for calculation purposes. As shown in Ref. 18, the general compression flange width limits seem appropriate for strength comparisons.

Other detailing questions indicated a wide difference of opinion on mechanical rebar splicing devices. Several states insist on welding citing concerns regarding fatigue and cracking of concrete cover over splice devices. Several states permit compression sleeve splices only. Almost equal numbers indicated they permit or forbid use of thermite fusion-type couplers. Some allow it only for 14 and 18 bars or in splicing to existing steel. No clear consensus emerged.

Suggestions for minimum wall thickness in hollow piers ranged from 10 in. to 30 in. with the most frequent values being 12 in. and 18 in. One proposal indicated a stepped scale ranging from 12 in. for ℓ_u < 10 ft to 24 in. for ℓ_u > 18 ft. This is another area where constructability is a large factor. An ℓ_u/t limit of 6 will tend to ensure realistic t values to improve placement.

It should be noted that the recent comprehensive work by Podolny and Muller [25] surveying a wide range of U.S. and European segmental bridges indicate much wider ranges of ℓ_u/t values in these large bridges in Europe. The California bridges cited range from 3 to 10 while the European bridges range from 8 to almost 20. The Houston Ship Channel bridge which has heavy European influence varies from 12 to 17 in one axis and 8 in the other.

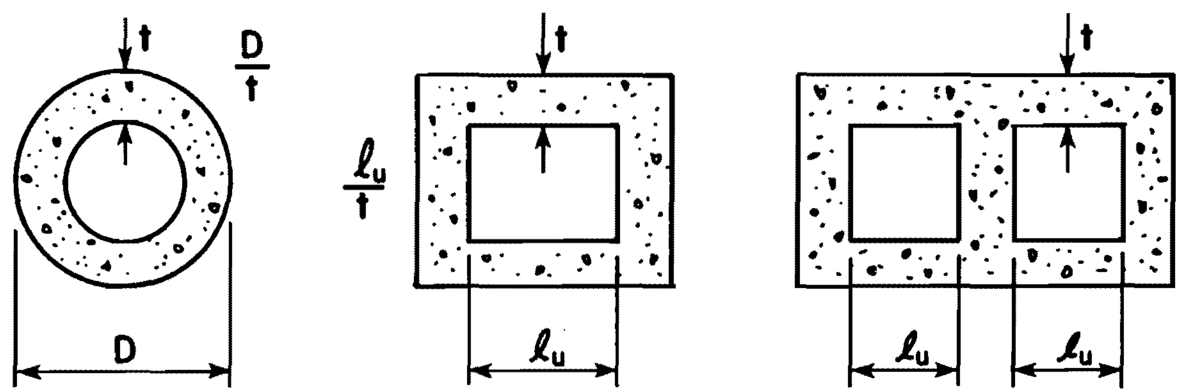


Fig. 2.16 Wall thickness parameters

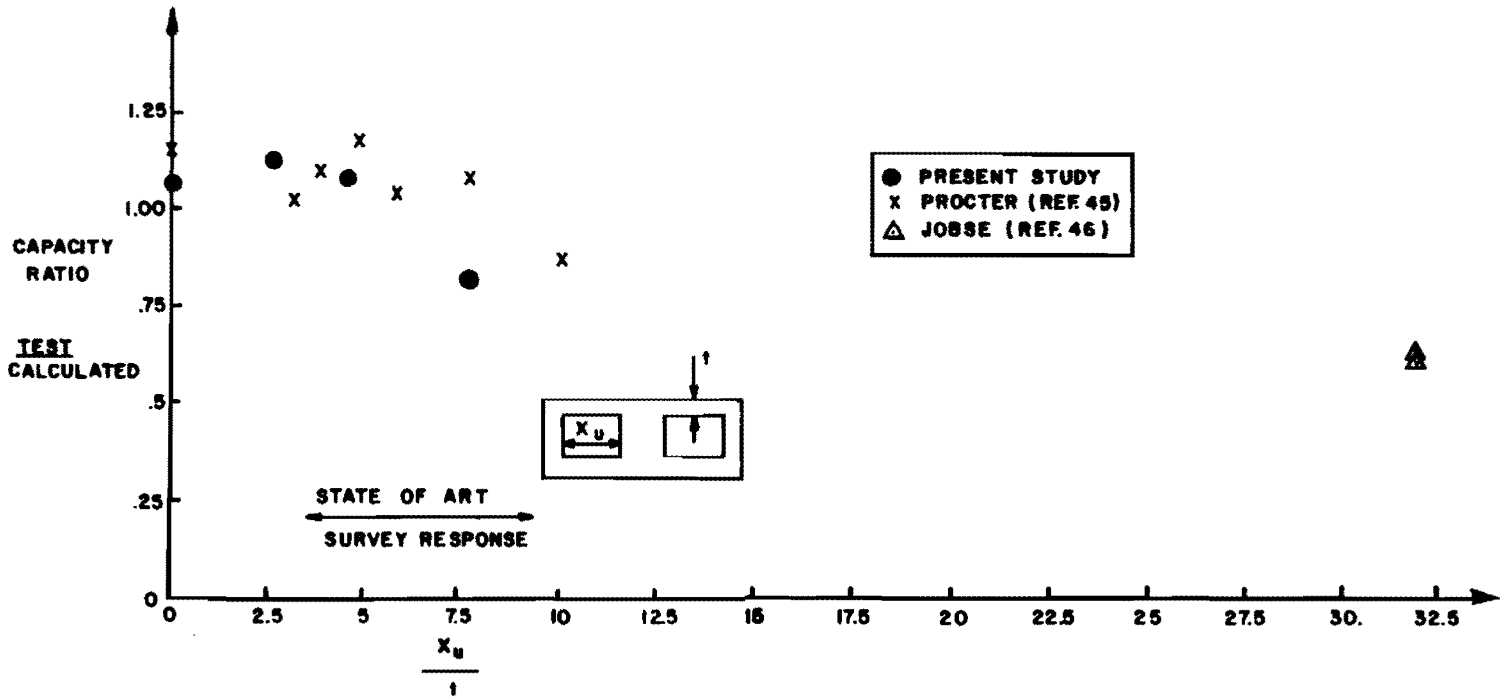


Fig. 2.17 Effect of wall thinness on pier capacity

2.10 Design Approaches

The survey results show that in the U.S. 30% of design is done under AASHTO Allowable Stress Design, 65% is done under AASHTO Load Factor Design, while a few respondents working on railroad bridges use AREA criteria. This indicates most pier design is under a factored load format and the programs were developed accordingly.

Respondents detailed a number of problems concerning current AASHTO slender pier design although almost 25% of the respondents indicated they simply did not run into very slender piers and the current AASHTO approach met their non-need as well as anything else.

Problems frequently mentioned regarding current AASHTO slenderness methods involved uncertainty regarding choice of $k\ell_u$, handling voids, treating biaxial loading cases, determination of β_d where signs reverse on dead load and total load, and the inability to get deflection behavior information. A frequent complaint was that the method was hard to use and needed to be computerized. A number of respondents indicated they were using PCA computer programs for column design and especially biaxial design with some degree of success.

About one-third of respondents indicated interest in improved equations and design aids for slenderness provisions of AASHTO Article 1.5.34(B). A similar number indicated the need for a computer analysis program which would check the adequacy of sections input for specified load conditions. A much smaller number were interested in optimized design-analysis programs.

This interest proved crucial in structuring the objectives in the design phase of this study.

CHAPTER 3

SLENDER BRIDGE PIER DESIGNS

3.1 Overview

The design of a bridge pier normally occurs only after many conditions and factors affecting the entire bridge are considered. Some of these factors are site geometry and foundation conditions, acceptable locations of supports (abutments and piers), longitudinal and transverse superstructure alternatives costs, aesthetics, and construction techniques. The interaction of these parameters is shown conceptually in Fig. 3.1. In the design process the superstructure is often chosen almost independently of the pier type. There are two design processes occurring. The first is the design of the superstructure. The second is the design of the pier to fit the superstructure system chosen. Even though the pier location is established early in the bridge layout, the exact type of pier and the actual pier design are often not determined until much later. This means little overall superstructure-pier interaction is taken into account.

Figure 3.2 shows some of the longitudinal framing systems currently in use. Most of the piers shown are prismatic and probably would have a solid cross section. Each of the systems represents different pier behavioral characteristics. The transfer of the load, the degree of fixity, and the imposed deflections are all different. The choice of method of connection between pier and superstructure can greatly affect the pier design. Very often the piers will have to provide for substantial superstructure movement due to thermal and other volume changes.

A wide variety of transverse pier configurations are also possible. Distributions found in the survey are shown in Figs.

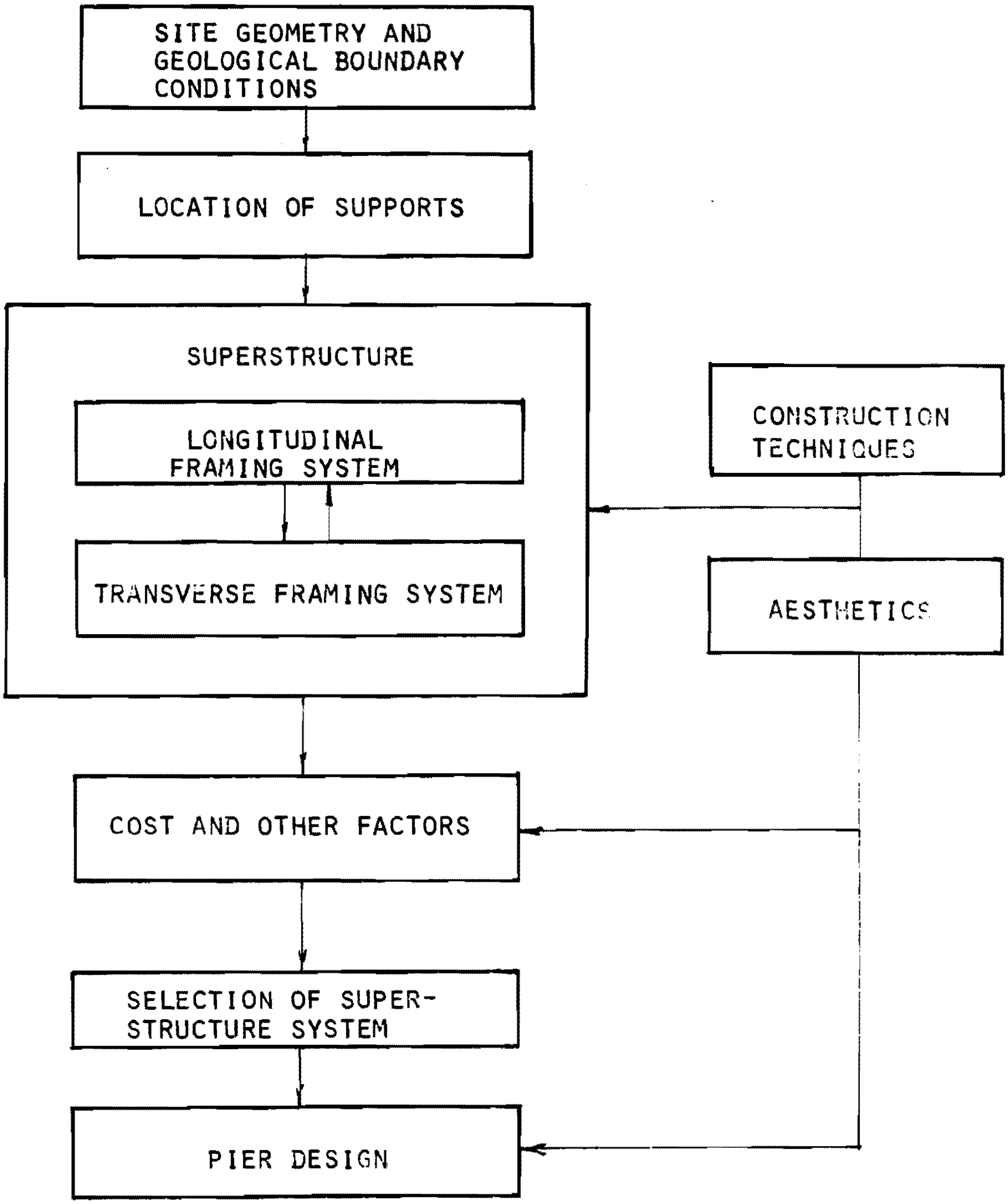
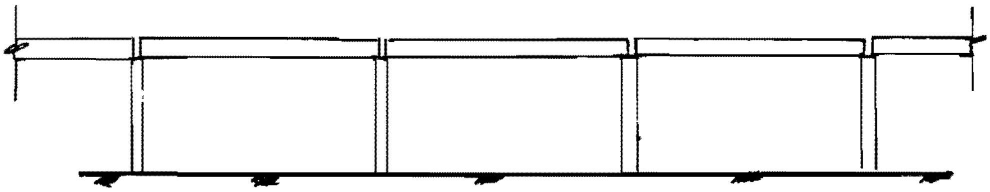
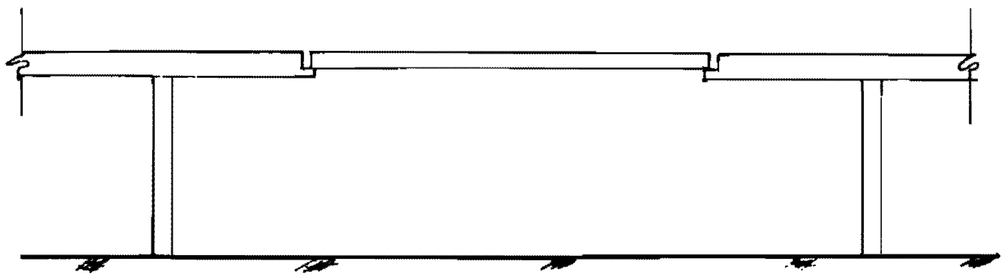


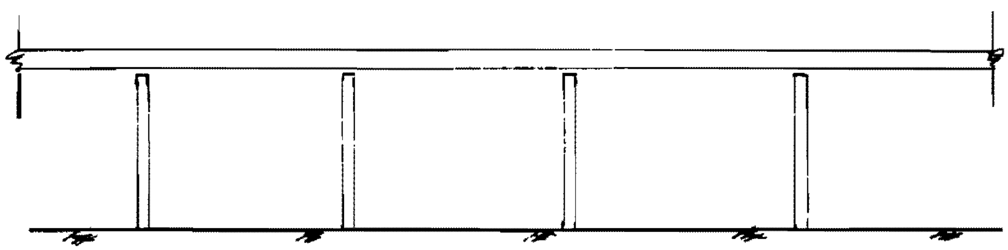
Fig. 3.1 Flowchart of bridge design



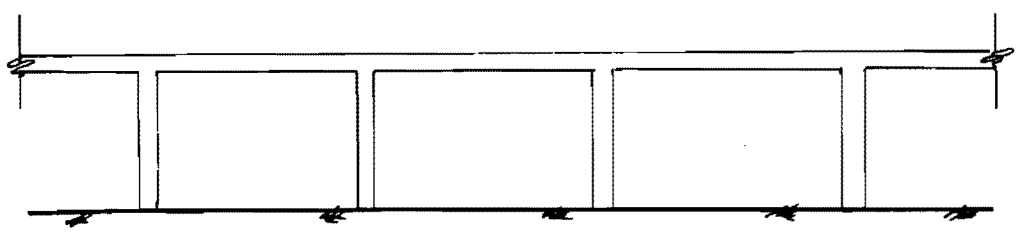
SIMPLE SPAN



CANTILEVERED SUSPENDED SPAN



CONTINUOUS SPAN



FULL CONTINUITY RIGID FRAME

Fig. 3.2 Longitudinal framing systems

2.5 and 2.6. Each of these configurations introduces new problems for the designer. Results from the survey reported in Chapter 2 clearly indicate that biaxial loading effects will be encountered in many piers and a substantial number will fall in slenderness ranges where the secondary moments due to pier deflection must be considered. While the survey indicated the vast majority of all piers would be solid in cross section, it also clearly indicated a growing trend towards hollow cross sections in large, tall piers.

The basic reason for the trend towards hollow piers can be seen in Fig. 3.3 which contrasts pairs of solid and hollow cross sections. The hollow sections were selected to have approximately the same moment of inertia about their x- and y-axis as the solid sections. Also shown are some relative dead load weights and relative costs. The costs were estimated using realistic values for materials and formwork and represent pier construction costs only. Savings in foundation cost due to the appreciable dead load reduction would result in additional cost savings. The formwork for the hollow section costs more, but the material savings cancels out this cost increase. Each of the three aspect ratios (width/depth) indicate a reduction in the weight of the hollow section over the solid section and the lower aspect ratios show a reduction in direct costs. The aspect ratio of 1.5 is of particular interest. This solid section is approximately the same size as the piers of the John F. Kennedy Memorial Causeway Bridge in Corpus Christi, Texas [32]. If the hollow section had been used, the dead weight of the pier would have been 45% of the solid section weight at about 75% of the cost. In addition, there would have been substantial savings in foundation costs due to the reduced dead load.

However, there are also some disadvantages in the use of hollow sections. Presently, there is no specific design criterion for their use although there have been successful projects incorporating slender piers with cellular cross sections built around

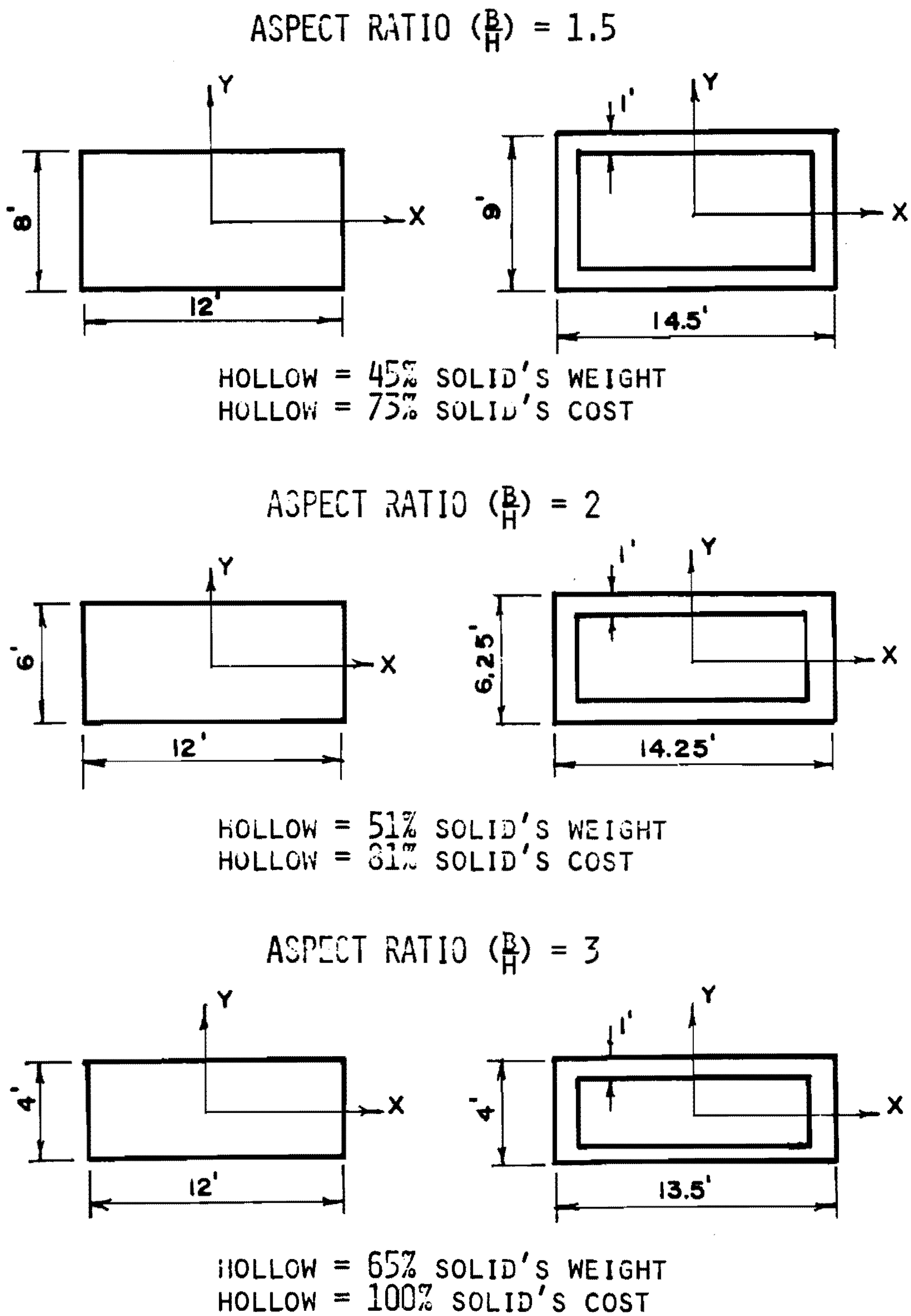


Fig. 3.3 Comparison of hollow to solid cross section

the world. Figure 3.3 suggests that direct cost advantages of hollow sections extend only to depth-to-width ratios of about 3. Above this ratio, extra formwork costs probably surpass material savings. This is indirectly confirmed by examination of the tall piers for segmental bridges reported by Podolny and Muller [25]. These piers are almost all in the aspect ratio range from 1.0 to 3.0 and the aspect ratios reported average 1.75. Other constraints are associated with detailing. For example, preliminary investigation indicates a minimum wall thickness of a hollow section may be 12 in. based solely on reinforcing steel cover requirements. Further investigation of applicable detailing requirements is urgently needed.

It is obvious that bridge pier design procedures need to be able to accommodate a wide variety of longitudinal and transverse configurations. It is also apparent that the general nature of support conditions in the United States tends towards nonfixed superstructure connections so that deflection criteria for piers might be as important as strength limits. This suggests that the most appropriate analysis techniques are those which will provide displacement values as well as appropriate strength information.

Although the design of slender bridge piers can become quite complex, the vast majority of piers actually used is very stocky and slenderness effects are minimal and may be neglected in design. As shown in the survey results given in Chapter 2, bridge piers tend to fall in the less than 30 ft height range with height-to-thickness ratios often well below 10. If reasonable resistance to sway is present no slenderness effect need be considered. If the pier is free to sway then a check of the $k\ell_u/r$ ratio is necessary. AASHTO Specifications Article 1.5.34(B)(4) indicates that slenderness effects may be neglected for compression members not braced against sidesway when $k\ell_u/r$ is less than 22. Such checks are extremely important and need to be expanded. A major simplification in pier design would be easier and more direct ways of diagnosing similar cases where slenderness will be unimportant and hence slenderness effect calculations can be avoided.

The various procedures for analysis of slender compression members will be briefly surveyed in subsequent sections. Suggested improvements or adaptations in several of the approximate methods will be suggested. Some changes in the AASHTO Specifications will be suggested. The basis and philosophy for comprehensive computer analysis procedures able to handle almost all cases reported in the state of art survey will be given. Development and use of the programs is given in subsequent chapters.

3.2 Differences between Slender Bridge Pier Design and Slender Building Column Design

While many factors affecting the design of slender bridge piers and slender building columns are the same, there are a number of important differences which have added to the uncertainty of design of bridge piers utilizing design procedures developed originally for slender building columns.

3.2.1 Longitudinal Restraint Conditions. A number of possible longitudinal pier configurations are shown in Fig. 3.4. All of these configurations have been reported in use for bridge piers. The degree of movement at the top greatly exceeds those found in buildings. Figure 3.4(a) illustrates a pseudo rigid frame action or fixed-fixed condition. This fixed condition at the top implies that there is no articulated joint requiring maintenance, and provides increased overall stability especially during construction. One of the disadvantages with the condition is the moments induced at the foundation. These moments can cause rotation of the foundation, θ , which in turn increases the effective length.

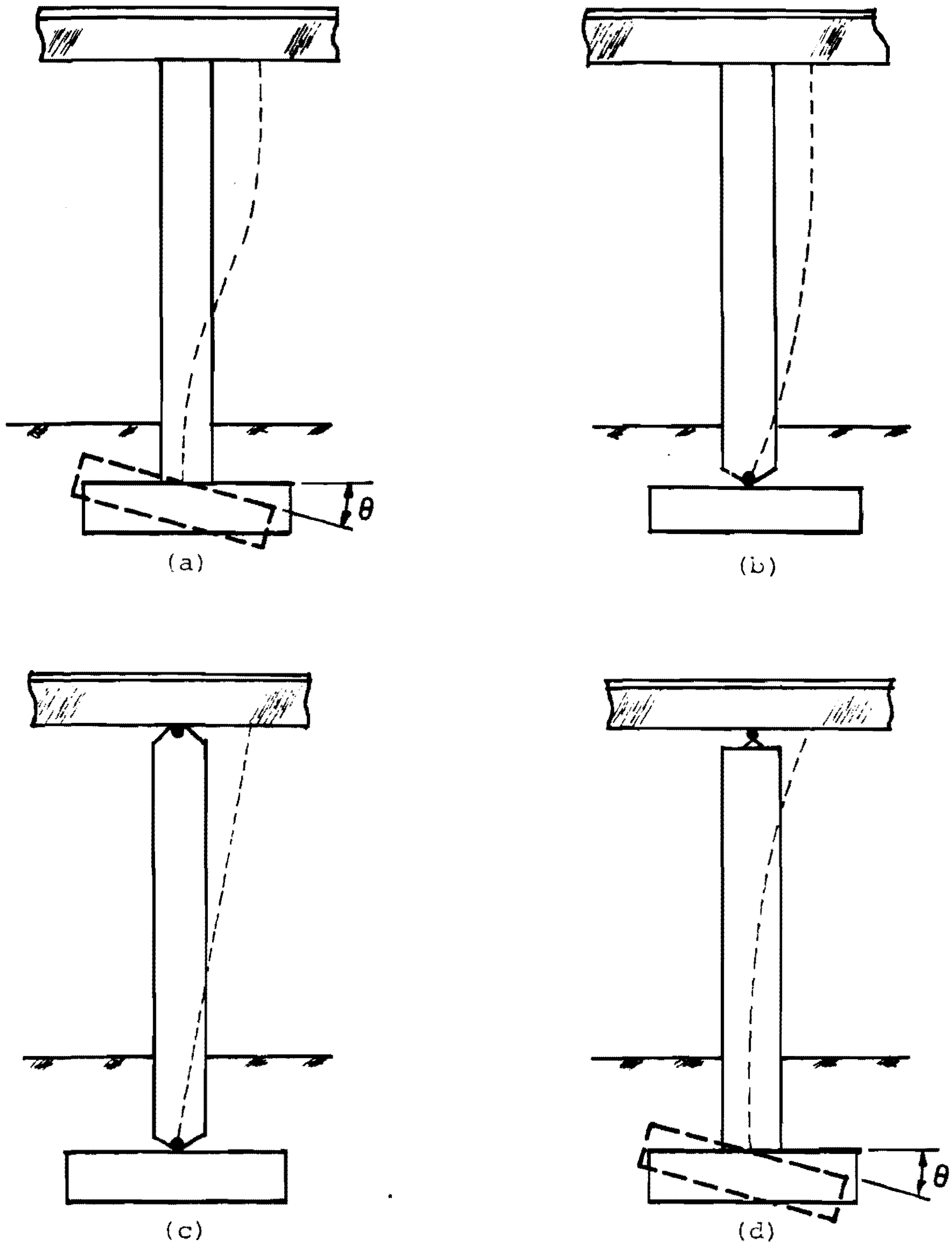


Fig. 3.4 Longitudinal configurations of slender piers

Another condition is shown in Fig. 3.4(b). This fixed-hinged condition provides for little or no moment at the foundation. However, there is a hinge detail requiring attention, and possible instability of the bridge unless the superstructure is continuous.

Figure 3.4(c) contains hinges at both ends of the pier. In this situation, the first-order forces are relatively easy to calculate since there are no primary moments. However, there are now two joints to detail and keep free to rotate substantial instability problems until the entire bridge is completed.

Finally, Fig. 3.4(d) depicts a quasi-free cantilever configuration. Any longitudinal force on the superstructure would impose a lateral load at the top end of the pier. This condition offers advantages in construction, but disadvantages in the maintenance of the bearings. The moments at the base can cause rotation greatly reducing stability.

Cases (a) and (d) are the ones most widely used in practical structures. The longitudinal displacement restraint at the top of the pier needs careful definition and greatly affects the effective lengths in these two cases. Similarly, pier-to-foundation rotational stiffness are not well-defined.

The present approach of designing the pier independently of the superstructure does not provide for possible interaction. In order to properly evaluate the effects of longitudinal stiffness, a comprehensive analytical approach which included overall girder and pier interaction is required. This is not included in the current AASHTO approximate slenderness calculation procedures but can be treated in a general analysis in which longitudinal spring stiffness at pier tops can be input.

The degree of rotational restraint at the foundation level is an extremely significant and uncertain variable. In the current AASHTO approximate slenderness procedure this must be input through its effect on the effective length factor k. The 1969 FHWA Ultimate Design Guide [33] provided guidance for relating pier stiffness to foundation stiffness. Numerous authors [20-24] provide guidance for determining approximate vertical, longitudinal and rotational stiffness for shallow foundations on various soil types. These approaches seem much more suited for use with comprehensive analysis procedures than with the approximate procedures.

3.2.2 Biaxial Bending. In most buildings appreciable biaxial bending effects are limited to corner columns. Usually the cross frames provide bracing against lateral translation at story levels. Few building cases seem to be governed by biaxial bending analyses. In the state-of-the-art survey a number of respondents indicated that biaxial bending was often a major design consideration in slender pier design. This is very difficult to do in the approximate procedures and another reason for the use of a general analysis.

3.2.3 Load Case Combinations. Most buildings are governed by either combinations of dead and live load or of dead, live and wind or seismic load. In contrast, as seen from the survey results in Chapter 2, bridge piers are subject to a much wider range of load combinations. In various locales ice, current, wind or seismic are appreciable while thermal and volume change effects greatly affect many restraint combinations. These complex load combinations make approximate analysis more difficult.

3.2.4 Cross Section. Most building columns are square, rectangular or circular. While occasional cruciform or L-shaped compression members occur in buildings , there are very few hollow, oval, or cellular sections. Few building columns taper, flare, or are stepped between floor levels. The wide variety of such occurrences in bridge piers greatly complicates their design.

3.3 Slender Compression Member Analysis Procedures

There have been several methods of analysis advocated for slender compression members and utilized in past or present American Codes or Specifications. These include:

(a) Reduction Factor Method

(b) Moment Magnifier Method

(c) Stability Index Procedure

(d) Computer based second order frame analysis

In the following subsections a very brief review is given of each method and suggestions for improvements in utilization of several are made. A paramount consideration is the need for a relatively simple way of determining whether slenderness effects must be considered in a given case. Several of the approximate procedures can be extremely useful in making this determination. This will be pointed out when applicable.

A very fundamental problem affecting highway bridge designers is the lack of a concurrent commentary document to the AASHTO Specifications. Commentary-type material is given when changes are first adopted but then is not printed in subsequent revisions. Much of the practical implementation of the ACI Building Code procedures for slender column design depends on material contained in the ACI Building Code Commentary. This severe lack should be rectified by AASHTO.

3.3.1 R-Method (Reduction Factor Method (ACI 318-63)).

(A procedure of this type was previously used in AASHTO Specifications.) The R-method of the ACI Building Code (ACI 318-63) uses a reduction factor, R, which was a linear function of the slenderness ratio h'/t $(k\ell_u/r)$. This factor was applied to the short column axial load and moment capacity to determine the permissible long column capacity. The required R was determined from empirical straight line equations considering the restraint conditions, eccentricities, and slenderness ratios. The method was shown to be unsafe for many members which were not restrained against sway [14]. A version which was virtually limited to columns restrained against sway was allowed in the ACI Building Code 1971 Commentary but the method was dropped from the ACI Building Code and subsequently from the AASHTO Specifications.

There are several arguments for not using this particular method for slender column design [14]. The R-method implies maintaining the same eccentricity for the short column and its long column counterpart. This condition does not physically represent the actual behavior. In actuality, the load-carrying capacity is decreased because of an increasing eccentricity. The eccentricity is changing because of secondary deflection moments. No sense of these deflections is given in using the procedure so that the necessary increase in restrained moment capacity is often overlooked. The R-method is attractive because of the relative ease of use, but neglects many important parameters. It was found that due to the neglect of these parameters, the equations were overly conservative in many practical cases. Also, tests indicated that the R-method could be extremely unsafe in some cases of instability of unbraced frames and members [14].

In the 1977 ACI Building Code Commentary the R-method is allowed as an alternate procedure for compression members not braced against sidesway for $k\ell_u/r$ not exceeding 40. However, the method discounts the capacity of any member with $k\ell_u/r$ greater than 9. This is considerably less than the present AASHTO Specification limit in Article 1.5.34(B)(4) which allows slenderness to be neglected for any unbraced member with $k\ell_u/r$ less than 22. In addition, the R-method has never envisioned nonprismatic or hollow members.

Considering the many problems associated with its use, the R-method does not represent a viable method for analysis of slender bridge piers.

3.3.2 Moment-Magnifier Method (ACI 318-71). The moment magnifier method is an approximate solution for the maximum moment in an elastic beam-column [14]. The magnification factor is calculated from the ratio of end moments, axial load applied, and the theoretical critical load of the column modified for reductions in stiffness due to cracking and nonlinear effects. This procedure is more rational than the R-method since it reflects the actual behavior of the column. The concept is very familiar to American engineers who use much of the same procedure for designing steel columns under the AISC Specifications [34]. It has been adopted by AASHTO as the basic approximate method for determination of slenderness effects in concrete compression members. However, the AASHTO Specifications do not reproduce the large amount of commentary contained in ACI Building Code Commentary Section 10.11 which is extremely helpful in its proper utilization.

There are some other limitations to application of the procedure to bridge piers. The range of variables originally investigated was limited. These ratios include slenderness ratios ($k\ell_u/r$), restraint conditions, and eccentricity to thickness of column ratios (e/h) comparable to those found in buildings [14]. Bridge piers are likely to have larger ratios. Also, the column tests against which the method was verified consisted of only prismatic and solid sections. Finally, the procedure is based on strength considerations and does not directly alert the designer if a deflection problem exists.

In practical application this method is basically a "hand" technique for making approximate second-order analysis. It was originally presented as a substitute approach for use when more exact second-order analysis procedures such as utilization of comprehensive computer analyses are not available. In order to extend and validate the procedures for nonprismatic or hollow pier design, a wide range of variables needs to be studied. A criterion for deflection would have to be incorporated into the magnification procedure to alert the bridge designer to possible deflection problems. The method would require major revision to treat variable cross sections. It is very difficult to use with biaxial bending. It is suggested that the method be used primarily for preliminary design or for design of prismatic piers which are not particularly slender.

However, for piers of low to moderate slenderness the method can be used to advantage. AASHTO Article 1.5.34(B)(4) is extremely useful in defining a large category of compression members for which slenderness effects can be safely neglected. For members braced against sidesway slenderness may be neglected for $k\ell_u/r$ less than 34 - $(12\ M_1/M_2)$. For members not braced against sidesway slenderness may be neglected for $k\ell_u/r$ less than 22. A prompt check of these values will eliminate many bridge piers from any need for further slenderness checks or computations.

This check requires knowledge of four factors:

(a) whether the member is braced or not braced against sidesway
(b) the effective length factor k
(c) the unsupported length ℓ_u
(d) the radius of gyration r $(I = Ar^2)$

The latter two factors are adequately defined in AASHTO Article 1.5.34(B)(1) and (2). However, responses to the state of art questionnaire indicated that there is considerable confusion and uncertainty as to what is proper for (a) and (b). This seems to stem from the omission of needed ACI Commentary material in Article 1.5.34(B). In absence of a companion AASHTO Commentary, some informative material should be added to the Specifications.

There are two practical ways to determine if the compression member is braced against sidesway. The first method is better suited for buildings where there is a possible mix of the compression members in a story with shear walls, shear trusses or other types of lateral bracing. In this type application a compression member may be assumed braced if the other bracing elements have a total stiffness resisting lateral movement of the story at least six times the sum of the lateral stiffness of all the compression members within the story. In applying this criterion to continuous bridges the engineer could compare the effective longitudinal plane stiffness of the deck-abutment system to the sum of the lateral stiffnesses of the piers to determine whether the system is braced.

The 1977 ACI Commentary gives another method for determining whether a structure is to be considered braced or unbraced. This method is based on the stability index, Q, which is outlined in the next subsection. When the stability index, $Q = (\Sigma P_u \Delta_u / H_u h_s)$, for a story is not greater than 0.04, the PΔ moments are minimal and the structure can be considered as braced. H_u is the total factored lateral force acting within the story and Δ_u is the elasticity computed first order lateral deflection due to H_u (neglecting PΔ effects) at the top of the story relative to the bottom of the story. This procedure seems very well-suited to typical highway pier structures, since many first order analysis programs are readily available for computing Δ_u for a wide range of base and top restraint conditions.

After determination of whether the structure is to be considered braced or unbraced, the designer must determine the effective length factor, k. This requires a determination of the relative rotational restraint at each end of the compression member and then determination of the effective length as a function of the ratios $\psi = \Sigma EI/\ell$ of compression members to $\Sigma EI/\ell$ of restraint members in planes at the ends of the member. Since the members are often restrained by a pier, footing, or other base this is probably best expressed as $\Psi = \Sigma K_c/K_r$ where K_c is the rotational stiffness of the compression member and K_r is the rotational stiffness of the top or bottom restraint. In calculating relative stiffness effects, reinforcement percentage and cracking should be considered. For restraining members cracked transformed section properties should be used and for compression members the EI from AASHTO Eq 6-17 [5] with $\beta_d = 0$ should be used. Typical calculations for spacing constants for foundation types may be found in Refs. 20-24. The FHWA suggested Ψ values for footings are in Ref. 33. When the values of Ψ at top and bottom have been determined, then k can be found from the nomographs of Fig. 3.5 (ACI 318 Commentary Fig. 10-3). A set of equivalent equation values are given in the ACI Commentary for use when charts are inconvenient such as in computer programs. These could be included in the AASHTO Specifications for designer guidance. These values are:

For braced compression members, an upper bound to the effective length factor may be taken as the smaller of the following two expressions:

$$k = 0.7 + 0.005\ (\Psi_A + \Psi_B) \leq 1.0 \qquad \text{(A)}$$

$$k = 0.85 + 0.05\Psi_{min} \leq 1.0 \qquad \text{(B)}$$

where Ψ_A and Ψ_B are the values of Ψ at the two ends of the column and Ψ_{min} is the smaller of the two values.

For unbraced compression members restrained at both ends, the effective length may be taken as:

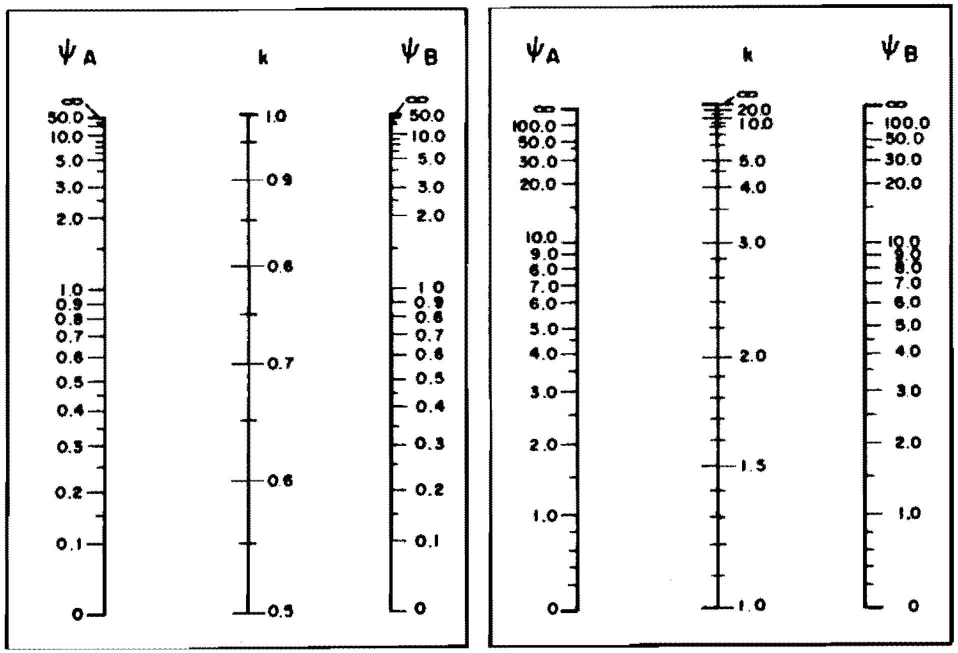
For $\Psi_m < 2$

$$k = \frac{20 - \Psi_m}{20}\sqrt{1 + \Psi_m} \qquad \text{(C)}$$

For $\Psi_m \geq 2$

$$k = 0.9\sqrt{1 + \Psi_m} \qquad \text{(D)}$$

where Ψ_m is the average of the Ψ values at the two ends of the compression member.



(a)

Braced Frames

(b)

Unbraced Frames

ψ = Ratio of $\sum(EI/\ell_c)$ of compression members to $\sum(EI/\ell)$ of flexural members in a plane at one end of a compression member

k = Effective length factor

Fig. 3.5 Effective length factors [ACI 318-77 Commentary Fig. 10-3]

For unbraced compression members hinged at one end, the effective length factor may be taken as:

$$k = 2.0 + 0.3\Psi \qquad \text{(E)}$$

where Ψ is the value at the restrained end.

Inclusion of these changes should greatly reduce the bridge designer's uncertainty in computation of effective length. For members with $k\ell_r/u$ less than 60 a simpler procedure of determining Ψ can be used. Flexural stiffness of restraint members can be based on 0.5 I_g and of compression members can be based on I_g. This will adequately model cracking effects for the determination of k. In braced members k can always be initially estimated as 1. These assumptions will speed the check as to whether slenderness effects need to be considered.

Another problem area in application of the moment magnification procedures indicated by respondents was in the determination of β_d. In the specifications, β_d is defined as the ratio of maximum dead load moment to maximum total load moment, always positive. Questions were raised as to what happens when dead load and total load moments were of opposite sign. The β_d factor is a crude way to express the creep effect. The intent was that if the moment being magnified (which is the total moment) was mostly sustained load moment then the factor should approach 1. If the moment being magnified was mostly short time load moment then the factor should approach zero. The present formulation is faulty since in its origin it did not clearly envision reversed signs on dead load and live load moments. In early stages it was formulated in terms of dead load and total load thrust values which are almost always positive. When converted to moment terms the sign problem was mishandled. Since the entire procedure is based on flexural stiffness β_d is more appropriately based on moments than on thrust values. However, the original intent would be more clearly realized and the present confusion would be removed if the definition were changed to "β_d is the ratio of the absolute value of maximum dead load moment to the sum of the absolute values of the moments comprising the total load moment." This would then be a positive value between 0 and 1 as originally envisioned.

In view of the relatively low percentage of hollow and cellular piers, the complex process of determining appropriate EI values is not justified. Second order analysis programs will be proposed for these cases as well as stepped, tapered, and flaring piers.

In very recent years several important developments have been recognized by ACI Committee 318 and adopted for the 1983 ACI Building Code. One of these affects the moment magnifier approach and has substantial potential for greatly reducing design moments in slender piers. The substantiating data are contained in Ref. 19. Completion of a RCRC-sponsored project resulted in these recommendations which clarify the application of the moment magnification procedure to unbraced frames. The proposed changes will generally result in substantially lessened pier design moments, since gravity load moments will be magnified by braced frame magnification factors in most cases. The new provisions will lead to economy in both piers and footing systems where unbraced piers are used. The proposed provisions do not require computation of more magnifiers than that of the current procedures. The text adopted by ACI Committee 318 but slightly modified for consideration by AASHTO is:

Introduce the following new notations in Article 1.5.34(B):

"δ_b = moment magnification factor for frames braced against sidesway <u>to reflect effects of member curvature between ends of compression members</u>

δ_s = moment magnification factor for frames not braced against sidesway <u>to reflect lateral drift resulting from lateral</u> and gravity loads

M_{2b} = value of larger factored end moment on compression members due to loads which result in no appreciable sidesway, calculated by conventional elastic frame analysis

M_{2s} = value of larger factored end moment on compression member due to loads which result in appreciable sidesway, calculated by conventional elastic frame analysis."

Revise Article 1.5.34(B)(5) on p. 108 to read as follows, and renumber subsequent equations accordingly:

"(5) Compression members shall be designed using the factored axial load P_u from a conventional frame analysis and a magnified factored moment M_c defined by:

$$M_c = \delta_b M_{2b} + \delta_s M_{2s} \tag{6-14}$$

where

$$\delta_b = \frac{C_m}{1 - \dfrac{P_u}{\phi P_c}} \geq 1.0 \tag{6-15}$$

$$\delta_s = \frac{1}{1 - \dfrac{\Sigma P_u}{\phi \Sigma P_c}} \geq 1.0 \tag{6-16}$$

and

$$P_c = \frac{\pi^2 EI}{(k\ell_u)^2} \tag{6-17}$$

ΣP_u and ΣP_c are the summations for all compression members on one level comprising a bent or connected integrally to the same superstructure, and collectively resisting the sidesway of the structure. <u>For frames not braced against sidesway, both δ_b and δ_s shall be computed.</u> For frames braced against sidesway, δ_s shall be taken as zero. In calculation of P_c, k shall be computed as for compression members braced against sidesway for δ_b and as for compression members unbraced against sidesway for δ_s."

(The remainder of Article 1.5.34(B)(5) on p. 109 of the 1977 Specification remains the same.)

Suggested commentary as follows:

"When Eq. (6-14) is used in the design of a braced bent, the second term becomes zero since the moment magnification factor δ_s is taken as zero for a braced bent ($\delta_s = 0$). For sway bents both terms must be evaluated. The moment M_{2b} is the moment resulting from gravity loads while M_{2s} is the moment resulting from lateral loads. In both cases these moments are computed using a conventional (first-order) frame analysis. The member stiffness used in the analysis should allow, at least approximately, for cracking of the flexural members.

"Since the moment magnification procedure of Article 1.5.34(B) is an approximate procedure, some judgment must be used in the application of Eq. (6-14). The application of gravity loads to an unbraced bent in an unsymmetrical pattern or to an unsymmetrical unbraced bent will result in some calculated sidesway of the bent. Unless this lateral deflection due to vertical loading is appreciable ($\Delta/\ell_u > 1/1500$), the minor effect of this sway component can be neglected and the corresponding moments can be considered as nonsway moments and should be magnified by the braced bent magnifier. The definition of M_{2b} and M_{2s} both contain the terminology "appreciable sway." For use in this approximate technique, a deflection ratio of $\Delta > \ell_u/1500$ is a reasonable upper limit above which sway deflections become "appreciable." In computing Δ for this purpose, only the nonlateral loads should be used.

"Equation (6-14) is new in this edition of the Specifications. Article 1.5.34(B) of the '77 specifications contained similar provisions for an approximate computation of the additional compression member secondary or $P\Delta$ moments due to slenderness effects. A distinction was made between the magnifier (δ) computations for braced and unbraced bents, but no distinction was made regarding the moment to be magnified. In the 1977 Specifications, design of compression members used the factored axial load P_u from a conventional frame analysis and a magnified factored moment M_c defined by:

$$M_c = \delta M_2 \qquad (3\text{-}1)$$

M_2 was the larger factored end moment on the compression member as calculated by a conventional elastic frame analysis. This definition strongly implied that in unbraced bents, M_c was the sum of the moments due to gravity loading (which generally would not produce major sidesway) and the moments due to lateral loading (which would produce sidesway).

"Specification provisions do provide for different methods of calculating δ for the braced and unbraced cases. For the braced bent, the magnifier δ uses effective length factors of 1.0 or less, while for the unbraced case they are greater than 1.0. C_m factors for braced bents can be from 0.4 to 1.0, while for unbraced bent, 1.0 is used. Thus, critical loads P_c are higher and magnification factors are smaller for the braced case. For unbraced bents, 1977 Specification Article 1.5.34(B)(9) required that two values of δ be computed and the larger value used. To check the effects of overall member group stability, the unbraced value of δ is computed as an averaged value for the entire group based on use of $\Sigma P/\Sigma P_c$. This reflects the interaction of all compression members in the bent on the $P\Delta$ effects since the lateral deflection of all compression members in the bent must be equal in the absence of twist. In addition, since it was possible that a particularly slender individual compression member in an unbraced bent could have substantial midheight deflections even if adequately braced against lateral end deflections by the other members in the bent, the 1977 Specifications required that each individual compression member be also checked using the braced bent magnifier value. The specific Specification wording implied that one calculated the two δ values, selected the larger, and applied it to the value of M_2.

"A fundamental problem was that the 1977 Specification used a single symbol δ, without subscript, for two very different cases, the braced or nonsway magnifier (δ_b) and the unbraced or sway magnifier (δ_s). These new symbols should be introduced in the 1983 Specifications to clearly distinguish between the two cases. An additional problem was that the 1977 Specifications used a single symbol M_2 for the moment to be magnified and defined it in such a way that it strongly implied M_2 was the sum of the factored nonsway moments and factored sway moments.

"Basic stability theory indicates that moments which produce sway should be magnified by a sway magnifier (δ_s) and moments which do not produce sway should be magnified by a braced magnifier (δ_b). In addition, it must be realized that the maximum magnified moments produced by δ_b occur at different locations, i.e., somewhere along the compression member length but not necessarily at the member end.

"Measurements made in frame tests clearly support the differentiation between sway or nonsway moments in computations of maximum design moments for the gravity plus lateral load case. Results indicate that

$$M_c = M_g + \delta_s M_s \tag{3-2}$$

where M_g and M_s are the gravity load moments and lateral load moments, respectively, should be used rather than the usual interpretation

$$M_c = \delta M_2 = \delta(M_g + M_s) \tag{3-3}$$

Test results showed Eq. (3-3) to be extremely conservative when compared to Eq. (3-2) [14]. All supportive evidence in the frame tests and accompanying computer analyses indicated that sway magnification of the gravity moment by the sway magnifier is unwarranted.

"If the gravity moments are significantly larger than the lateral load moments and δ_b is large, the maximum moment for the gravity plus lateral load case can theoretically occur at some midheight region of the compression member. The specific location is not known and a conservative approximation can be made by recognizing that, while $\delta_b M_g$ and $\delta_s M_s$ do not occur at the same location, the actual moment cannot exceed their sum. Hence, the Specification may use a conservative approximation of the design moment as

$$M_c = \delta_b M_g + \delta_s M_s \tag{3-4}$$

"Equation (3-4) has two distinct advantages over Eq. (3-1). It will almost always reduce the design moments from the extremely conservative levels of the 1977 Specification, and it more clearly indicates the application of the two magnification factors required to be computed for unbraced frames under 1971 Specifications Art. 1.5.34(B)(9). That article may then be deleted.

"If the lateral load deflections involve a significant torsional displacement, the moment magnification in the compression members farthest from the center of twist may be underestimated by the moment magnifier procedure. In such cases a second-order analysis is recommended."

3.3.3 <u>"Q" or Stability Index Procedure</u>. This method of including second order effects has been suggested by several authors [29,30,31]. Some elements of its use are currently found in the ACI Building Code Commentary [2]. The Commentary suggests calculation of the stability index, Q, of a story of constant height to determine if the structure is braced.

$$Q = (\Sigma P_u \Delta_1 / (H_u h_s)) \qquad (3\text{-}5)$$

where

H_u = total factored lateral load acting on the story

Δ_1 = elastic first order deflection of the story

ΣP_u = summation of the factored axial load of the story

h_s = center-to-center height of the story

When the stability index for a story is not greater than 4%, the PΔ moments should not exceed 5% of the first order moments, and the structure could be considered braced [2].

In reality the Q methods application for bridge pier design is somewhat more complex. The formulation by MacGregor and Hage on which the ACI Commentary is based assumes that columns are stiffer than beams and all columns in a story have equal height. In those cases where bridge piers are connected monolithically to superstructures, the flexural restraint stiffness of the superstructure is often significantly greater than that of the pier. The Q method expression for stability index needs to be modified for such a case. Similarly, the entire procedure must be modified for piers of different heights.

The Q method is based on an expression suggested by Rosenbleuth

$$\Sigma P_{cr} = \frac{KL}{\gamma} \tag{3-6}$$

where γ = 1.0 for stiff columns, flexible beams

γ = 1.22 for stiff beams, flexible columns

$K = \frac{H}{\Delta}$ = translational stiffness or force per unit deflection

When the values of ΣP_{cr} are checked against the more conventional expression

$$P_{cr} = \frac{\pi^2 EI}{(k\ell_u)^2} \tag{3-7}$$

it is found that good correlation exists through a wide range of relative EI values as long as the approximate value of γ is used.

Q' is defined as P_u/P_{cr}, thus

$$Q' = \frac{P_u}{P_{cr}} = \frac{P_u\,\gamma}{K\,L} = \frac{P_u\,\gamma\,\Delta}{H\,L} \tag{3-8}$$

Note the absence of the γ term when Eq. (3-8) is compared with the ACI Commentary Q expression given by Eq. (3-5). MacGregor has assumed γ = 1 is appropriate for building columns. However, it is not appropriate for some bridge pier conditions so that the Q method needs modification for bridge use.

The application of the stability index in second order moment analysis is shown in Fig. 3.6 by application to a cantilever beam. In this case it is hard to see the parallel with the moment magnification approach. However, if the relationship of Eq. (3-6) is substituted in Eq. (3-10) then since

$$P_{cr} = \frac{KL}{\gamma} = \frac{Hh}{\Delta_1 \gamma} \tag{3-11}$$

$$\frac{P_u}{P_{cr}} = \frac{P_u \Delta_1 \gamma}{Hh} \tag{3-12}$$

$$M_2 = Hh + P\Delta_1 \left(\frac{1}{1 - \frac{P\Delta_1 \gamma}{Hh}} \right) \tag{3-13}$$

$$= Hh + P\Delta_1 \left(\frac{Hh}{Hh - P\Delta_1 \gamma} \right) = Hh + Hh \left(\frac{P\Delta_1}{Hh - P\Delta_1 \gamma} \right) \tag{3-14}$$

$$M_2 = Hh + Hh \left(\frac{\frac{P\Delta_1}{Hh}}{1 - \frac{P\Delta_1 \gamma}{Hh}} \right) \tag{3-15}$$

For those cases where $\gamma = 1.0$ (stiff columns with flexible beams) then

$$Q = \frac{P\Delta_1}{Hh}$$

so

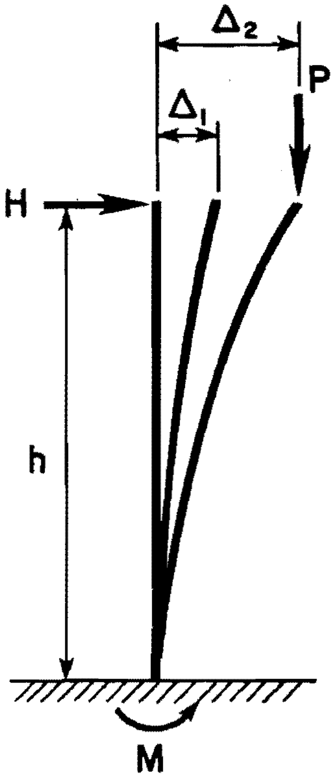
$$M_2 = Hh + Hh \left(\frac{Q}{1 - Q} \right) = Hh \left(\frac{1}{1 - Q} \right) \tag{3-16}$$

Since Hh is the primary moment, the quantity (1/1-Q) becomes in effect the moment magnifier.

For those cases where $\gamma = 1.22$ (flexible columns with stiff beams) then still using $Q = P\Delta_1/Hh$

$$M_2 = Hh + Hh \left(\frac{Q}{1 - \gamma Q} \right) = Hh + Hh \left(\frac{Q}{1 - 1.22Q} \right) \tag{3-17}$$

$$M_2 = Hh \left(\frac{1 - .22Q}{1 - 1.22Q} \right) \tag{3-18}$$



$$\Delta_1 = \frac{Hh^3}{3EI}$$

$$Q' = \frac{P_u}{P_{cr}}$$

$$\Delta_2 = \left(\frac{1}{1 - \frac{P_u}{P_{cr}}}\right) \Delta_1 \qquad (3\text{-}9)$$

$$M_2 = Hh + P\Delta_2 = Hh + P\left(\frac{1}{1 - \frac{P_u}{P_{cr}}}\right)\Delta_1$$

$$M_2 = Hh + P\Delta_1\left(\frac{1}{1 - Q'}\right) \qquad (3\text{-}10)$$

Fig. 3.6 Stability analysis index

Thus the effective moment magnifier is (1 - .22Q)/(1 - 1.22Q). The advantage of this procedure is that the stability index Q can be computed by ordinary analysis procedures without resort to effective lengths. However, in computing Q it is extremely important to recognize that the stiffness $K = H/\Delta$ will vary with load level since the relation between lateral force H and resultant first order displacement Δ_1 should consider realistic stiffness, reinforcement effects, cracking effects, creep effects, etc. Similarly, the term $P_u/\phi P_u$ is necessary since the designer cannot count on ideal properties. These qualifications have often been overlooked in application of the procedure.

Application of the Q or stability index procedure is fairly straightforward for the uniform height pier groups shown in Fig. 3.7(a). Assuming proper stiffness modeling of the overall bridge frame properties, a normal analysis will yield the first order displacement Δ for factored lateral load H and the P_u values. Q can then readily be determined as

$$Q = \frac{P_u \Delta_1}{\phi Hh} \qquad (3\text{-}19)$$

and the magnified moment for design computed from Eq. (3-17) or Eq. (3-18) depending on the level of relative column and restraint stiffness. For moments occurring at ends of the member this will be an accurate procedure. It does not determine possible instability of a slender braced member and this must be still checked as in AASHTO Article 1.5.34(B) for individual braced members.

The application to more general bridge cases where some piers will have different heights as shown in Fig. 3.7(b) is more difficult. For this case it is necessary to determine an averaged Q by determining an equivalent K which considers all column properties.

For this application there are two approaches for finding ΣP_{cr}. The first is to work with effective lengths. For this case A

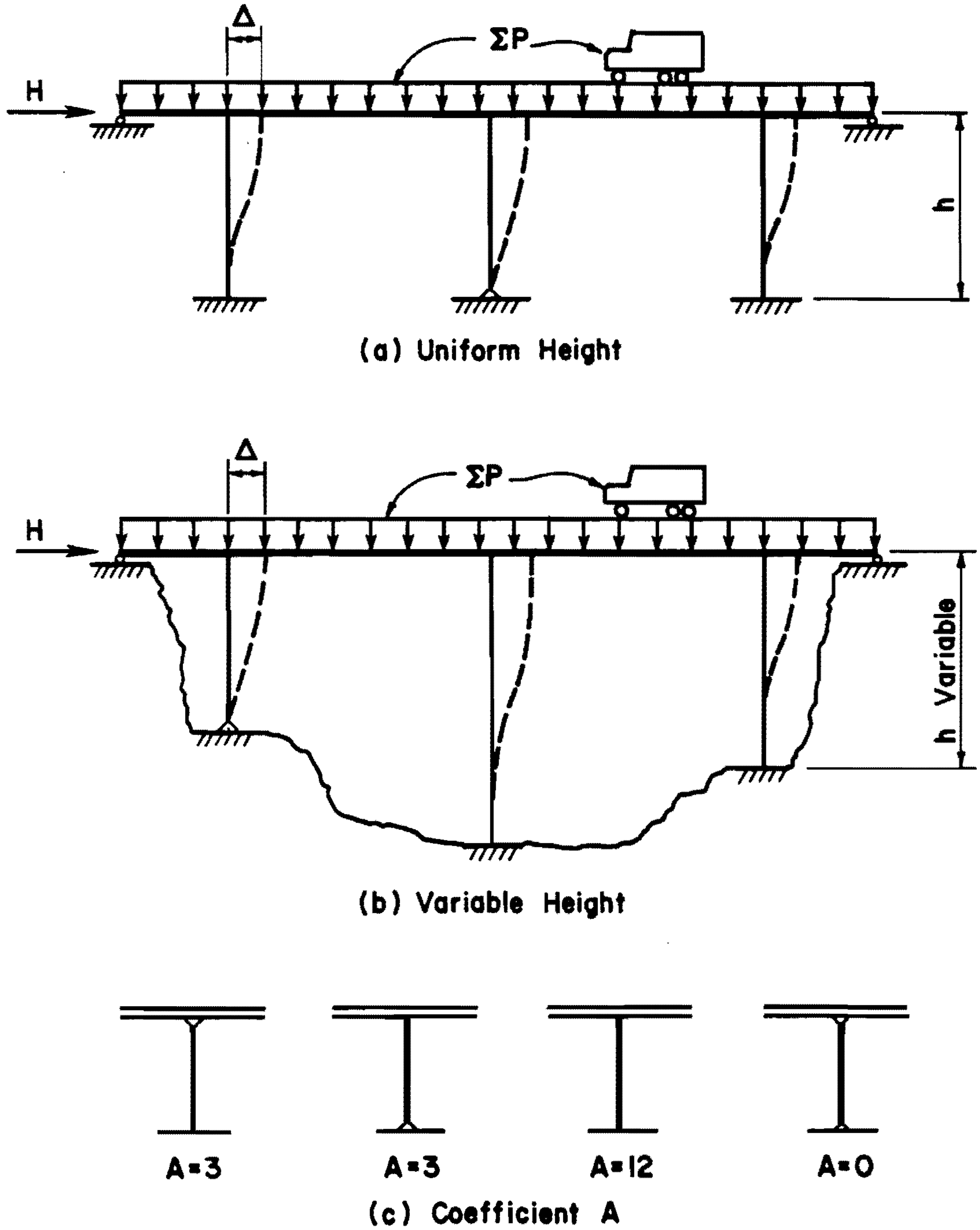


Fig. 3.7 Application of stability index

$$\text{(A)}\quad \Sigma P_{cr} = (P_{cr})_1 + (P_{cr})_2 + (P_{cr})_3 + \ldots$$

$$= \left[\frac{\pi^2 (EI)_1}{(k_1 h_1)^2} + \frac{\pi^2 (EI)_2}{(k_2 h_2)^2} + \frac{\pi^2 (EI)_3}{(k_3 h_3)^2} + \ldots\right] \quad (3\text{-}20)$$

The second approach uses Eq. (3-6). For this case B

$$\text{(B)}\quad \Sigma P_{cr} = \frac{1}{\gamma}\,[k_1 h_1 + k_2 h_2 + k_3 h_3 + \ldots] \quad (3\text{-}21)$$

$$= \frac{1}{\gamma}\left[\frac{A_1 (EI)_1}{(h_1)^2} + \frac{A_2 (EI)_2}{(h_2)^2} + \frac{A_3 (EI)_3}{(h_3)^2} + \ldots\right]$$

In this equation the value of γ is from 1.0 to 1.22 depending on relative stiffness and the values of the coefficient A_i depend on the pier end conditions as shown in Fig. 3.7(c). Either method is used to determine P_{cr}.

A normal frame analysis is run in which reasonable stiffnesses are input to model cracked, reinforced concrete. This analysis will yield the first order displacement Δ for the factored lateral load H.

At this stage an equivalent value of h can be determined from

$$\Sigma P_{cr} = \frac{K\,h}{\gamma} = \frac{\Delta_1\ h}{H\ \gamma} \quad (3\text{-}6)$$

$$h_e = \frac{\gamma\ \Sigma P_{cr}\ H}{\Delta_1}$$

Then Q can be computed as $P_u \Delta_1 / H h_e$ and the averaged magnifier found for each column.

Overall the Q method shows great promise for analysis of moderate slenderness unbraced systems. The procedure must consider the reinforced concrete properties at the factored load limit state in determination of relative stiffness, EI, creep effects, and all the other complications in the general problem. It is strictly a frame or member analysis tool and does not include cross section strength or dimensioning ability. However, by allowing use of more conventional first order elastic analysis programs for computation of Δ_1 values, it can simplify computations for some complex situations. It certainly should be allowed for a moderate slenderness range, say $\ell_u/r \leq 50$. Above that range the normal first order analysis needs to be modified to include the effect of axial load on stiffness and carryover factors. In addition, there are more possibilities of member instability in a braced mode and this method may be unreliable and unconservative.

One of the biggest advantages of the Q-method is that there is no need for absolute differentiation between "braced" and "unbraced" structures. Most structures lie somewhere in between. This method is also a wise way of judging the degree of "bracing" present for use of the moment magnification methods.

3.3.4 Biaxial Slenderness Effects. The R-method, moment-magnifier method, and stability index procedure are fairly straightforward for the uniaxial bending case. However, the various shortcomings of each method are amplified when considering biaxial bending. There have been many suggestions made for the shape of the interaction surface of solid sections from which, knowing the uniaxial strengths, the biaxial bending strengths may be calculated [15]. At present this surface is not accurately described, even for solid sections. No major attempts have been made for hollow and cellular sections. Few attempts have been made to treat biaxial bending in tapered, flared, or stepped members. The procedures outlined in Sections 3.3.1 - 3.3.3 for analysis of slender compression members are greatly complicated when biaxial bending is considered.

3.3.5 Computer Methods. A comprehensive computer analysis of slender piers which would be able to handle solid or hollow, prismatic or varying, uniaxial or biaxial loaded members or bents would need to include both section behavior and member behavior. The program must also be verified for both section and member behavior by comparison with experimentally measured data.

The most common technique which has been utilized in computer analyses of section and member behavior is an iterative search procedure [3,6,35,36,37,38]. In this procedure, there is first a determination of the relationship between axial load, moment, and curvature of a section. The plane of strains which results from the load and moment must be found. When the correct plane of strain has been determined, the curvature of the column at a particular point is known, thus the P-M-ϕ relationship. The deflected shape is then determined from the curvatures along the member. The final component of the solution is an iterative search for the deflected shape. The use of this method has been limited mainly to rectangular prismatic columns. This method would be difficult to extend to hollow or nonprismatic members.

Other possible methods are based on stiffness formulations which are easily adaptable for computer usage [3,11]. The methods are very general and can easily incorporate nonlinearities such as second order deflection effects. A general stiffness method would include formulating the stiffnesses of the members, assembly into a total system matrix, then solution of the equilibrium equations for the unknown displacements. Material nonlinearities such as for reinforced concrete would present problems if a general three-dimensional stiffness formulation was utilized. The three-dimensional constitutive relationships of concrete would need to be known, and at present they are not well-defined [13]. The main problem in using stiffness methods are the vast amounts of computer storage and time needed for a solution. One method, the fiber model, needs only a uniaxial compressive stress-strain definition of concrete and minimizes computer usage if some assumptions are made. This method will be discussed in a later section.

There are several state-of-the-art assumptions necessary for any method of computer analysis chosen [6,11,35,36,37]. The first is commonly used with engineering beam theory based on Navier's hypothesis that plane sections before bending remain plane after bending [12]. This assumption is valid provided shear distortions are not a problem. This has been shown true for solid sections, and was verified in the companion report for hollow and cellular sections with moderate to thick walls [18]. Another assumption is that the maximum strain of concrete does not vary as the shape of the compression area changes. This assumption was found not true by Rüsch [39]. A conservative lower bound value for the ultimate strain corresponding to a rectangular compression area should be utilized. There must also be an assumption about the effect of torsion on a member. Most methods ignore torsional effects altogether. Other methods only consider elastic torsional theory and include only St. Venant torsion. Other simplifying assumptions must be made about time-dependent effects, and confinement effects of transverse ties and moment gradients due to the complexity of the problems.

The following sections describe some of the methods utilized and available for axial load-moment-curvature analysis of a section, and load-deflection analysis of slender piers including those which are nonprismatic or hollow and which may be subject to biaxial loading.

3.3.5.1 Axial Load-Moment-Curvature (P-M-ϕ) Analysis of a Section. There are several possible methods for biaxial P-M-ϕ analysis of sections. The first is termed the "grid method" [3,9]. In this partitioned element method, discrete strips are divided parallel to the assumed neutral axis as in Fig. 3.8. Knowing the plane of strain of the strips, the resulting axial load and moments can be found. When the correct plane of strain is determined, the curvature of the section is known, thus the P-M-ϕ relationship.

Farah and Huggins described another procedure for predicting biaxial bending deformations [40]. This method is analogous to the previous grid method, except there are an infinite number of grids. Therefore, the solution is "exact." The procedure uses a set of equations to be solved simultaneously for the P-M-ϕ relationship. There could be some problems extending this procedure to hollow sections because of boundary conditions.

One final method is termed the "fiber model" [3]. This procedure is similar to the grid method, in that the section is discretized, but into fibers shown in Fig. 3.9. In this procedure, incremental curvatures about the x- and y-axes are applied and the associated changes in strain found. Incremental stiffness parameters are then found, from which the incremental moments about the x- and y-axes can be determined. This procedure is continued until the entire P-M-ϕ relationship of the section is known. The method is particularly well-suited for arbitrary cross sections such as hollow sections.

3.3.5.2 Load Deflection Analysis of a Member. Most of the procedures are extensions of the previous P-M-ϕ methods, and find the deformation of the column by elastic beam theory of integrating curvatures. There are some other methods available using stiffness formulations.

Brettle and Warner assumed a sine curve for the deflected shape of their single curvature column [41]. By numerically integrating curvatures, deflection of the column can be found. However, their procedure is limited to the column bent in single curvature, a situation rare in actual bridge piers.

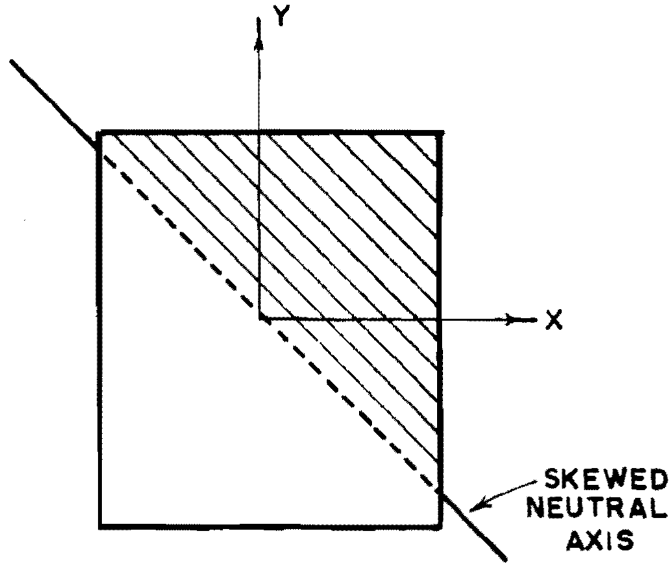


Fig. 3.8 Parallel discretization of grid method

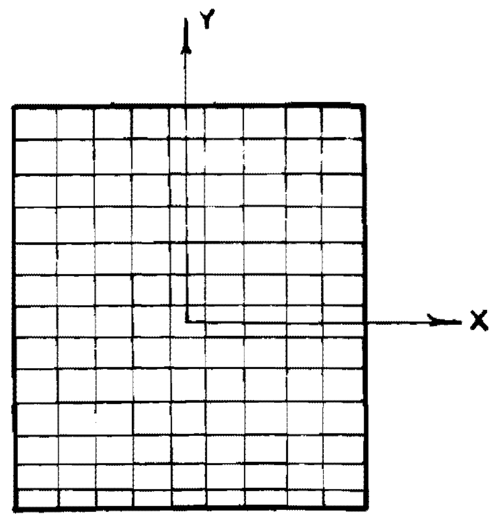


Fig. 3.9 Two-way discretization of fiber model

Farah and Huggins [40] and Redwine [42] used a closed-form solution for finding P-M-ϕ relationships of the section, then also integrated curvatures to find the deflected shape. If the deformation of the column found for the curvatures did not agree with that initially assumed, a new shape was assumed and the procedure repeated until the deformed shape was found. This procedure was only carried out for prismatic members.

Finite difference method (FDM) is another possible solution technique for a beam-column [43]. Bending equations of a member are approximated by finite difference equations which are expressed as a function of deflections at grid points. Problems encountered using the method are extensions of the equations from planar to space behavior, arbitrary or unsymmetrical loading, and imposed boundary conditions.

The finite element method (FEM) is the most general analytical technique available. The pier would be divided into subregions, the deformation patterns of which are assumed. When displacement compatibility is explicitly enforced at two nodal points, the displacements along the element interface connecting the two points will be compatible. Once the displacement assumptions are made, the element stiffness matrix may be derived, and the analysis is carried out by the direct stiffness method. The primary disadvantage associated with the finite element method is the large number of equilibrium equations to be solved. A conceptual finite element model of a prismatic rectangular solid pier is shown in Fig. 3.10. Each node point would have 6° of freedom. As can be seen, there would be many degrees of freedom to be solved. The finite element method generally requires vast computer storage and execution time for even small problems.

The final method is an extension of the fiber model discussed in the previous section [3]. In this procedure, the pier is divided into longitudinal segments, and the segments into

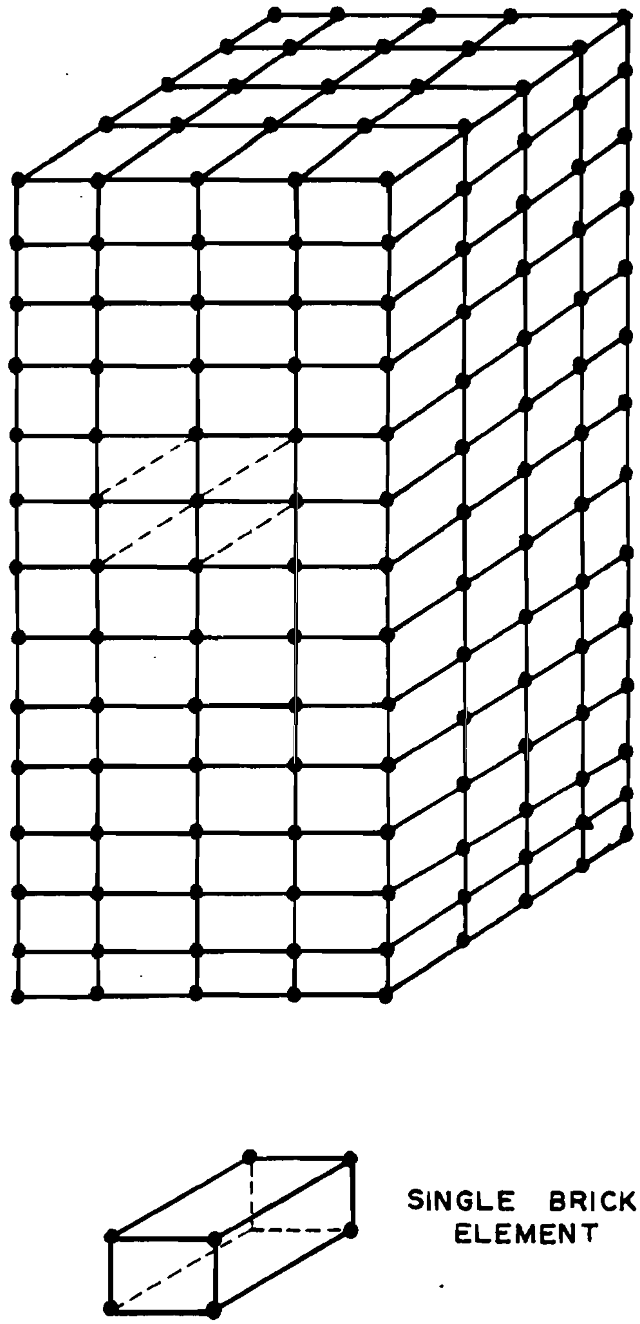


Fig. 3.10 Conceptual finite element model of a rectangular solid prismatic bridge pier

sections as shown in Fig. 3.11. Each section is then divided into fibers. The desired load for the pier is applied in increments. For each increment of load applied, the incremental displacements and forces are found. New stiffness parameters are found, and the procedure repeated for each increment of load. The stiffness of each section is placed in a total stiffness matrix for the pier, and the resulting equilibrium equations solved. The method is not "exact" but can be used easily for arbitrary cross sections and nonprismatic members.

3.4 Objectives of Computer Program Development

The state-of-the-art survey summarized in Chapter 2 showed increasing utilization of slender compression members becoming more common for very slender applications. It further indicated that biaxial bending needed to be considered for many applications of practical interest and that both isolated pier and multiple pier bents needed to be considered. A range of nonuniform shapes such as tapered, flared, and stepped compression members require design and analysis treatment.

The basic objective of the computer program development stage of this project was to develop a group of general analysis programs which would meet all requirements of AASHTO Specifications Art. 1.5.34(A)(1).

> 1.5.34 Slenderness Effects in Compression Members
> (A) General Requirements
> (1) Design of compression members shall be based on forces and moments determined from an analysis of the structure. Such an analysis shall take into account the influence of axial loads and variable moment of inertia on member stiffness and fixed-end moments, the effect of deflections on the moments and forces, and the effects of the duration of the loads.

Three such programs were developed and are outlined in detail in Chapter 5. Listings and user guides are given in Appendices. The

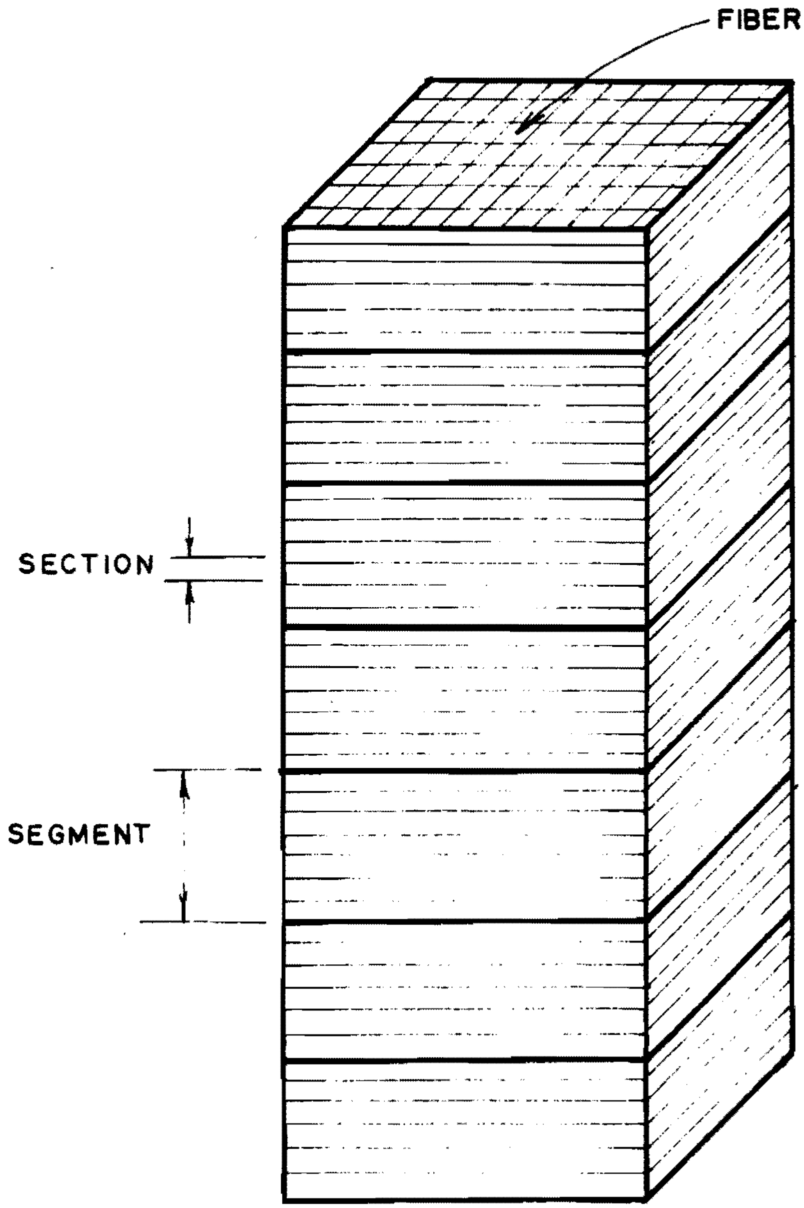


Fig. 3.11 Fiber model representation of a rectangular solid prismatic bridge pier

programs are verified by extensive comparison with experimental results in Ref. 18. The programs will at present handle rectangular solid, rectangular hollow, rectangular cellular, circular solid, circular hollow and oval cross sections. The programs are easily adaptable to any arbitrary cross section. The programs handle single pier bent of arbitrary longitudinal cross section variation as well as multiple bay, multiple story pier bents.

The model selected for the study is the "fiber model" introduced in the previous sections and treated in detail in Chapter 4. It was chosen because of its adaptability to handle arbitrary cross sections and arbitrary longitudinal configurations. With this method, the nonlinear characteristics of concrete and column slenderness effects can be easily handled. There are various computer analyses available to handle the elastic beam-column, but the treatment of arbitrary cross section and arbitrary longitudinal configurations of reinforced concrete piers or multiple pier bents seems unique to this report.

CHAPTER 4

FIBER MODEL FORMULATION

4.1 Introduction

The basic analytical model selected in this study uses a tangent stiffness formulation [3]. Incremental loads are applied to the structure, then the incremental displacements and forces are calculated and added to the previous displacements and forces to obtain the new displacements and forces. The stiffness method is used to calculate the incremental displacements from the applied loads, and the incremental forces are then computed from the displacements. The model uses a linear formulation which has been modified to include various nonlinear effects. The nonlinearities included in the study are second-order deflection effects (P-delta), geometric nonlinearity, and material nonlinearity.

In the analysis of the idealized bridge pier, it is assumed to be made of many prismatic segments assembled together. The actual pier may be nonprismatic, but is represented by discrete prismatic segments. This concept is illustrated in Fig. 4.1.

4.2 Derivation of the Stiffness Matrix

To obtain the incremental force-displacement relationship, consider a member loaded only at its ends with the forces and displacements assumed as in Figs. 4.2 and 4.3. For the assumption that small changes in strain can be related linearly to small changes in member force, the following relationships hold:

$$\Delta N = \int_{\text{cross section}} E_t \; \Delta\epsilon \tag{4.1}$$

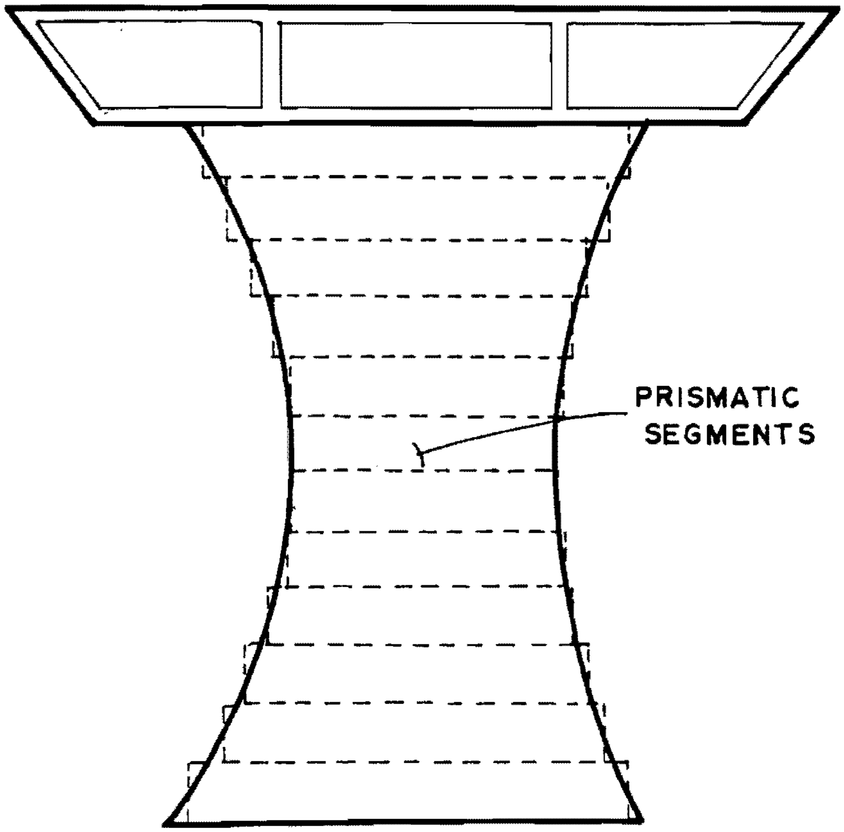


Fig. 4 .1 Assembly of prismatic members representing a nonprismatic pier

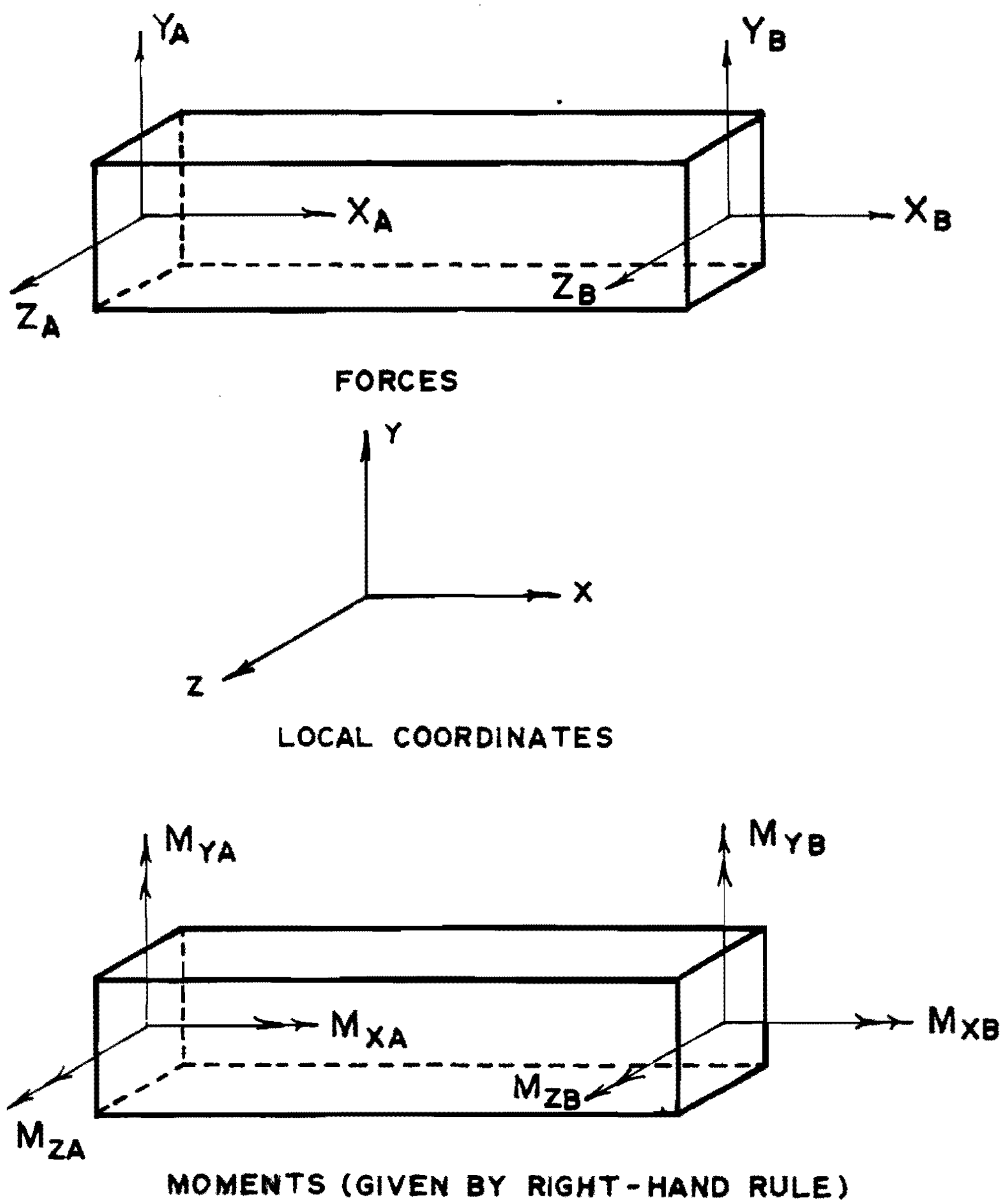


Fig. 4.2 Forces and moments acting on member

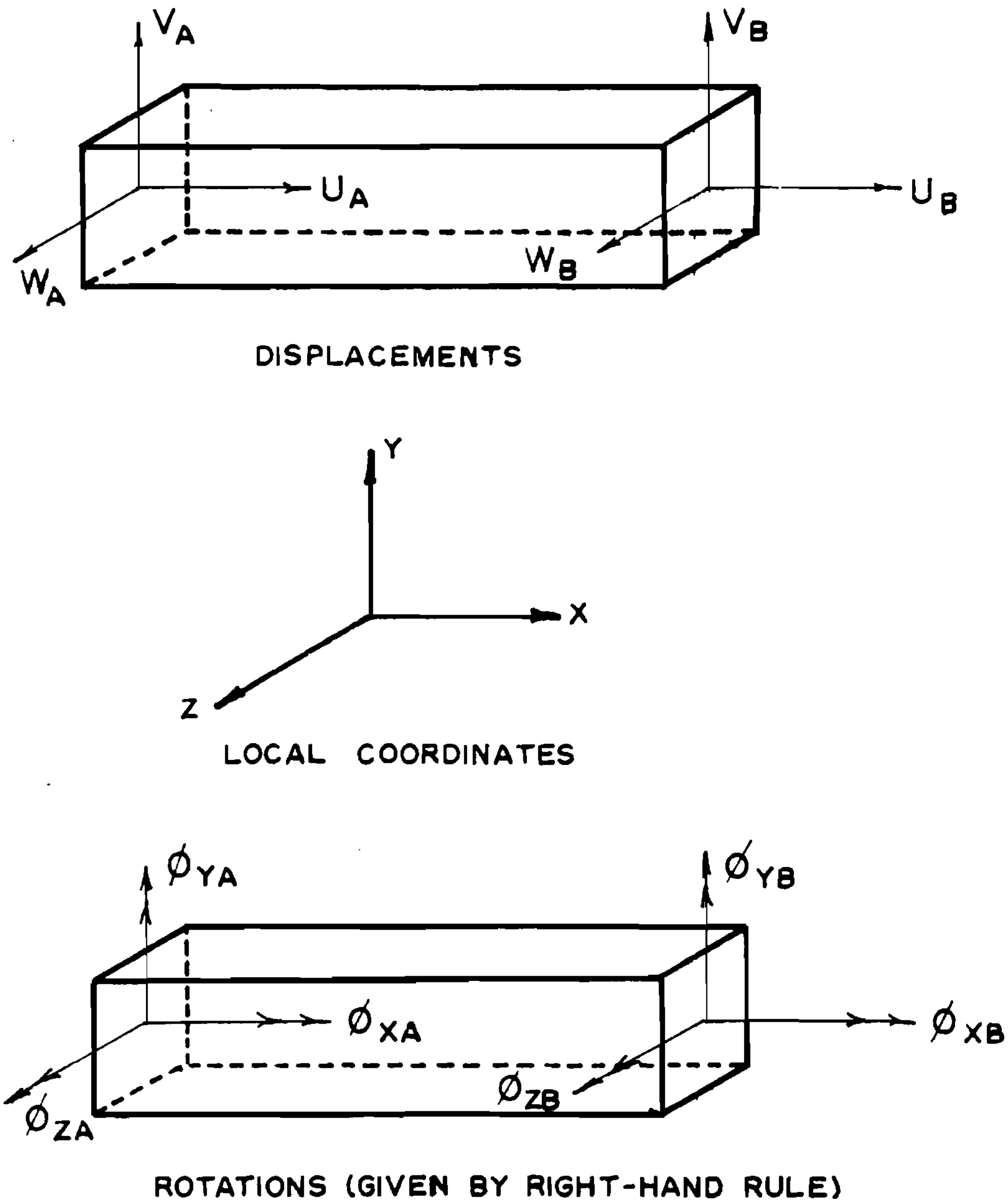


Fig. 4.3 Displacements and rotations of member

$$\Delta M_y = \int_{\text{cross section}} z\ E_t\ \Delta\epsilon \tag{4.2}$$

$$\Delta M_z = -\int_{\text{cross section}} y\ E_t\ \Delta\epsilon \tag{4.3}$$

where ΔN = increment in axial force

ΔM_y = increment in moment about the y-axis

ΔM_z = increment in moment about the z-axis

$\Delta\epsilon$ = increment in strain

E_t = tangent modulus of the section

z = distance to differential element in z-direction

y = distance to differential element in y-direction

The change in strain, $\Delta\epsilon$, is a function of two directions and given by the following:

$$\Delta\epsilon(y,z) = \Delta\epsilon_o + z\ \Delta\ \varphi_y' - y\ \Delta\ \varphi_z' \tag{4.4}$$

where $\Delta\epsilon_o$ = increment in axial strain

$\Delta\varphi_y'$ = increment in rotational strain about the y-axis

$\Delta\varphi_z'$ = increment in rotational strain about the z-axis

This expression can be substituted into Eqs. (4.1), (4.2), and (4.3) to give the following expressions:

$$\Delta N = \int_{\text{cross section}} E_t\ (\Delta\epsilon_o + z\Delta\varphi_y' - y\Delta\varphi_z') \tag{4.5}$$

$$\Delta M_y = \int_{\text{cross section}} zE_t\ (\Delta\epsilon_o + z\Delta\varphi_y' - y\Delta\varphi_z') \tag{4.6}$$

$$\Delta M_z = -\int_{\text{cross section}} yE_t \quad (\Delta\epsilon_o + z\Delta\varphi'_y - y\Delta\varphi'_z) \tag{4.7}$$

If the member cross section is divided into finite pieces or "fibers," the integrals can be replaced with the discrete summations.

$$\Delta N = \sum_{\text{fibers}} E_t(\Delta\epsilon_o + z\Delta\varphi'_y - y\Delta\varphi'_z) \tag{4.8}$$

$$\Delta M_y = \sum_{\text{fibers}} zE_t(\Delta\epsilon_o + z\Delta\varphi'_y - y\Delta\varphi'_z) \tag{4.9}$$

$$\Delta M_z = -\sum_{\text{fibers}} yE_t(\Delta\epsilon_o + z\Delta\varphi'_y - y\Delta\varphi'_z) \tag{4.10}$$

These equations can be simplified further to

$$\Delta N = a_{11}\Delta\epsilon_o + a_{12}\Delta\varphi'_y + a_{13}\Delta\varphi'_z \tag{4.11}$$

$$\Delta M_y = a_{21}\varphi\epsilon_o + a_{22}\Delta\varphi'_y + a_{23}\Delta\varphi'_z \tag{4.12}$$

$$\Delta M_z = a_{31}\Delta\epsilon_o + a_{32}\Delta\varphi'_y + a_{33}\Delta\varphi'_z \tag{4.13}$$

where

$$a_{11} = \sum_{\text{fibers}} E_t$$

$$a_{12} = \sum_{\text{fibers}} zE_t$$

$$a_{13} = -\sum_{\text{fibers}} yE_t$$

$$a_{21} = \sum_{\text{fibers}} zE_t$$

$$a_{22} = \sum_{\text{fibers}} z^2E_t$$

$$a_{23} = -\sum_{\text{fibers}} zyE_t$$

$$a_{31} = -\sum_{\text{fibers}} yE_t$$

$$a_{32} = -\sum_{\text{fibers}} zyE_t$$

$$a_{33} = \sum_{\text{fibers}} y^2E_t$$

Inversion of the incremental force-strain equations gives the following strain-force equations.

$$\Delta\epsilon_o = \Delta u' = b_{11}\Delta N + b_{12}\Delta M_y + b_{13}\Delta M_z \tag{4.14}$$

$$\Delta\varphi' = -\Delta w'' = b_{21}\Delta N + b_{22}\Delta M_y + b_{23}\Delta M_z \tag{4.15}$$

$$\Delta\varphi_z' = \Delta v'' = b_{31}\Delta N + b_{32}\Delta M_y + b_{33}\Delta M_z \tag{4.16}$$

where

$$b_{11} = \frac{a_{22}a_{33}-a_{23}a_{32}}{\text{Det A}} \qquad b_{12} = \frac{a_{13}a_{32}-a_{12}a_{33}}{\text{Det A}} \qquad b_{13} = \frac{a_{11}a_{23}-a_{13}a_{21}}{\text{Det A}}$$

$$b_{21} = \frac{a_{23}a_{31}-a_{21}a_{33}}{\text{Det A}} \qquad b_{22} = \frac{a_{11}a_{33}-a_{13}a_{31}}{\text{Det A}} \qquad b_{23} = \frac{a_{13}a_{21}-a_{13}a_{21}}{\text{Det A}}$$

$$b_{31} = \frac{a_{21}a_{32}-a_{22}a_{31}}{\text{Det A}} \qquad b_{32} = \frac{a_{12}a_{31}-a_{11}a_{32}}{\text{Det A}} \qquad b_{33} = \frac{a_{11}a_{22}-a_{12}a_{21}}{\text{Det A}}$$

$$\text{Det A} = \begin{vmatrix} a_{11} & a_{12} & a_{13} \\ a_{21} & a_{22} & a_{23} \\ a_{31} & a_{32} & a_{33} \end{vmatrix}$$

$$= a_{11}a_{22}a_{33} + a_{12}a_{23}a_{31} + a_{13}a_{21}a_{32}$$

$$-\left[a_{13}a_{22}a_{31} + a_{11}a_{23}a_{32} + a_{12}a_{21}a_{33}\right]$$

For the member shown in Fig. 4 4, the incremental forces in terms of end B forces at a section (L - x) away are given by

$$\Delta N = \Delta x_B \tag{4.17}$$

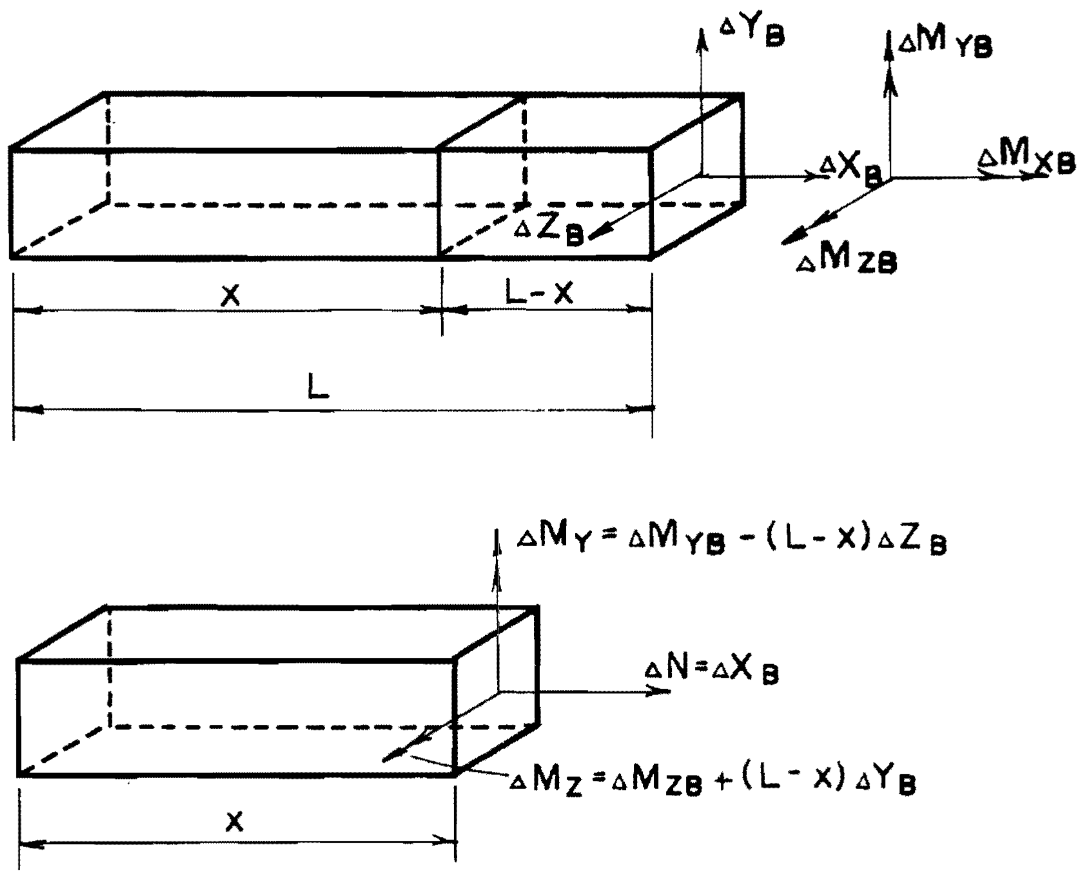


Fig. 4.4 Relationship of section forces in terms of end B forces

$$\Delta M_y = \Delta M_{yB} - (L - x)\Delta z_b \tag{4.18}$$

$$\Delta M_z = \Delta M_{zB} + (L - x)\Delta y_B \tag{4.19}$$

If these expressions are substituted into Eqs. (4.14), (4.15), and (4.16), the following equations result.

$$\Delta u' = b_{11}\Delta x_B + (L - x)b_{13}\Delta y_B - (L - x)b_{12}\Delta z_B + b_{12}\Delta M_{yB} + b_{13}\Delta M_{zB} \tag{4.20}$$

$$\Delta v'' = b_{31}\Delta x_B + (L - x)b_{33}\Delta y_B - (L - x)b_{32}\Delta z_B + b_{32}\Delta M_{yB} + b_{33}\Delta M_{zB} \tag{4.21}$$

$$\Delta w'' = -b_{21}\Delta x_B - (L - x)b_{23}\Delta y_B + (L - x)b_{22}\Delta z_B - b_{22}\Delta M_{yB} - b_{23}\Delta M_{zB} \tag{4.22}$$

where $\Delta u'$ = axial strain

$\Delta v''$ = curvature about the z-axis

$\Delta w''$ = curvature about the y-axis

Integrating, $\Delta u'$ once and $\Delta v''$ and $\Delta w''$ twice, over the length of the member gives the following relationships:

$$\Delta u = \int \Delta u' dx \qquad \Delta u_B = \int_0^L \Delta u' dx \tag{4.23}$$

$$\Delta v' = \Delta\varphi_z = \int \Delta v'' dx \qquad \Delta\varphi_{zB} = \int_0^L \Delta v'' dx \tag{4.24}$$

$$\Delta w' = -\Delta\varphi_y = -\int \Delta w'' dx \qquad \Delta\varphi_{yB} = -\int_0^L \Delta w'' dx \tag{4.25}$$

$$\Delta v = \iint \Delta v'' dxdx \qquad \Delta v_B = \int_0^L \int_0^x \Delta v'' \, dxdx \tag{4.26}$$

$$\Delta w = \iint \Delta w'' \, dxdx \qquad \Delta w_B = \int_0^L \int_0^x \Delta w'' dxdx \tag{4.27}$$

Figure 4.5 shows the resulting displacement-force relationship in matrix form. This matrix represents the flexibility of the member at end B for displacements imposed at end B, and is given by f_{BB}. The stiffness is found by inversion of the flexibility matrix.

$$K = f^{-1} \tag{4.28}$$

From the displacement-force relationship written in matrix notation,

$$\Delta U = f \Delta F \tag{4.29}$$

the force-displacement relationship is found by multiplying both sides of the equation by f^{-1}

$$f^{-1} \Delta U = f^{-1} f \Delta F \tag{4.30}$$

$$K \Delta U = \Delta F \tag{4.31}$$

From the flexibility matrix f_{BB}, the final force-displacement relationship of the member can be found. The equilibrium equations of the member are:

$$\Delta x_A = -\Delta x_B \tag{4.32}$$

$$\Delta y_A = -\Delta y_B \tag{4.33}$$

$$\Delta z_A = -\Delta z_B \tag{4.34}$$

$$\Delta M_{xA} = -\Delta M_{xB} \tag{4.35}$$

TABLE 4.1 DISPLACEMENT-FORCE RELATIONSHIP IN MATRIX FORM

$$
\begin{bmatrix}
\int_0^L b_{11}dx & \int_0^L (L-x)b_{13}dx & -\int_0^L (L-x)b_{12}dx & 0 & \int_0^L b_{12}dx & \int_0^L b_{13}dx \\
\int_0^L\int_0^x (b_{31}dx)dx & \int_0^L\int_0^x ((L-x)b_{33}dx)dx & -\int_0^L\int_0^x ((L-x)b_{32}dx)dx & 0 & \int_0^L\int_0^x (b_{32}dx)dx & \int_0^L\int_0^x (b_{33}dx)dx \\
-\int_0^L\int_0^x (b_{21}dx)dx & -\int_0^L\int_0^x ((L-x)b_{23}dx)dx & \int_0^L\int_0^x ((L-x)b_{22}dx)dx & 0 & -\int_0^L\int_0^x (b_{22}dx)dx & -\int_0^L\int_0^x (b_{23}dx)dx \\
0 & 0 & 0 & \frac{L}{GJ} & 0 & 0 \\
\int_0^L b_{21}dx & \int_0^L (L-x)b_{23}dx & -\int_0^L (L-x)b_{22}dx & 0 & \int_0^L b_{22}dx & \int_0^L b_{23}dx \\
\int_0^L b_{31}dx & \int_0^L (L-x)b_{33}dx & -\int_0^L (L-x)b_{32}dx & 0 & \int_0^L b_{32}dx & \int_0^L b_{33}dx
\end{bmatrix}
\begin{pmatrix} \Delta x_B \\ \Delta y_B \\ \Delta z_B \\ \Delta M_{xB} \\ \Delta M_{yB} \\ \Delta M_{zB} \end{pmatrix}
\begin{pmatrix} \Delta u_B \\ \Delta v_B \\ \Delta w_B \\ \Delta \varphi_{xB} \\ \Delta \varphi_{yB} \\ \Delta \varphi_{zB} \end{pmatrix}
$$

$$\Delta M_{yA} = -\Delta M_{yB} + L\Delta z_B \tag{4.36}$$

$$\Delta M_{zA} = -M_{zB} - L\Delta y_B \tag{4.37}$$

This can be written in matrix notation

$$\begin{Bmatrix} \Delta x_A \\ \Delta y_A \\ \Delta z_A \\ \Delta M_{xA} \\ \Delta M_{yA} \\ \Delta M_{zA} \end{Bmatrix} = \begin{bmatrix} -1 & 0 & 0 & 0 & 0 & 0 \\ 0 & -1 & 0 & 0 & 0 & 0 \\ 0 & 0 & -1 & 0 & 0 & 0 \\ 0 & 0 & 0 & -1 & 0 & 0 \\ 0 & 0 & L & 0 & -1 & 0 \\ 0 & -L & 0 & 0 & 0 & -1 \end{bmatrix} \begin{Bmatrix} \Delta x_B \\ \Delta y_B \\ \Delta z_B \\ \Delta M_{xB} \\ \Delta M_{yB} \\ \Delta M_{zB} \end{Bmatrix}$$

and symbolically as

$$\Delta F_A = -T^T \Delta F_B \tag{4.38}$$

The compatibility equations of the member are

$$\Delta u_B = \Delta u_A \tag{4.39}$$

$$\Delta v_B = \Delta v_A + \Delta\varphi_{zA} L \tag{4.40}$$

$$\Delta w_B = \Delta w_A - \Delta\varphi_{yA} L \tag{4.41}$$

$$\Delta\varphi_{xB} = \Delta\varphi_{xA} \tag{4.42}$$

$$\Delta\varphi_{yB} = \Delta\varphi_{yA} \tag{4.43}$$

$$\Delta\varphi_{zB} = \Delta\varphi_{zA} \tag{4.44}$$

This can be written in matrix notation as

$$\begin{Bmatrix} \Delta u_B \\ \Delta v_B \\ \Delta w_B \\ \Delta\varphi_{xB} \\ \Delta\varphi_{yB} \\ \Delta\varphi_{zB} \end{Bmatrix} = \begin{bmatrix} 1 & 0 & 0 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 & 0 & L \\ 0 & 0 & 1 & 0 & -L & 0 \\ 0 & 0 & 0 & 1 & 0 & 0 \\ 0 & 0 & 0 & 0 & 1 & 0 \\ 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix} \begin{Bmatrix} \Delta u_A \\ \Delta v_A \\ \Delta w_A \\ \Delta\varphi_{xA} \\ \Delta\varphi_{yA} \\ \Delta\varphi_{zA} \end{Bmatrix}$$

and symbolically as

$$\Delta U_B = TU_A \tag{4.45}$$

From the previous equations the following can be written

$$f_{BB}F_{BB} = \Delta U_B - T\Delta U_A \tag{4.46}$$

$$\Delta F_B = K_{BB}(\Delta U_B - T\Delta U_A) \tag{4.47}$$

The force at end A is given by Eq. (4.38). Substituting Eq. (4.47) in for ΔF_B yields

$$\Delta F_A = -T^T K_{BB}(\Delta U_B - T\Delta U_A) \tag{4.48}$$

The final incremental force-deformation relationship looks like

$$\begin{Bmatrix} \Delta F_A \\ \Delta F_B \end{Bmatrix} = \begin{bmatrix} T^T K_{BB} T & -T^T K_{BB} \\ -K_{BB} T & K_{BB} \end{bmatrix} \begin{Bmatrix} \Delta U_A \\ \Delta U_B \end{Bmatrix}$$

or in the standard form

$$\begin{Bmatrix} \Delta F_A \\ \Delta F_B \end{Bmatrix} = \begin{bmatrix} K_{AA} & K_{AB} \\ K_{BA} & K_{BB} \end{bmatrix} \begin{Bmatrix} \Delta U_A \\ \Delta U_B \end{Bmatrix}$$

In equation form

$$\Delta F = K \Delta U \tag{4.49}$$

where K = stiffness matrix of member.

The analysis is performed by numerically forming the flexibility matrix f_{BB}, inverting it to form the stiffness matrix K_{BB}, then computing K_{AB}, K_{BA}, and K_{AA}, for each section. The member or segment stiffness matrix is added to the total pier stiffness matrix.

The stiffness matrix derived is for a horizontal member as shown in Fig. 4.5a. It is convenient to rotate this member to a specified global coordinate system which rotates the member to a vertical position. The rotation of the member is illustrated in Fig. 4.5b. The local x-axis transforms into the global y-axis, the local y-axis transforms into the global z-axis, and the local z-axis transforms into the global x-axis. If a rotation matrix R is defined as

$$R = \begin{bmatrix} 0 & 1 & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 \\ 1 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 1 & 0 \\ 0 & 0 & 0 & 0 & 0 & 1 \\ 0 & 0 & 0 & 1 & 0 & 0 \end{bmatrix}$$

then Eq. (4.49) in global axes is given by

$$R^T \Delta F = R^T K R \Delta U \tag{4.50}$$

which is equivalent to

$$\Delta F_g = K \Delta U g \tag{4.51}$$

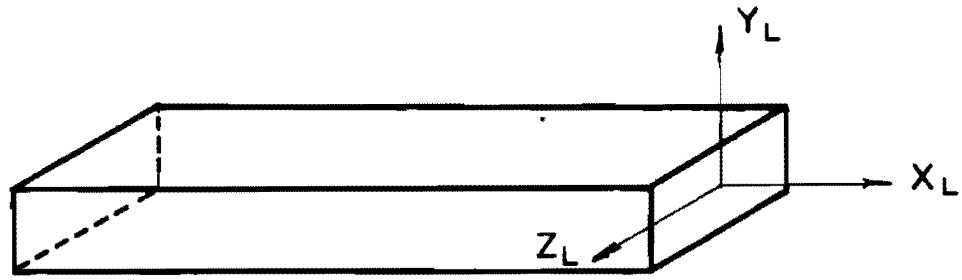


Fig. 4.5a Member in local axes

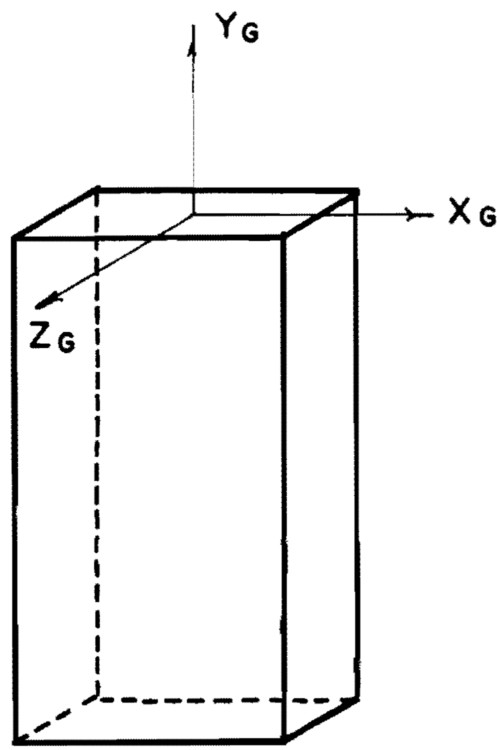


Fig. 4.5b Member in global axes

where ΔF_g = incremental forces in global axes

ΔU_g = incremental displacements in global axes

This rotation was not performed each time since it is the same. The stiffness matrix of each section of the pier is derived in terms of the global axes as shown in Fig. 4.5b. Therefore, the displacements and forces determined are in the global axes.

4.3 P-delta (PΔ) Effect

The P-delta effect represents the correction necessary for satisfying equilibrium in the deformed position[3]. Since the linear stiffness matrix is assembled in the initial position, the additional forces caused by the axial load moving as the structure deforms are not considered in the linear model unless the P-delta effect is included.

Assume a member is initially oriented as shown in Fig. 4.6a along the line AB with axial load N acting as shown. If due to the applied forces the member deforms as in Fig. 4.6b, additional forces are produced since the axial load moves through displacements. In order to satisfy equilibrium about point A, the following equations must be true.

$$N(\Delta u_B - \Delta u_A) = DFX \times L \tag{4.52}$$

$$N(\Delta w_B - \Delta w_A) = DFZ \times L \tag{4.53}$$

Thus, the change in the forces in the x and z axes are

$$DFX = \frac{(\Delta u_B - \Delta u_A)N}{L} = \frac{N}{L}\Delta u \tag{4.54}$$

$$DFZ = \frac{(\Delta w_B - \Delta w_A)N}{L} = \frac{N\Delta w}{L} \tag{4.55}$$

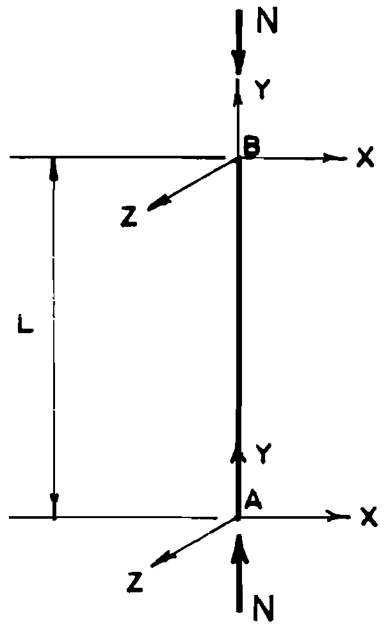


Fig. 4.6a Undeformed member

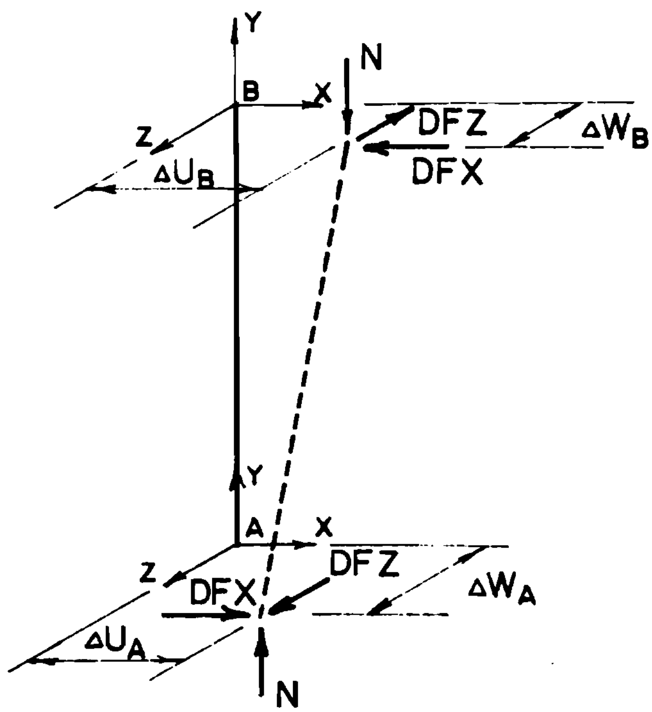


Fig. 4.6b Deformed member

These P-delta corrections can be applied to the global fixed end force vector or the member stiffness matrix. It was chosen to correct the stiffness matrix because it provides a more accurate amplification of the axial load as it approaches the stability limit [3]. Figure 4.7 shows the global stiffness matrix of a member and the appropriate locations of the P-delta correction. Each member stiffness is corrected and placed in the total stiffness matrix of the pier. This implies the nonlinearity will be the summation of all individual member effects to the base.

When correcting the member stiffness matrix the value of the incremental displacement can become negative, because the incremental change in stiffness can be negative once the critical load is exceeded. The results then are no longer valid.

4.4 Change in Geometry

This nonlinear effect is also caused by considering the linear stiffness matrix in the initial position [9]. If loads are applied on the assembly of segments or members of the pier, displacements and rotations will result. For the next increment of load, the incremental force-displacement equations should be corrected for the new position of each segment. This is done by rotating the displacements of each segment of the pier to the global axes of the structure.

In the linear model assumed, the change in geometry is handled by changing the joint coordinates of each segment of the pier at every increment [3]. Thus, at every step the joint displacements are added to the previous coordinates, and a new set of rotation matrices and lengths are computed for each segment of the pier. This procedure makes no allowance for individual member or segment curvature, hence the only way to reproduce pier deformation accurately is to subdivide the pier into many segments. The linear model assumes a straight line between joints; therefore, the

$$
\left[\begin{array}{cccccc|cccccc}
b_{11AA}+\frac{N}{L} & \cdot & \cdot & \cdot & \cdot & \cdot & b_{11AB}-\frac{N}{L} & \cdot & \cdot & \cdot & \cdot & \cdot \\
\cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot \\
\cdot & \cdot & b_{33AA}+\frac{N}{L} & \cdot & \cdot & \cdot & \cdot & \cdot & b_{33AB}-\frac{N}{L} & \cdot & \cdot & \cdot \\
\cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot \\
\cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot \\
\cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot \\
\hline
b_{11BA}-\frac{N}{L} & \cdot & \cdot & \cdot & \cdot & \cdot & b_{11BB}+\frac{N}{L} & \cdot & \cdot & \cdot & \cdot & \cdot \\
\cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot \\
\cdot & \cdot & b_{33BA}-\frac{N}{L} & \cdot & \cdot & \cdot & \cdot & \cdot & b_{33BB}+\frac{N}{L} & \cdot & \cdot & \cdot \\
\cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot \\
\cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot \\
\cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot
\end{array}\right]
\left[\begin{array}{c}
\Delta u_A \\ \Delta v_A \\ \Delta w_A \\ \Delta\varphi_{xA} \\ \Delta\varphi_{yA} \\ \Delta\varphi_{zA} \\ \Delta u_B \\ \Delta v_B \\ \Delta w_B \\ \Delta\varphi_{xB} \\ \Delta\varphi_{yB} \\ \Delta\varphi_{zB}
\end{array}\right]
=
\left[\begin{array}{c}
\Delta x_A \\ \Delta y_A \\ \Delta z_A \\ \Delta M_{xA} \\ \Delta M_{yA} \\ \Delta M_{zA} \\ \Delta x_B \\ \Delta y_B \\ \Delta z_B \\ \Delta M_{xB} \\ \Delta M_{yB} \\ \Delta M_{zB}
\end{array}\right]
$$

Fig. 4.7 Location of P-delta correction in stiffness matrix of member change in geometry in the model only considers the effect of the pier as a whole and not individual segment geometry.

Consider the member or segment shown in Fig. 4.8a. In this figure x-y-z represents the global axes of the structure. If the member deforms through an angle θ_z, shown in Fig. 4.8b where $\theta_z = \Delta u/L$, then the relationships between the global displacements and the new displacements are

$$u = u_1 + v_1\theta_z \tag{4.56}$$

$$v = -u_1\theta_z + v_1 \tag{4.57}$$

$$w = w_1 \tag{4.58}$$

If the member now deforms from the previous displaced position through an angle θ_x, shown in Fig. 4.8c where $\theta_x = \Delta w/L$, the relationships between the new deformed position and the previous deformed position are

$$u_1 = u_2 \tag{4.59}$$

$$v_1 = v_2 - w_2\theta_x \tag{4.60}$$

$$w_1 = v_2\theta_x + w_2 \tag{4.61}$$

Substituting these values of u_1, v_1, and w_1 into Eqs. (4.56), (4.57), and (4.58) yield

$$u = u_2 + v_2\theta_z \tag{4.62}$$

$$v = -u_2\theta_z + v_2 - w_2\theta_x \tag{4.63}$$

$$w = v_2\theta_x + w_2 \tag{4.64}$$

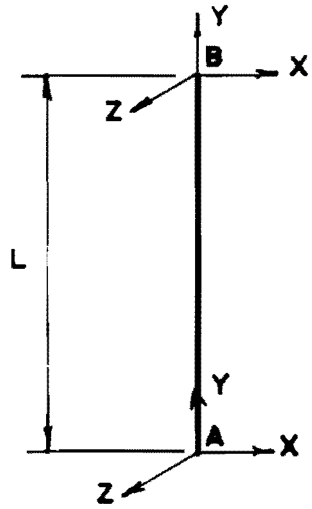


Fig. 4.8a Undeformed member

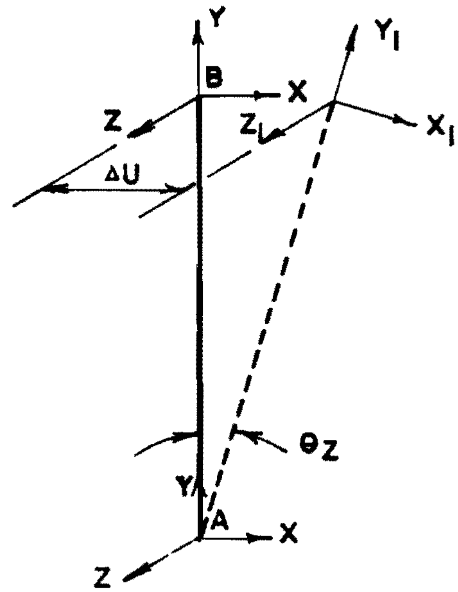


Fig. 4.8b Member deformed through angle θ_z

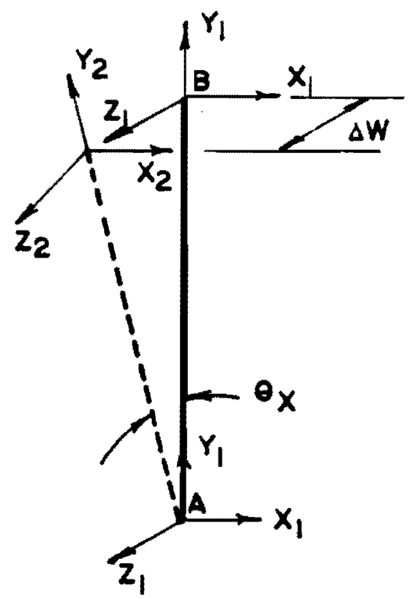


Fig. 4.8c Member deformed through angle θ_x

representing the displacements in the global axes in terms of the incremental deformed position. The rotation distortions are rotated to the global axes in the same manner. The transformation of coordinates can be expressed in matrix form as

$$\begin{Bmatrix} \Delta u_g \\ \Delta v_g \\ \Delta w_g \\ \Delta\varphi_{xg} \\ \Delta\varphi_{yg} \\ \Delta\varphi_{zg} \end{Bmatrix} = \begin{bmatrix} 1 & \theta_z & 0 & 0 & 0 & 0 \\ -\theta_z & 1 & -\theta_x & 0 & 0 & 0 \\ 0 & \theta_x & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & 1 & \theta_z & 0 \\ 0 & 0 & 0 & -\theta_z & 1 & -\theta_x \\ 0 & 0 & 0 & 0 & \theta_x & 1 \end{bmatrix} \begin{Bmatrix} \Delta u_L \\ \Delta v_L \\ \Delta w_L \\ \Delta\varphi_{xL} \\ \Delta\varphi_{yL} \\ \Delta\varphi_{zL} \end{Bmatrix}$$

or

$$U_g = RU_L$$

where U_g = global displacements

R = rotation matrix

U_L = local displacements

The corrected stiffness matrix for the geometry effect is given by

$$K_g = RK_LR^T \tag{4.65}$$

This correction will be significant for very flexible members.

4.5 Material Nonlinearity

The final nonlinearity considered in the fiber model is material nonlinearity. This is easy to handle in the fiber model. For a strain of a given fiber, a tangent stiffness is determined. The value of strain is entered into the given function of the stress-strain relationship of that particular type of fiber, and the derivative of that function or slope at that point is the tangent stiffness. This concept is illustrated in Fig. 4.9. The tangent

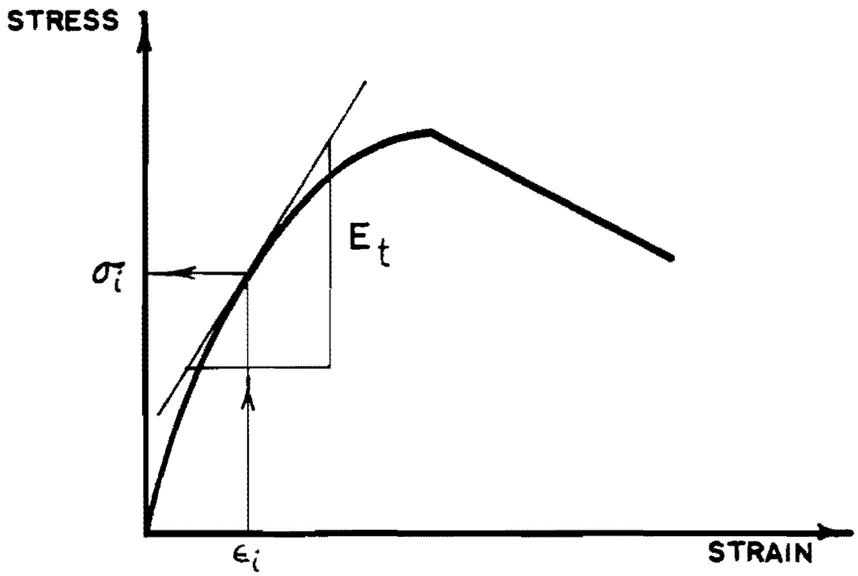


Fig. 4.9 Tangent stiffness of a fiber

stiffness of each fiber is used in computing the incremental stiffness of each member in each increment of load.

4.5.1 Modeling of Materials. For each fiber in the pier and its given strain, the stress and tangent modulus of that fiber must be found. This information is obtained from the appropriate functions describing the stress-strain relationship for that type of fiber. In the pier models in this study there are two types of fibers, concrete and reinforcing steel. The stress-strain relationship is considered the same over the entire fiber.

4.5.1.1 Formation of Flexural Cracks in Reinforced Concrete Members. An important consideration needed in an analysis of slender compression members is the influence of cracking of the concrete on the stiffness of the member. The formation of cracks is an important factor that greatly influences the behavior of a reinforced concrete member. Cracking occurs in the tension zone of a member subjected to flexure or axial tension. Cracks are formed when the stress in the concrete subjected to a loading exceeds the tensile stress of the concrete. The cracks may form perpendicular to the axis of the member from flexure or axial tension if there is no significant shear. When the shear is significant, cracks may form inclined to the member axis.

Consider the axially loaded tension member in Fig. 4.10 [15]. Cracks will form when the tensile strength of the concrete is exceeded at weak sections distributed at random. At the cracks, the concrete has no stress, the reinforcing steel carrying the load alone. Tensile stress is present in the concrete between the cracks due to bond between the concrete and reinforcing steel. The magnitude and distribution of the bond between the cracks determines the distribution of tensile stresses in the concrete and steel between the cracks. There can be additional cracking between the

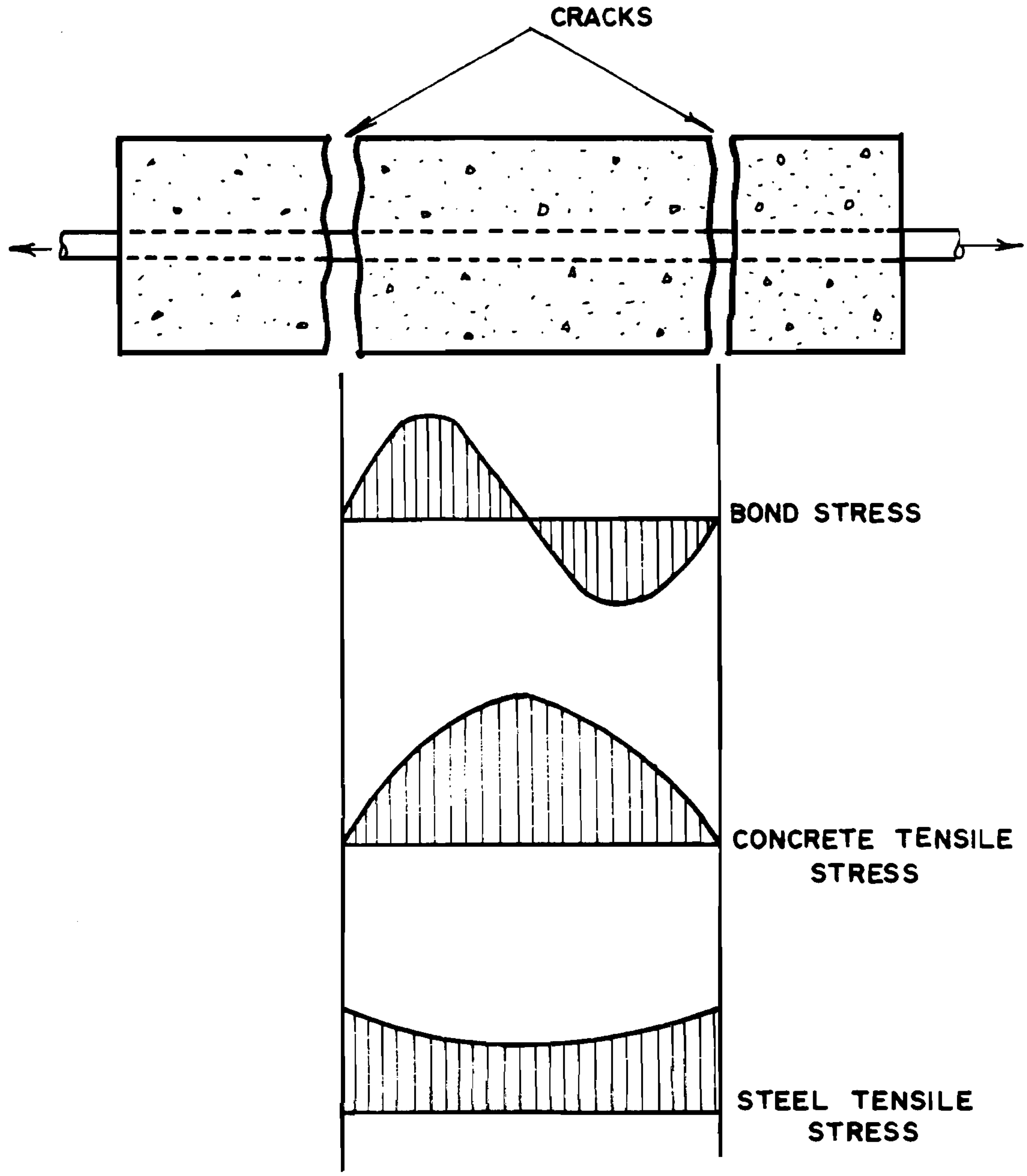


Fig. 4.10 Cracking of a member with axial tension [15]

initial cracks at higher load when the tensile strength is exceeded at those sections [15,16].

The approach used in this study for the cracking behavior in reinforced concrete members follows that presented by Alani [4] and was initially developed by Borges et al. [6,7,8]. The following equation describes the mean distance of cracks when concrete strains in the region of tensile stress between cracks and shrinkage effects are neglected as compared to steel strains.

$$w_m = \epsilon_m \times s \qquad (4\ 66)$$

where w_m = mean crack width

ϵ_m = mean steel strain

s = spacing between cracks

Beam tests and tensile tests of bars embedded in concrete have shown a dependency of the calculated steel stress (f_s) and the mean strain (ϵ_m) as the steel percentage ratio (ρ) varies [4]. The strain corresponding to a given stress decreases as the steel percentage decreases. This relation can be expressed by

$$\epsilon_m = \frac{f_s}{E_s}\left(1 - \frac{k'}{\rho f_s}\right) \qquad (4.67)$$

where k' = coefficient to calculate correction in steel strain

4.5.2 Steel Stress-Strain Relationship. To determine the true behavior of the reinforcement in a reinforced concrete section, the effect of cracking in the member on the steel stresses and strains should be considered. Therefore, a determination of the mean steel strain needs to be evaluated. The main variable to be considered in the stress-strain relationship is the reinforcement ratio (ρ). This means a family of curves is suggested to represent the steel stress-strain relationship. This approach has proven to be very effective in dealing with the nonlinear analysis of reinforced concrete structures [4,7,8]. Except for the correction made in the steel strain, the stress-strain relationship for the steel is considered elasto-plastic.

Equation (4.67) can be written in the following form

$$\epsilon_m = \epsilon_s - \frac{k'}{\rho E_s} \tag{4.68}$$

The value for k' selected in this study was 57, as previously used by Alani [4]. The second term of the right-hand side of the equation represents a reduction in the steel strain to arrive at the mean steel strain. The relationship to account for the cracking of the concrete can be defined in terms of the mean steel strain throughout the section. The mean steel strain is given by expression (4.68). However, if ρ is less than 0.5%, the correction in the mean steel strain is given by

$$\epsilon_m = \epsilon_s - \left[\frac{57}{\rho E_s} + \left(\frac{57}{\rho E_s} - \frac{57}{(0.005)E_s}\right) \times \frac{\rho}{0.005}\right] \tag{4.69}$$

This expression from Alani [4] allows for a smaller reduction than would be given by Eq. (4.68). From the previous expressions for the mean steel strain, the stress is found by simply multiplying the strain by the modulus of the steel. If the stress is greater than the yield stress, the stress is taken as the yield stress. Arbitrarily, the steel is considered to fracture if a fiber strain reaches 1%.

Using the previous equations, a family of curves with different steel percentages for a yield stress of 40 ksi was calculated and is shown in Fig. 4.11. The method of finding mean steel strain to handle cracking is easily performed by computer methods which generate the required curve for each problem considered.

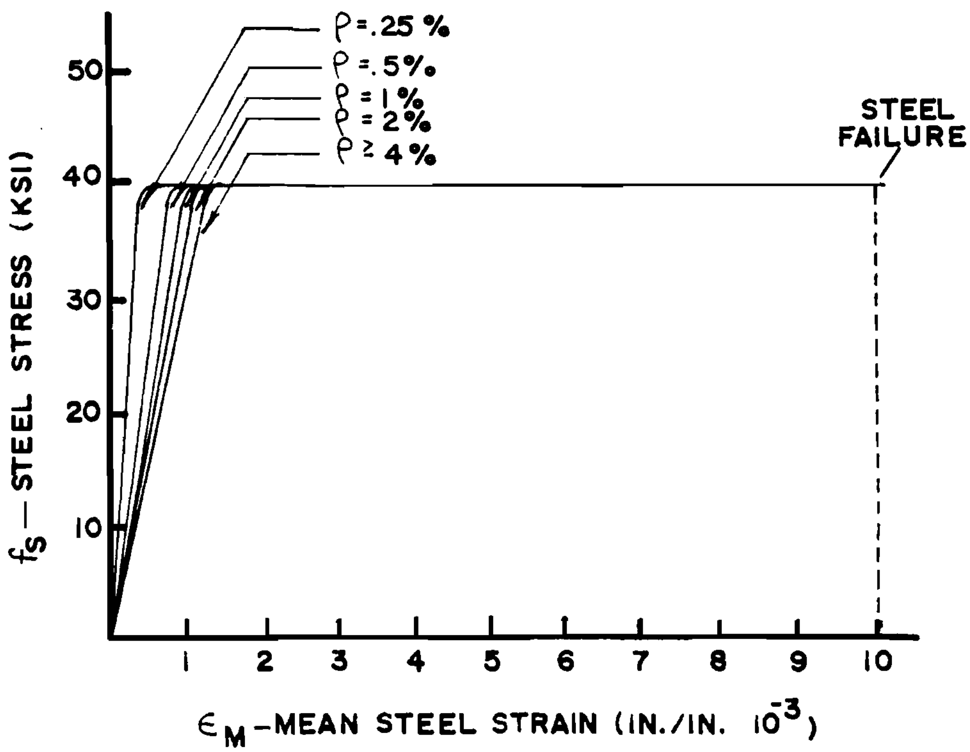


Fig. 4.11 Stress-strain characteristics of steel (f_y = 40 ksi) taking into account the correction in steel strain [4]

4.5.3 <u>Concrete Stress-Strain Relationship</u>. There have been many theoretical curves proposed by different authors describing the stress-strain relationship of concrete in compression. One of the most widely used was suggested by Hognestad and is shown in Fig. 4.12 [12]. The curve was based on test results from 120 columns. The ACI Code [1] recommends Eq. (4.70) for the modulus of elasticity of concrete in psi with w_c varying between 90 and 155 lbs per cu. ft.

$$E_c = w_c^{1.5}\ 33\sqrt{f'_c} \qquad (4.70)$$

The Hognestad curve has been shown to give good results as long as the concrete is not confined by lateral ties or a moment gradient [4]. It was adopted for use in this study when no confinement effects were present. In Fig. 4.12, Hognestad originally took the value of k as 0.85. However, from other studies [37] it was determined that a k value of 0.85 was appropriate only for vertically cast columns, and that k should be 0.95 for horizontally cast columns.

Figure 4.13 from Kent and Park illustrates the effect confined concrete will have on the value of the ultimate strain (ϵ_u) of the concrete [13]. Ford [11] in his study proposed a different stress-strain relationship of confined concrete as is shown in Fig. 4.14. His stress-strain curve for confined concrete was adopted in this study. The ultimate strain is given by

$$\epsilon_u = 0.003 + 0.02\,\frac{b}{z} + \left(\frac{\rho''_h\, f_y}{14.5}\right)^2 \quad \text{in./in.} \qquad (4.71)$$

which is an adaptation of the Mattock and Corley expression [10]. The terms reflect a base strain, an effect of a moment gradient, and the effect of lateral confinement of ties. For concrete to be considered confined, enough lateral ties must be provided to meet the equivalent of ACI Building Code 318-83, Appendix A Sec. A.4.4 for compression members [47]. In Fig. 4.14, k is taken as 0.95 for all cases in which the concrete is considered confined.

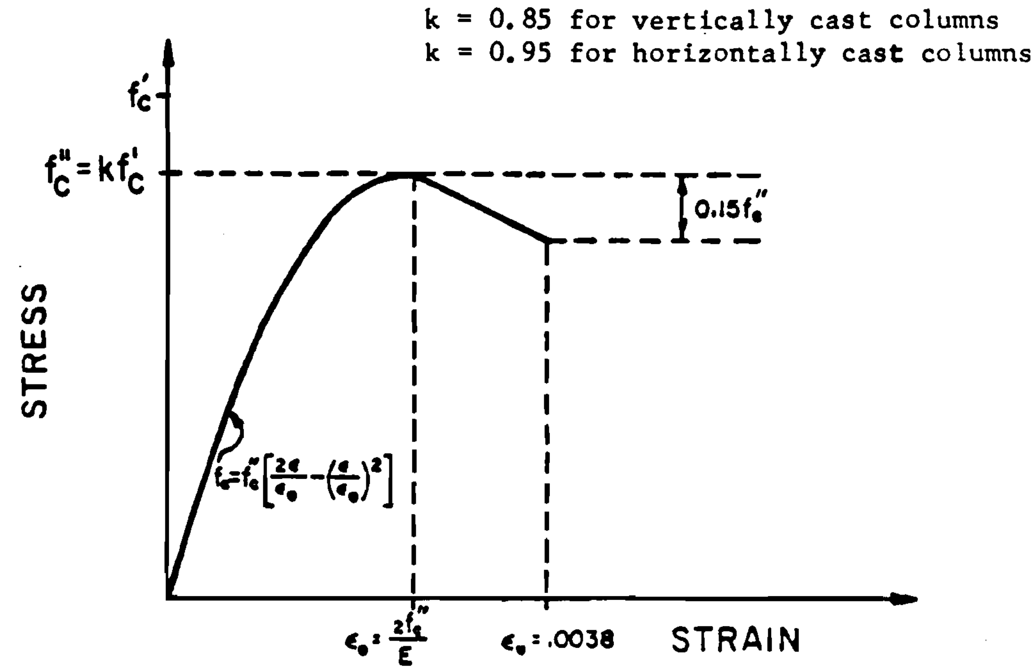


Fig. 4.12 Hognestad stress-strain curve [12]

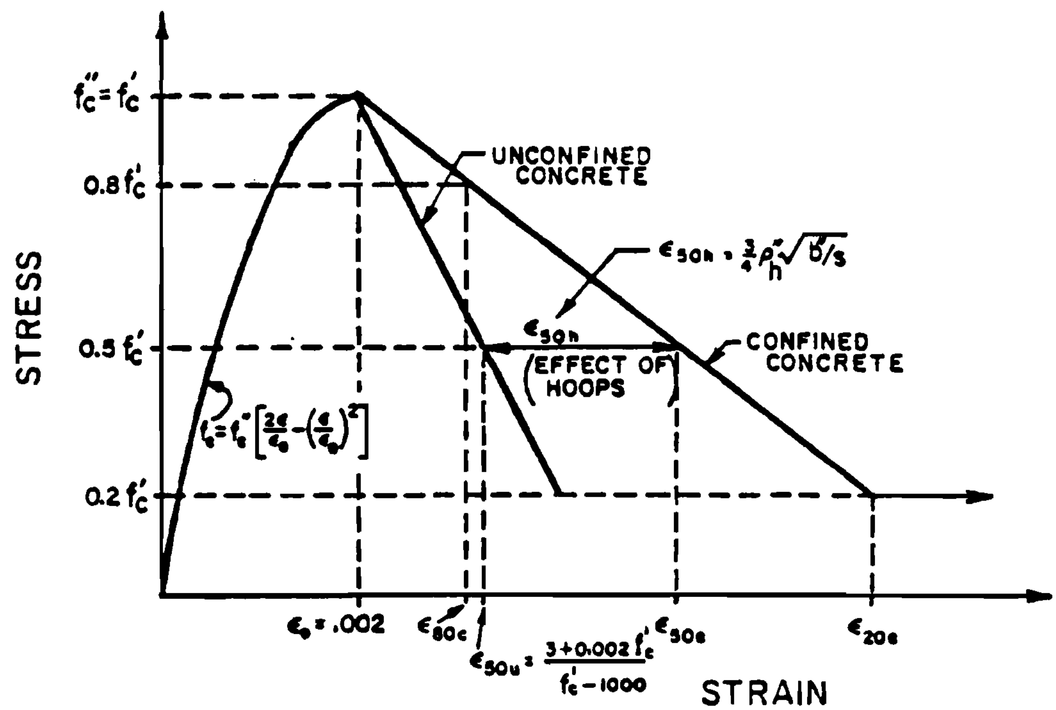


Fig. 4.13 Stress-strain curve for concrete confined by rectangular hoops from Kent and Park [13]

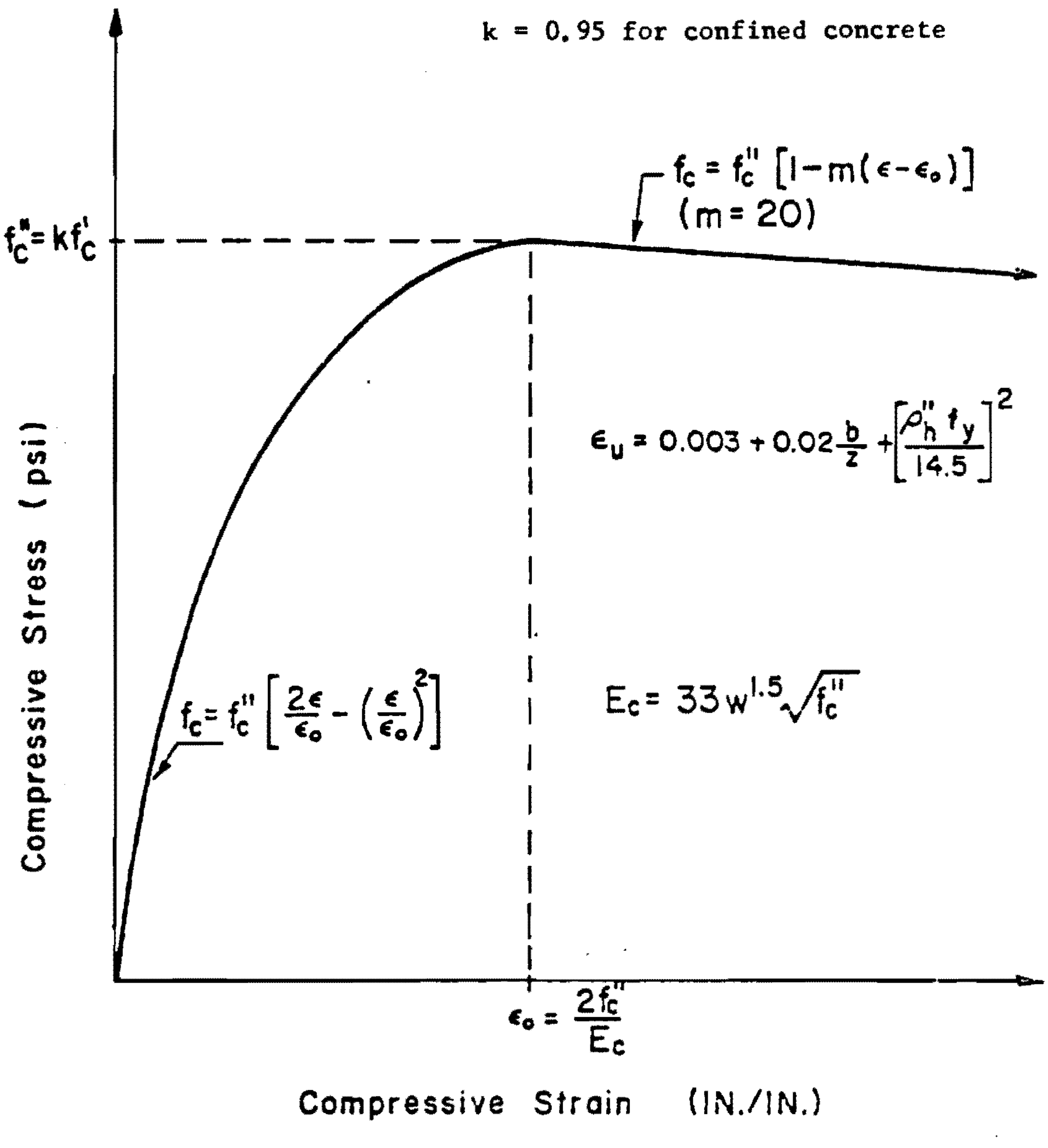


Fig. 4.14 Compressive stress-strain curve of confined concrete from Ford [11]

Concrete tensile stress capacity is not directly included in the model. The residual tensile capacity between cracks can have a pronounced effect on stiffness and is indirectly considered by the modification of the steel stress-strain relationship outlined in Sec. 4.5.2.

In order to account for the effect of load duration or sustained load effects on piers, a modification of the short-time stress-strain relationship for concrete must be made. In this study the relatively simple, yet accurate, procedure suggested by Chovichien et al. [17] is used. The modification consists of expressing both the maximum compressive stress for concrete and the concrete strain corresponding to maximum stress in terms of logarithmic functions of time. The expressions used are simplified variations of those recommended by Chovichien et al. [17]. The values used are

$$f''_c \ (t) = [0.85] \ f'_c \tag{4.72}$$

$$\varepsilon_o \ (t) = 0.002 + 0.00085 \ \ell_n(t + 1) \tag{4.73}$$

For $t > 2$ years

$$\varepsilon_o \ (t) = 0.0076 + 0.000375 \ \ell_n(t - 730) \tag{4.74}$$

In all cases, t = time under sustained loading in days. This modification was shown by Chovichien et al. to give very good agreement with test results and was independently verified against results of several sustained load tests after being incorporated into the fiber model [16]. In addition, the expression for ε_u becomes

$$\varepsilon_u \ (t) = 1.9 \ \varepsilon_o \ (t) \tag{4.75}$$

which reverts to a value of 0.0038 when no time effects are considered.

4.6 Summary of the Fiber Model

The principal assumption of the method is that small changes in displacement can be linearly related to small changes in force [3]. In order for this to be true, a segment or member must be analyzed in the undeformed position, thus individual member stability is not considered, but only overall pier stability.

Each member is divided into a series of cross sections and each cross section into fibers. The first step in assembling the stiffness matrix is calculation of flexibility coefficients from the fiber dimensions and stress state of each cross section. The integration over the cross section is accomplished by summing over each fiber for all fibers of a cross section. Once these flexibility coefficients have been determined the flexibility matrix is formed by summing the appropriate flexibility term for each cross section over all the cross sections of the segment or member. The member flexibility matrix is then inverted to form the member stiffness matrix. The member stiffness matrix is then rotated into global coordinates and added to the total pier stiffness matrix.

After the new joint displacements and forces have been computed by the stiffness method, the inverse of the original member force-strain assumption is used to calculate the incremental strains at each cross section. From these strains the strain of each fiber in the cross section can be computed. This now makes it possible to recalculate the stiffness coefficients and the procedure begins again.

In summary, the assumption made in the fiber model is that small changes in member force can be linearly related to small changes in member strains, and the geometry of the member in the deformed position is a straight line. A method of generating cross sections and fibers is used to assemble the stiffness matrix and monitor the strains. The stiffness matrix is reassembled for every load increment and the cycle is continued until all specified increments of load have been applied to the pier, or there is a material, stability, or plastic hinging failure of the pier. P-delta, geometrical, and material nonlinearities are included in the formulation.

CHAPTER 5

ANALYSIS PROGRAMS

5.1 Introduction

The previous chapter outlined the fiber model formulation which is the key element in the method selected for the generalized second order analysis of bridge piers. Both section behavior (P-M-ϕ) and member behavior can be obtained from the general fiber model formulation. Utilizing that approach, three FORTRAN IV language computer programs were developed. The first program, BIMPHI, is essentially for verification purposes. When used as a subprogram it provides the basic biaxial load-moment-curvature relationship of a section. The second program, PIER, analyzes a single column bridge pier subjected to biaxial static loads. The third program, FPIER, analyzes a multiple level multiple bay bridge bent subjected to biaxial static loads. This chapter introduces the computer programs. Detailed input guides are continued in Appendices A through C.

5.2 Program BIMPHI

BIMPHI was originally written in FORTRAN IV for The University of Texas at Austin CDC Cyber 170/750 computer. The program is easily adaptable to any other system since it contains little computer hardware dependency. It is in conformance with the Texas State Department of Highways and Public Transportation requirement that programs conform to FORTRAN 77. A listing of the FORTRAN 77 program and explanation of its use can be found in Appendix A.

5.2.1 Program BIMPHI Details. This program as a stand alone program is of little direct use in pier design. However, it is an important subprogram in the design-oriented programs PIER and FPIER.

Thus, a full explanation is given herein. The program generates the theoretical biaxial P-M-ϕ' relationship of a reinforced concrete section and considers concrete cracking. BIMPHI can obtain the relationship in several ways. The usual way is to increment both axes curvatures and find the corresponding moments. If one increments both axes moments then it finds the curvatures. It is also possible to increment moment for one axis and curvature for the other axis. In this case it finds the other curvature and the other moment. In performing any one of these procedures, BIMPHI assumes the ratio of incremental moments or the ratio of incremental curvatures or the ratio of incremental moment to incremental curvature to be constant. This is an important limit which precludes direct use of BIMPHI for analysis of a biaxially loaded column where second order deflection effects will be large. The actual end moments being applied may be constant, but because the P-delta moments about each axis will likely be different, the ratio of the incremental moments will be changing. BIMPHI as a stand alone program will yield incorrect results when second order effects become important in later stages of loading. This limitation is overcome by use of PIER and FPIER. Thus, BIMPHI is restricted to cross section analysis.

Normally, experimental tests utilize a controlled load procedure in which known forces and moments are incremented and the curvatures are measured. This experimental technique is generally restricted to the ascending portion of a M-ϕ' curve as shown in Fig. 5.1(a) since failure occurs suddenly when the peak is reached. Controlling curvatures and measuring moments is a complicated experimental technique, but allows definition of the entire M-ϕ' curve for both the ascending and the descending branches as shown in Fig. 5.1(b). The program can simulate either case. The other variable which can be incremented relates to the axial load. Normally, in checking test behavior, the axial load is specified

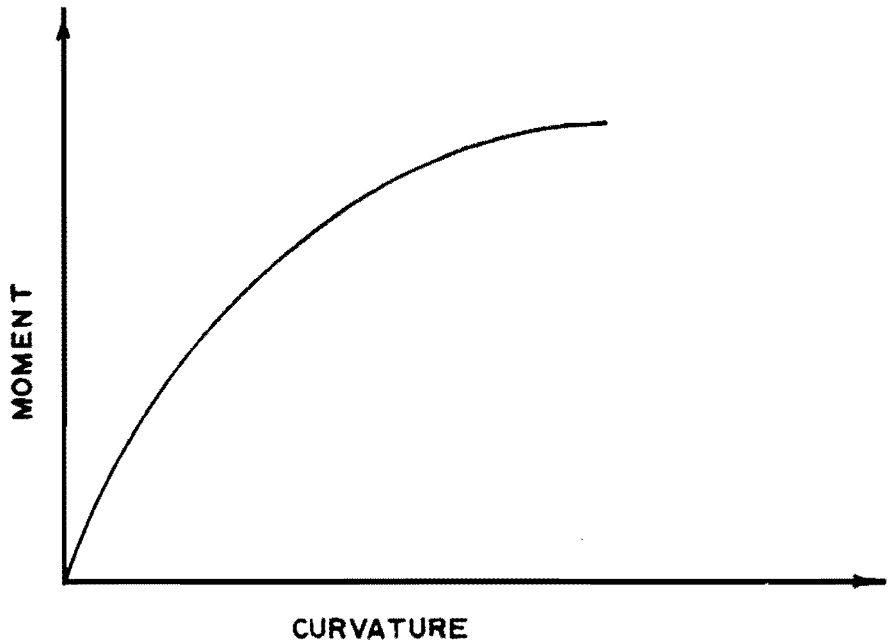


Fig. 5.1a $M-\varphi'$ curve obtained for a constant axial load by controlling forces

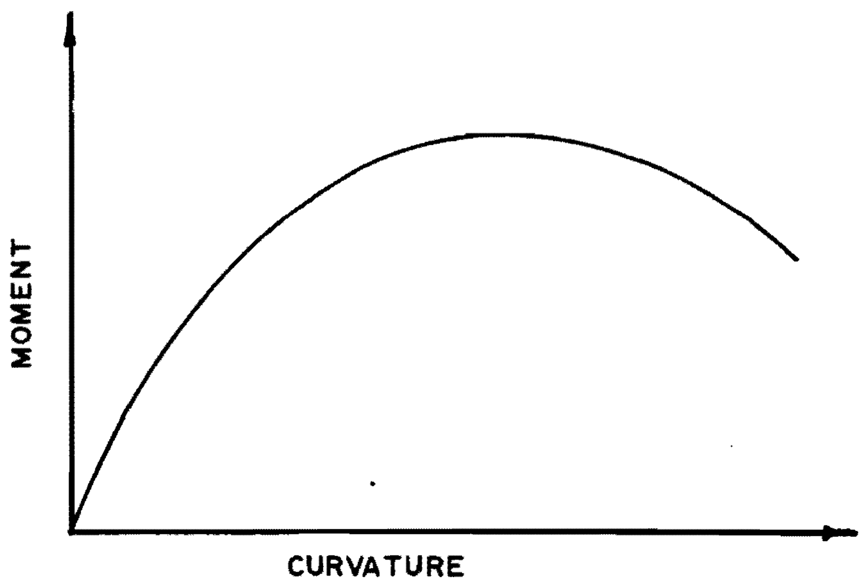


Fig. 5.1b $M-\varphi'$ curve obtained for a constant axial load by controlling curvatures

and held constant. However, for more design-related checks, the program will allow for incrementing the axial load, or specifying an increment in axial strain.

The type of analysis performed is given by a release code. There are three degrees of freedom for a section: (1) axial strain or force; (2) curvature or moment about the x-axis; and (3) curvature or moment about the y-axis. An "0" specified for a degree of freedom implies an incremental force (or moment) may be given. A "1" specified for a degree of freedom implies an incremental deformation may be given. An increment of force or deformation may be specified but not both for the same degree of freedom. The most common analyses performed will be one of the two following cases.

	Degree of Freedom		
Case	Axial	x-curvature	y-curvature
1	0	1	1
2	0	0	0

The first implies that an increment of axial load and incremental curvatures about the x- and y-axes will be specified. The second example implies that an incremental axial load, incremental moment about the x-axis, and incremental moment about the y-axis will be specified. The normal procedure in the two cases would be to specify an initial axial load and then to specify the incremental axial load as zero in order to obtain the M-ϕ' relationship for a constant axial load. If the axial load is constant, the axial degree of freedom could have been specified as 1, but BIMPHI would expect an increment of axial strain which the user would specify as 0. However, it was found that in actuality due to stiffness changes the axial load calculated at each increment tended to decrease even though the incremental axial strain was 0. Therefore, it is suggested that the axial strain release code equal "0" (incremental force) for the constant load case. If the user desires to specify an incremental axial strain which is not zero, a "1" is specified for the axial degree of freedom and the appropriate value of the incremental strain is specified. If by accident both an incremental deformation and force are specified for a single degree of freedom, the force will be used if a "0" was given in the release code, the deformation will be used if a "1" was given in the release code.

The positive axes of the section are shown in Fig. 5.2(a). Positive incremental curvatures and moments are defined by the right-hand vector rule shown in Fig. 5.2(b). Axial deformation and load is positive if it causes tension, negative if it causes compression.

A general flowchart of BIMPHI is shown in Fig. 5.3. The procedure is repeated until the section undergoes an assumed material failure. A compression failure is assumed if any concrete fiber strain exceeds the specified concrete ultimate strain. A tension failure is assumed if any steel fiber strain exceeds 1%.

The input required for a given problem is minimal. The program includes several subroutines which generate the fiber meshes for the steel and concrete, and compute the needed fiber properties such as x and y centroid and area. The program accounts for the concrete displaced by the reinforcing steel. Each fiber generator subroutine includes optimization procedures which generate as many fibers as possible up to 200 fibers each of concrete and reinforcing steel. The reinforcing steel of a section is modeled as a ring of steel as shown in Fig. 5.4. This procedure requires little geometry input from the user. If desired, the user may input individual reinforcing bar location and area.

At present, the program will handle any of the sections shown in Fig. 5.5. The circular section is a special case of the oval section. The needed input geometry for each is described in Appendix A.

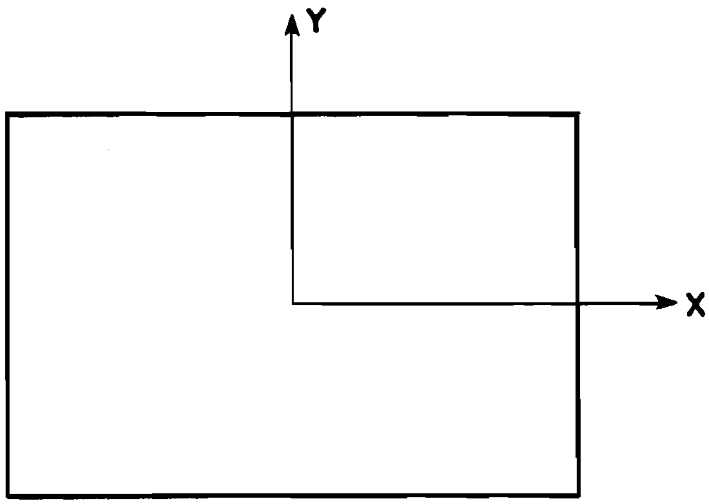


Fig. 5.2a Positive axes of section

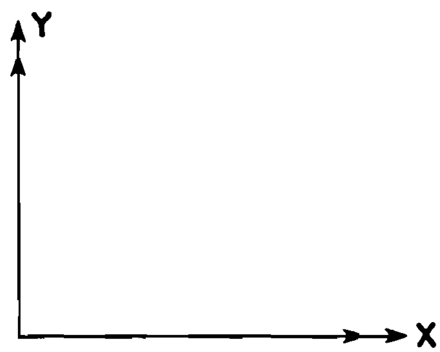


Fig. 5.2b Positive curvatures and moments given by right-hand rule

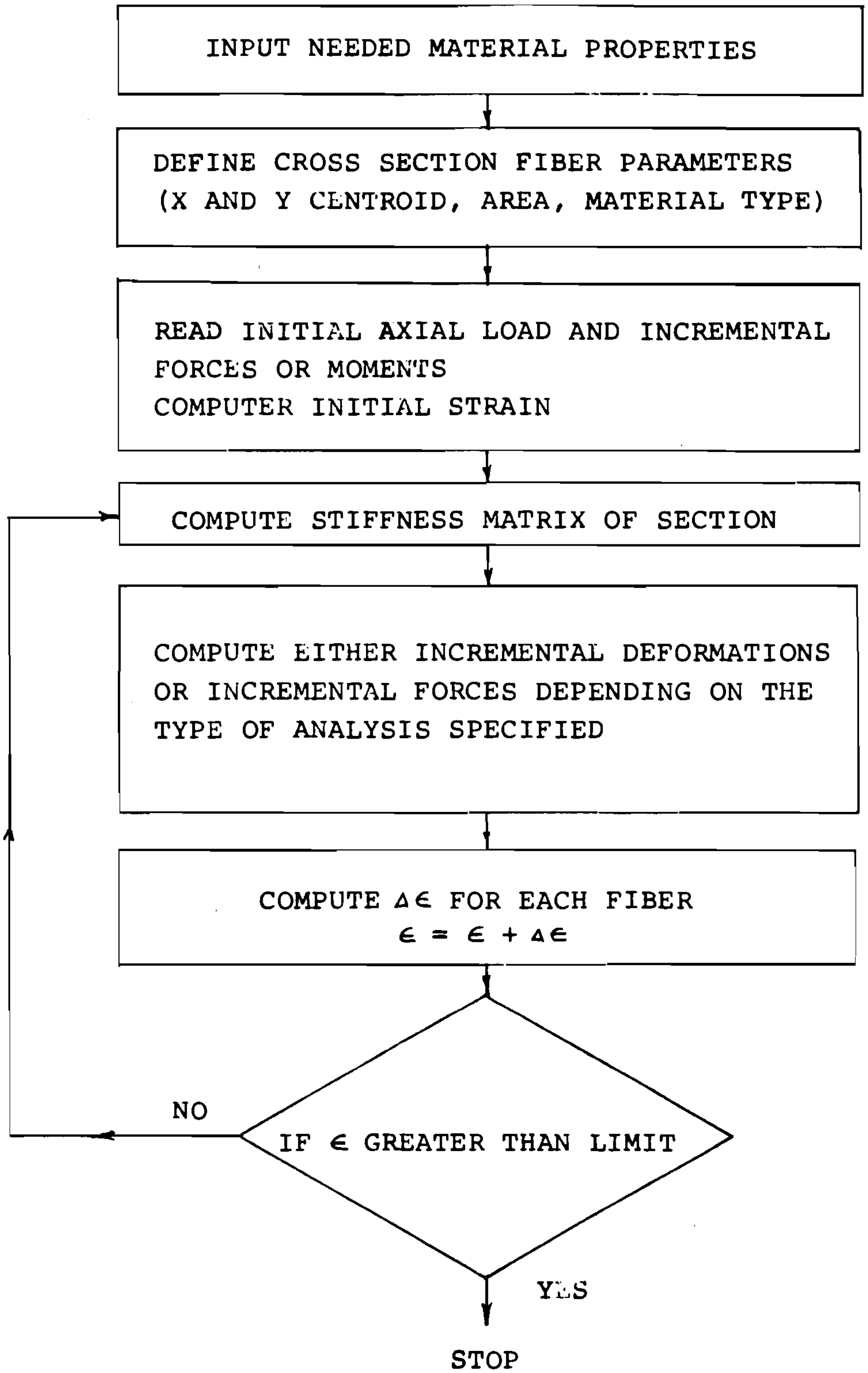
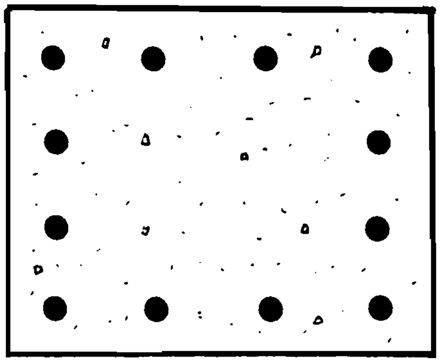
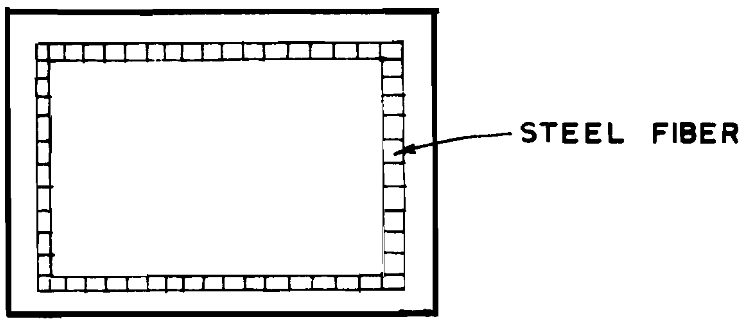


Fig. 5.3 Flowchart of program BIMPHI



ACTUAL CONCRETE SECTION



SECTION SHOWING REINFORCING STEEL
MODELLED AS MANY STEEL FIBERS
IN A RING

Fig. 5.4 Modeling of reinforcing bars

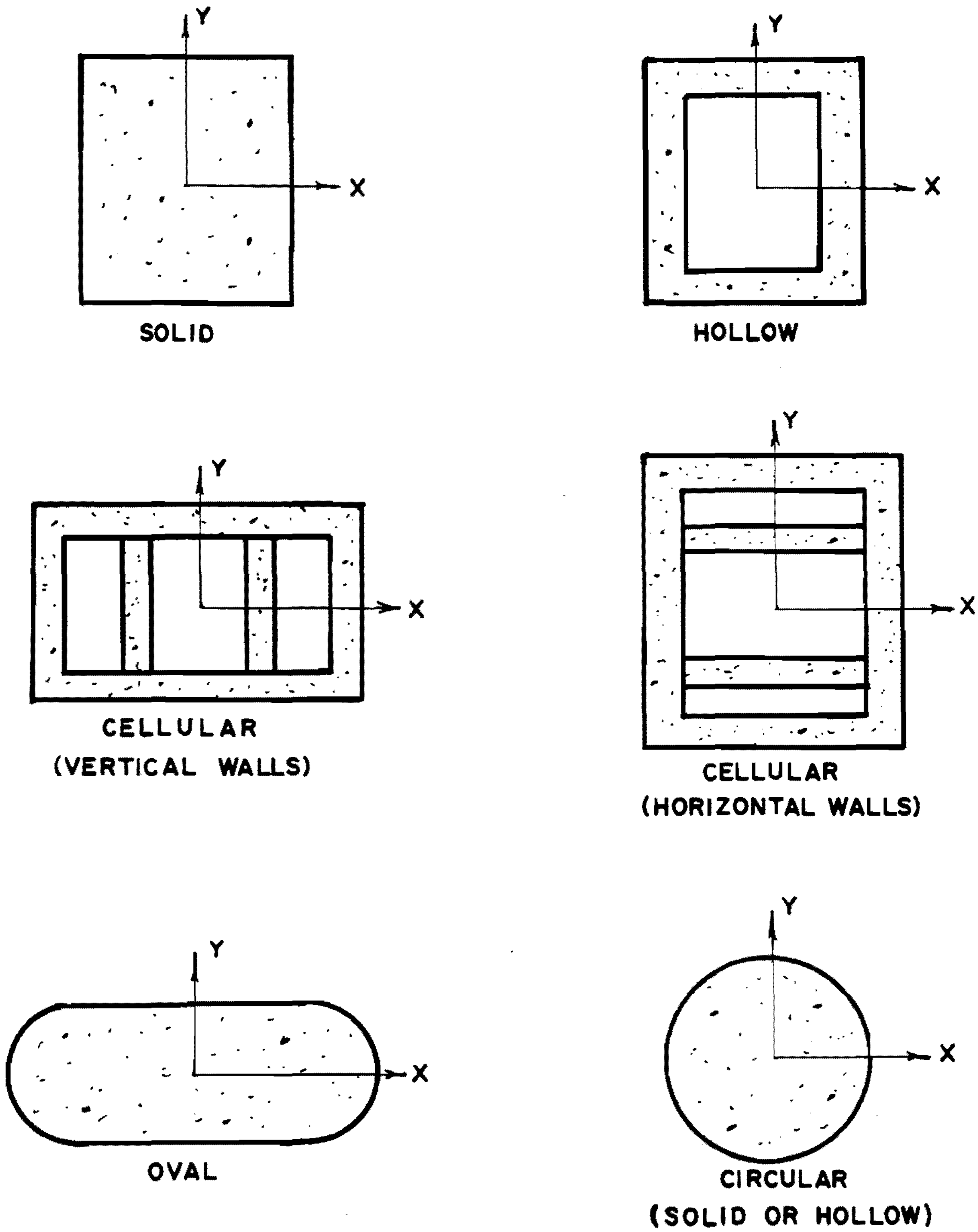


Fig. 5.5 Sections handled by BIMPHI, PIER, and FPIER

The output from the computer analysis for each step or increment includes the axial load, moment about the x-axis, moment about the y-axis, curvature about the x-axis, and curvature about the y-axis. For each increment there will be two lines of values printed. The first values are calculated from the assumed strains. The second set of values is the result of equilibrium checks. Ideally the two sets should be in complete agreement. However, a convergence tolerance permits small differences. If the values of axial load calculated from the two methods vary by more than 5%, a correction procedure is performed on that increment one time, and the corrected values printed. This correction is valuable if the incremental curvature or moment selected is too large. When BIMPHI performs this correction it is signaled on the output by the increment or step number being printed four times. This is a warning that the increment of force or deformation selected is too large for accurate results. Also, if only a few increments are printed, the increment chosen was probably too large and the results are probably not valid. The procedure to obtain accurate results requires some judgement. As guidance, if there are no more than 50 increments printed before an assumed failure, or if the consequence correction procedure has been required for many increments, the increment of force or deformation selected should be reduced. For example, if only 25 increments were performed before an assumed failure, the analysis should be repeated with the increment size cut in about one-half. The output should include at least 50 increments for accurate results. The user should not reduce the increment size too rapidly to avoid excessive computer time.

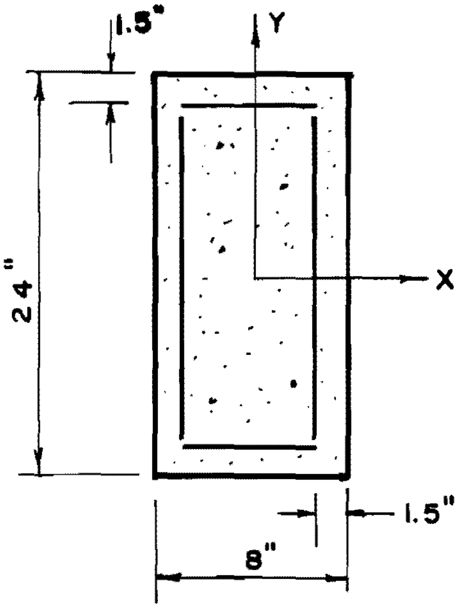
Another error check for the constant load case is the examination of the printed axial loads at each increment. The axial loads at each step should be within a few percent of the constant applied axial load. If the axial loads are changing, check to make sure the release code is "0" for the axial degree of freedom. As mentioned previously, if the axial degree of freedom is specified as "1" with the incremental axial strain equal to 0, due to the nature of the analysis the axial load at each increment tends to decrease. Therefore, it is best to use "0" in the release code for the axial degree of freedom and specify the incremental axial load as 0.

5.2.2 Examples Using Program BIMPHI. Several example sections were input for program execution to demonstrate the program capability. The accuracy of the program when compared to experimental results is shown in Ref. 18. The analysis of the solid section shown in Fig. 5.6 was performed for a constant axial load equal to 0.6 of the pure axial load strength (P_o) with the following combinations of incremental curvatures:

Case	$\Delta\phi'_x$	$\Delta\phi'_y$
1	5×10^{-6} rad./in.	0
2	0	5×10^{-6} rad./in.
3	5×10^{-6} rad./in.	2.5×10^{-6} rad./in.
4	2.5×10^{-6} rad./in	5×10^{-6} rad./in.
5	5×10^{-6} rad./in.	5×10^{-6} rad./in.

The P-M-ϕ' curve for each axis (if $\Delta\phi' \neq 0$) of each load case is shown in Figs. 5.7 through 5.11. Table 5.1 gives the maximum moments and curvatures attained for each case.

The analysis for each case is as expected. Accuracy of these values was verified experimentally in Ref. 18. Case 1 which is uniaxial bending about the strong axis shows much higher load and smaller curvature at ultimate than Case 2 which is uniaxial bending about the weak axis. This is the correct prediction. Case 3 compared to Case 1 shows that the ultimate moment and curvature developed on the strong axis is reduced with the presence of a lesser amount of curvature applied about the weak axis.



Side steel = 1.44 in.^{2}

End steel = 0.48 in.^{2}

Total steel =

2(1.44 + 0.48) = 3.84 in.^{2}

f'_c = 4000 psi (unconfined concrete)

f_y = 40000 psi

P = $0.6P_o$ = 476k

Fig. 5.6 Solid section analyzed

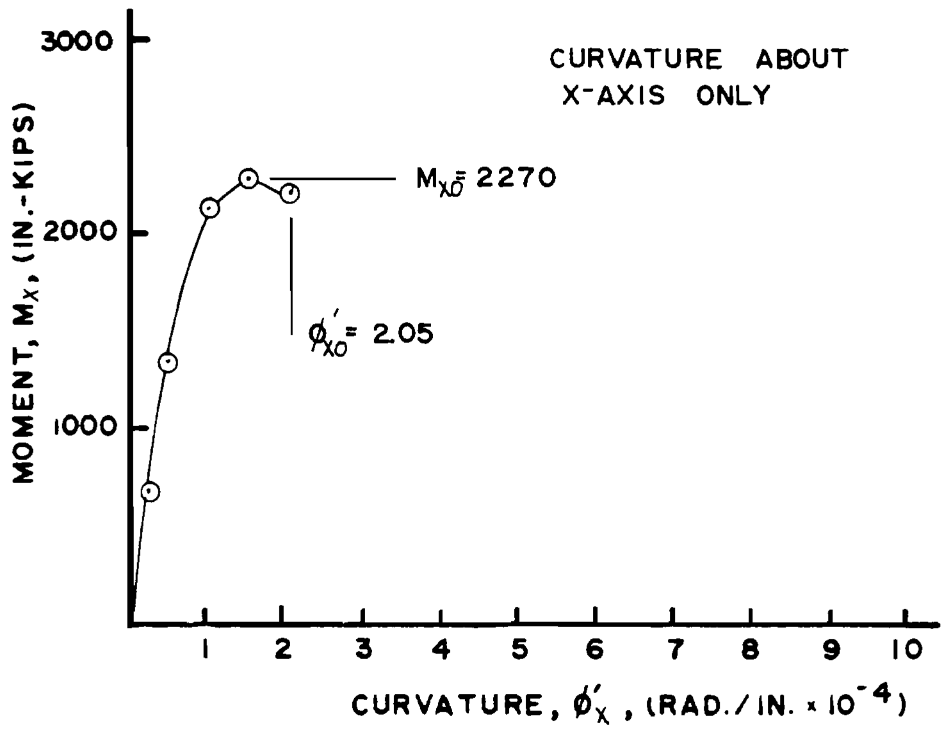


Fig. 5.7 P-M-ϕ' curve for Case 1

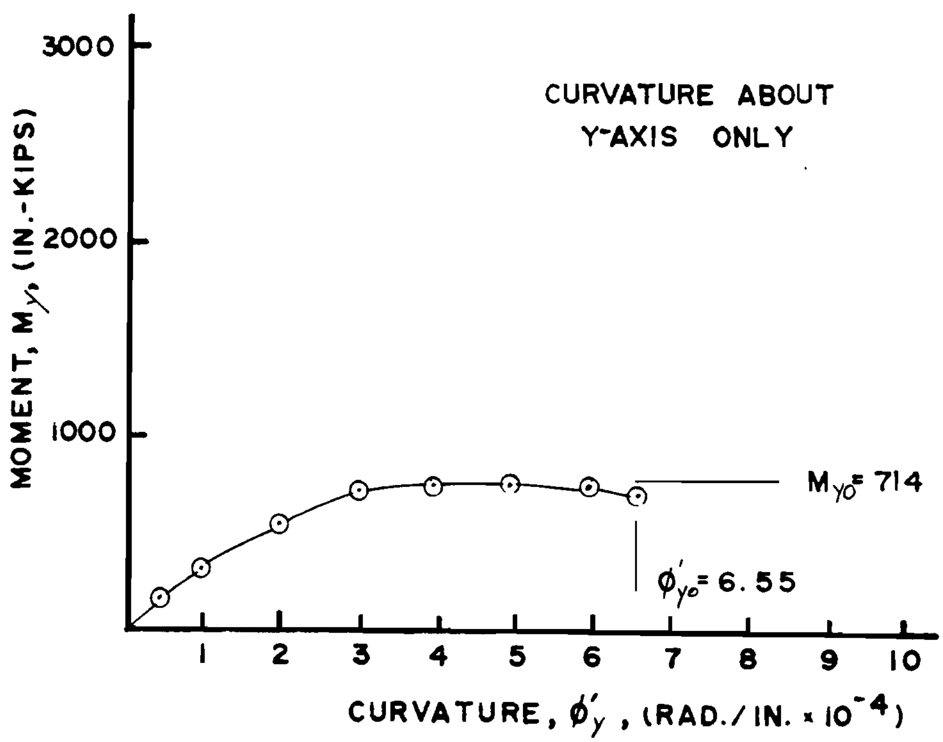


Fig. 5.8 P-M-ϕ' curve for Case 2

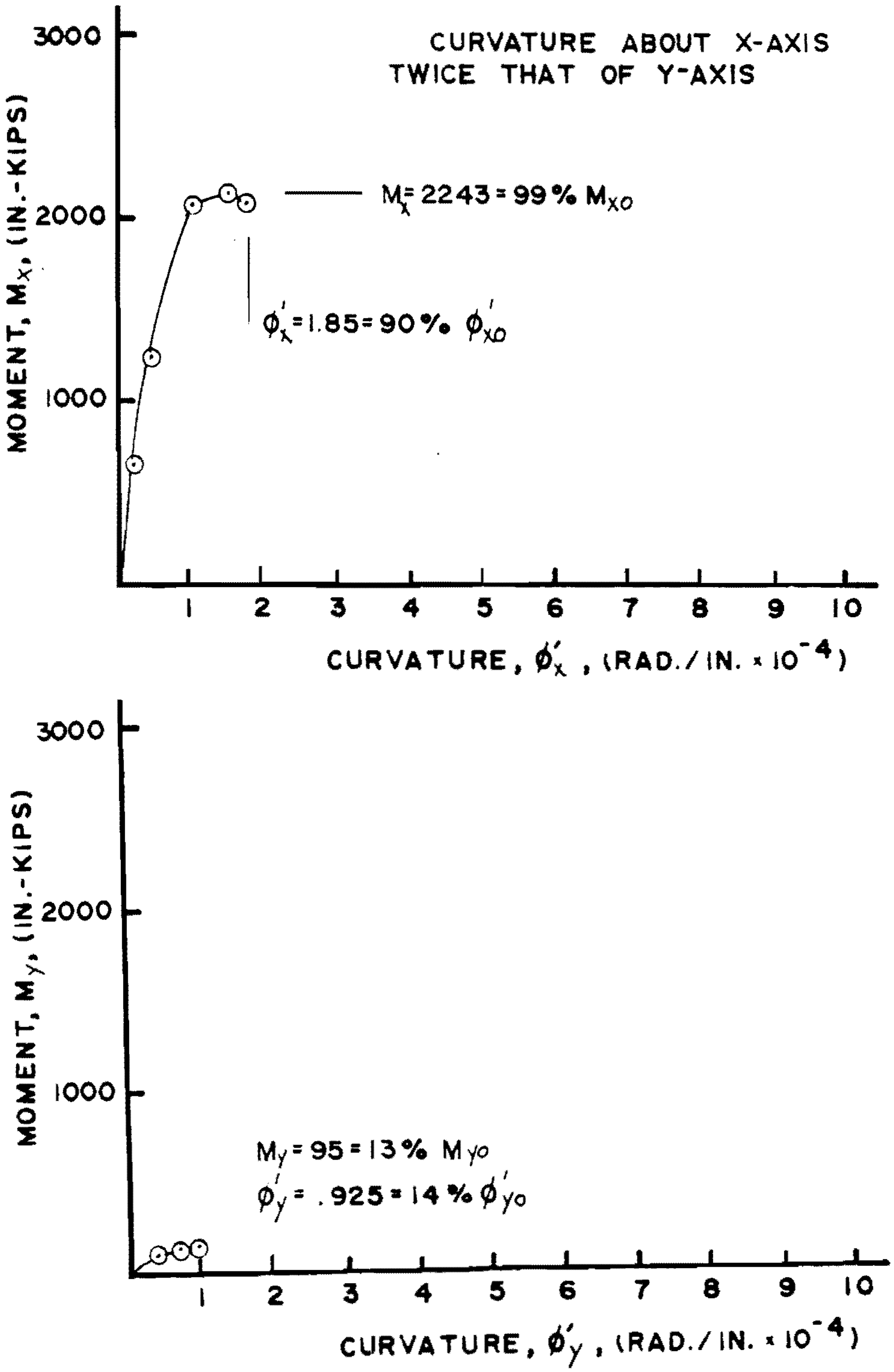
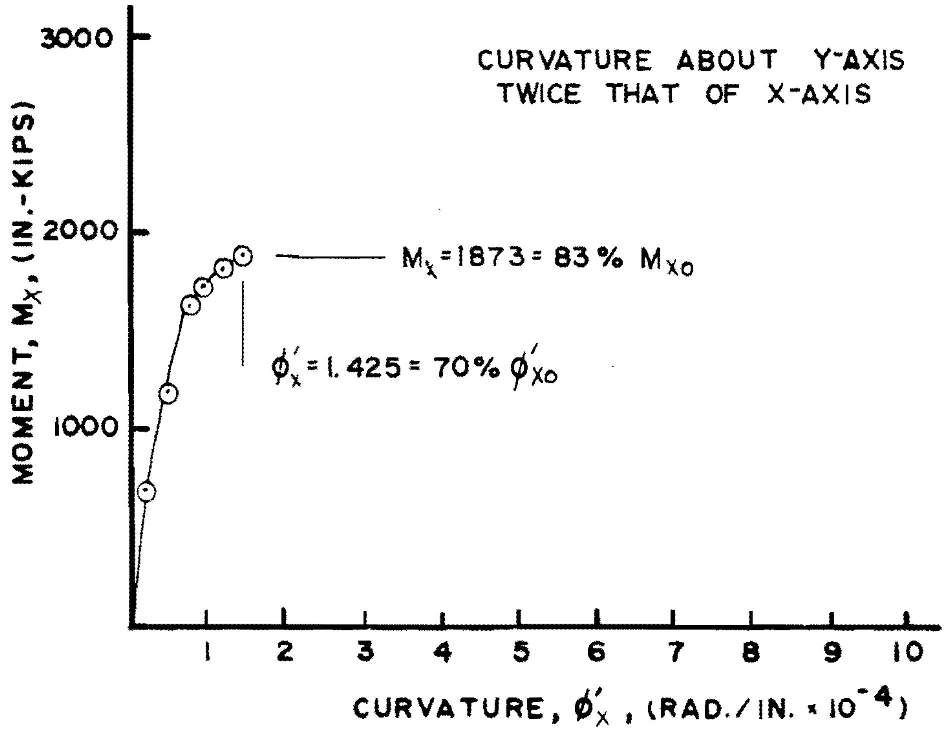


Fig. 5.9 P-M-ϕ' curves for Case 3



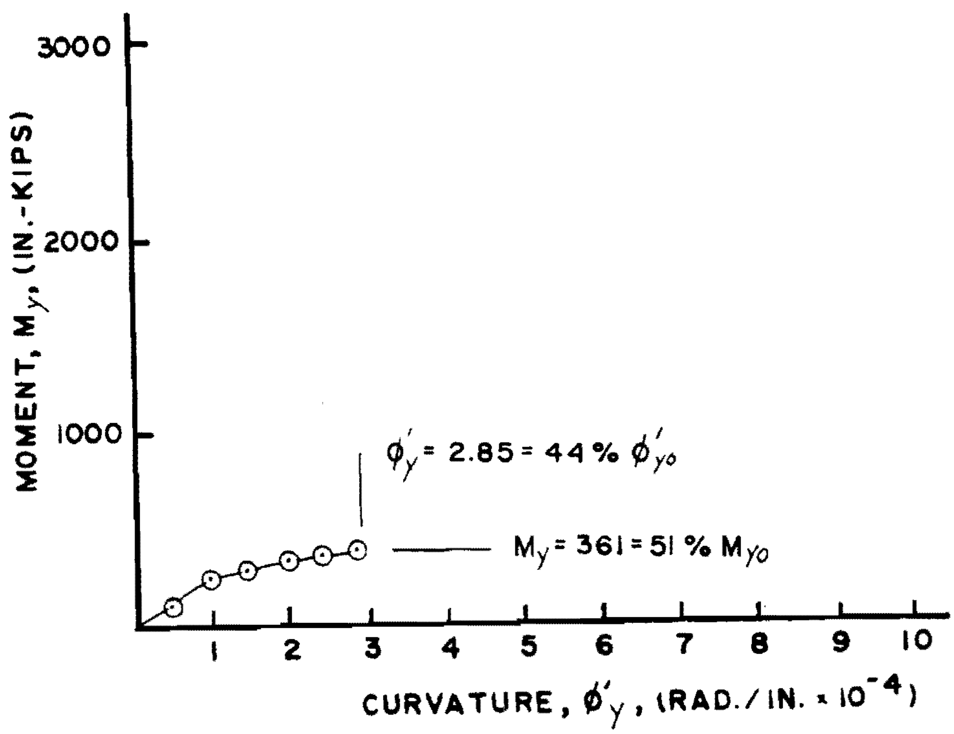


Fig. 5.10 P-M-ϕ' curves for Case 4

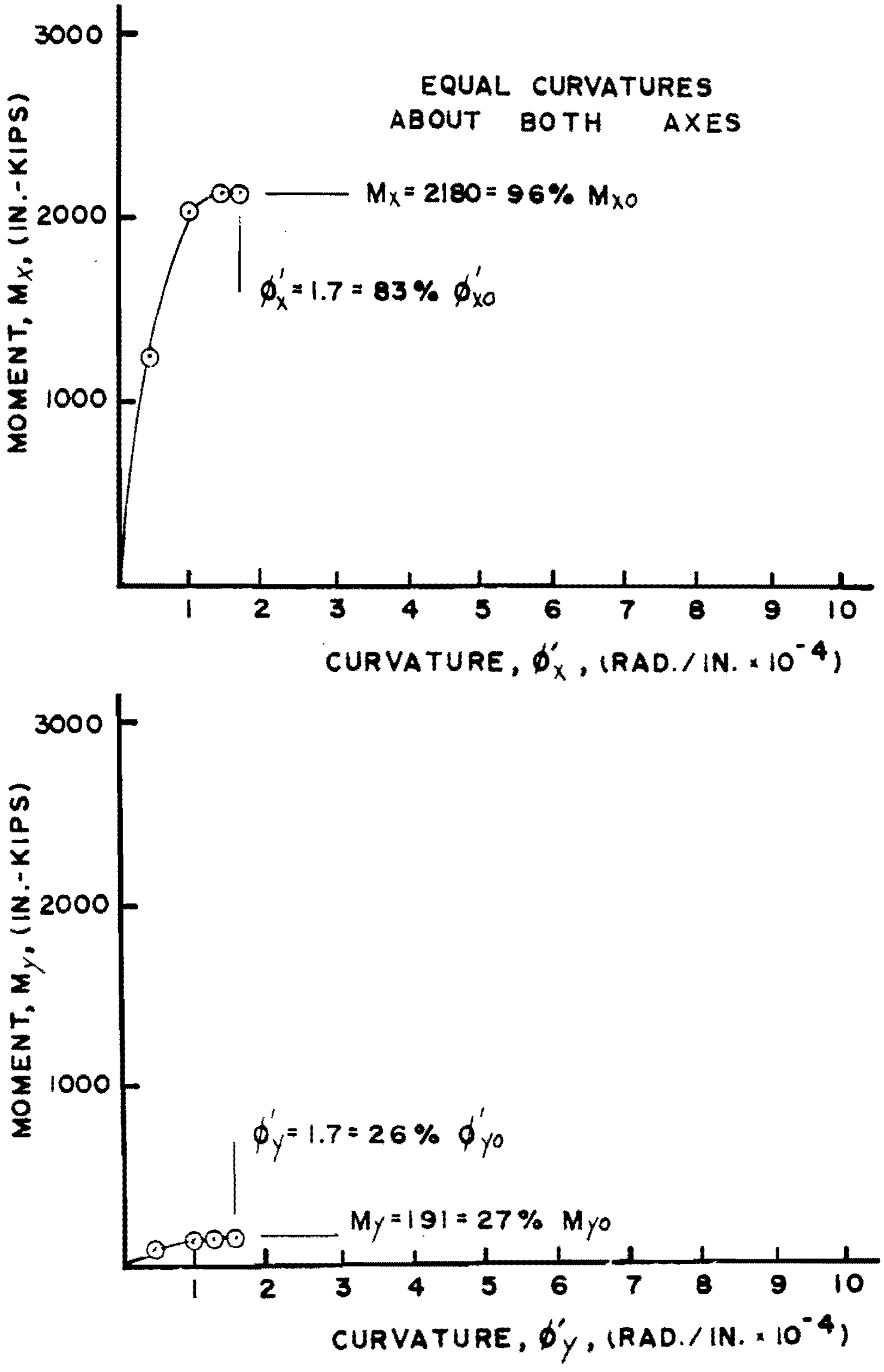


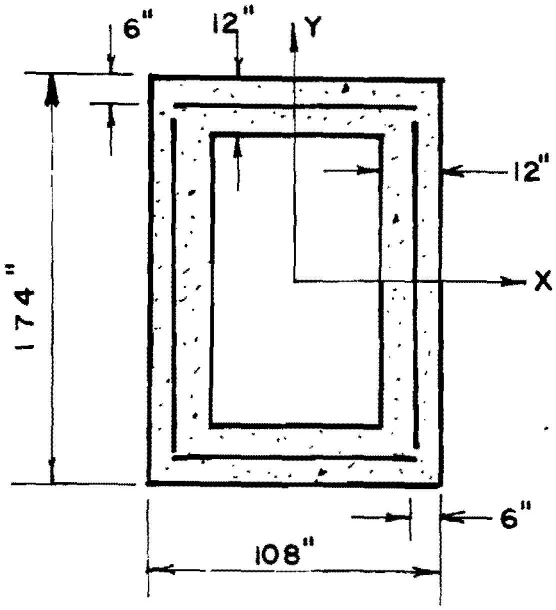
Fig. 5.11 P-M-ϕ' curves for Case 5

TABLE 5.1 COMPARISON OF RESULTS

Case	M_{max} (in.-kips)		ϕ'_{max} (rad./in. × 10^4)	
	M_x	M_y	ϕ'_x	ϕ'_y
1	2270	--	2.05	--
2	--	714	--	6.55
3	2243	95	1.85	0.925
4	1873	361	1.425	2.85
5	2180	191	1.7	1.7

This should occur since the section is loaded biaxially. The capacity of a section loaded biaxially is reduced compared to the same section loaded uniaxially. Case 4 compared to Case 2 also shows that the ultimate moment and curvature of the section loaded uniaxially about its weak axis is reduced by the presence of simultaneous bending about the strong axis. Case 5, equal simultaneous curvatures applied about both axes, also shows a reduction in moment capacity and ultimate curvature obtained for each direction when compared to the values found for the section loaded uniaxially about each axis. BIMPHI shows that the capacity of a section loaded biaxially is reduced compared to the same section loaded uniaxially. It also shows that the ultimate curvature obtained for a section loaded biaxially is less than if the section was loaded uniaxially.

The hollow section shown in Fig. 5.12 subject to a constant load and uniaxial bending about its strong axis was analyzed by BIMPHI. Note that this section has a wall unsupported length to thickness ratio of 150/12 = 12.5 and hence the applicability of plane section theory is somewhat questionable since X_u/t is greater than 7.5 (see Ref. 18). For this section both an incremental load procedure and an incremental curvature procedure were performed. Figure 5.13(a) shows the M-ϕ' relationship obtained from the incremental curvature procedure, and Fig. 5.13(b) shows the relationship obtained from the incremental moment procedure. As can be observed, the maximum moment and corresponding curvature obtained is the same except the incremental moment procedure does not yield the descending branch of the M-ϕ' relationship. The same result would occur in a physical test. If a column was tested in a controlled load procedure only the ascending branch of the M-ϕ' curve would be obtained, whereas if a controlled deformation procedure was utilized the entire curve, both ascending and descending branches, would be found.



Side steel = 19.1 in.^{2}

End steel = 11.8 in.^{2}

Total steel = 61.8 in.^{2}

f'_c = 4000 psi (unconfined concrete)

f_y = 40000 psi

P = $0.5P_o$ = 11660k

Fig 5.12 Hollow section analyzed

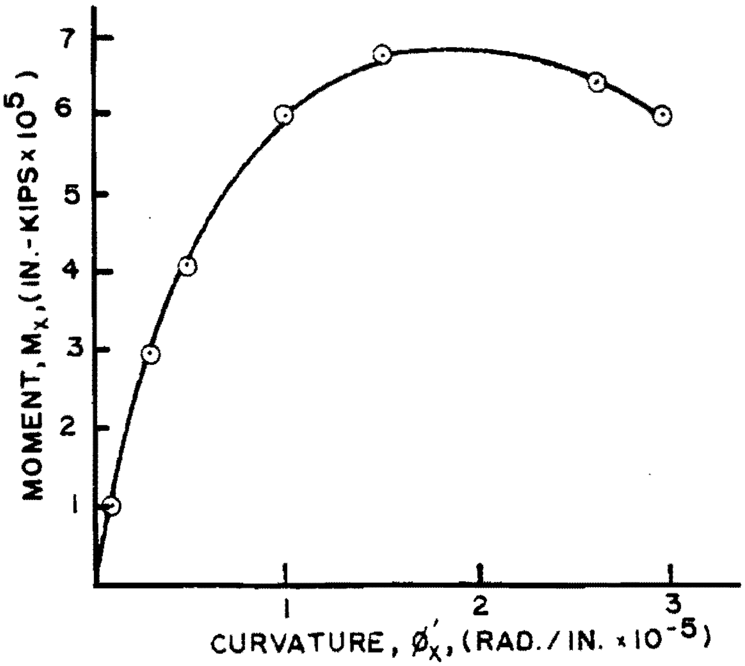


Fig. 5.13a P-M-ϕ' for hollow section using incremental curvature procedure

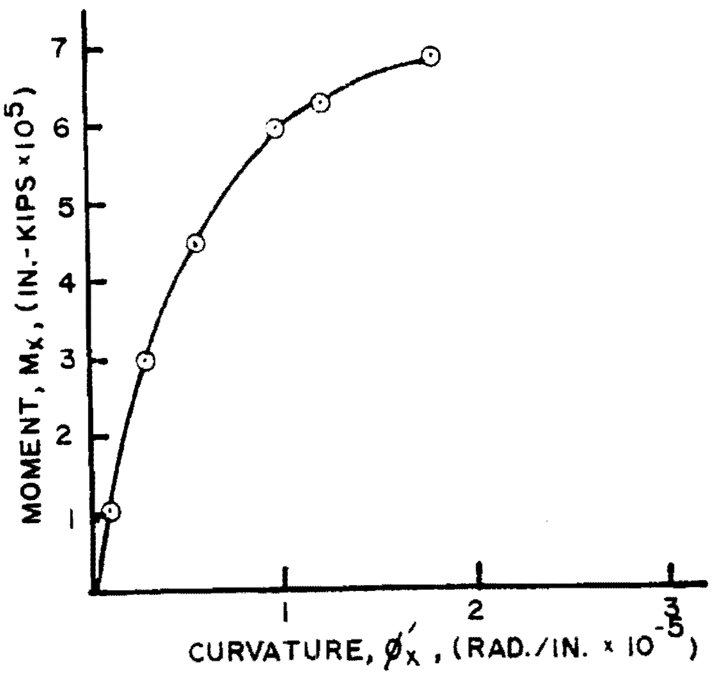


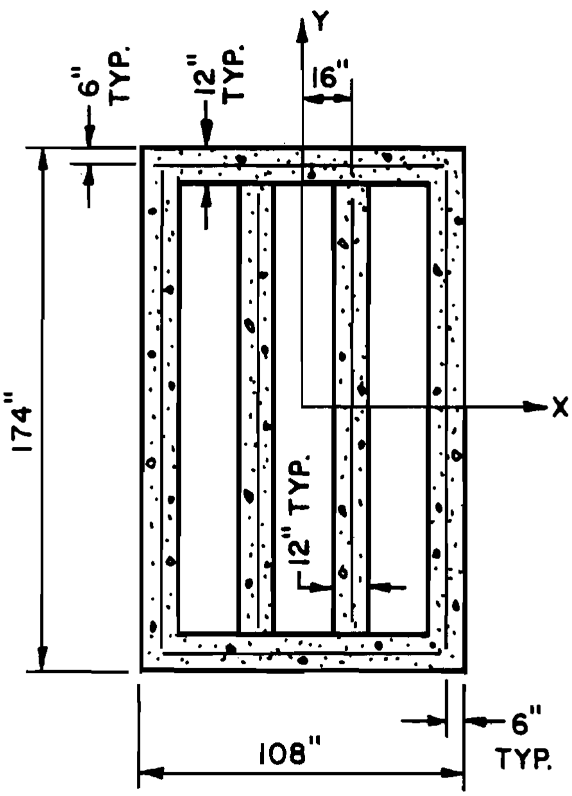
Fig. 5.13b P-M-ϕ' for hollow section using incremental moment procedure

A final example is shown in Fig. 5.14. This is the same section as in Fig. 5.12 except there are two interior walls to stiffen the section in the long axis direction. The section was analyzed by an incremental uniaxial curvature procedure with the same constant axial load and uniaxial bending about the strong axis as in the previous example. The results shown in Fig. 5.15 from the analysis are as expected. The addition of the inner walls and the reinforcement in the inner walls provided for a larger attained moment and ultimate curvature.

5.2.3 Program BIMPHI Limitations and Extensions. Any analytical method is limited by the assumptions made in arriving at the mathematical model. The principal assumption of the fiber model that small changes in strain can be linearly related to small changes in force and other assumptions such as plane sections before bending remain plane after bending were enumerated in Chapter 4. The user should be aware of them and recognize the associated limitations.

BIMPHI is limited to closed symmetrical sections. At present, the program handles the sections illustrated in Fig. 5.5. The subroutines which generate the fiber mesh are limited to 200 fibers. Other limitations on input variables include a maximum of three steel rings to model the reinforcement, and a maximum number of interior walls for a cellular section of five. BIMPHI is equipped with some automatic error checks, but the user should prepare and check all input data carefully.

BIMPHI is easily adaptable to other sections not presently considered in the program. Subroutines which generate the concrete fibers and the steel fibers for a different type section would have to be prepared. The calls for these subroutines would be made at the appropriate locations in the driver program. If it is desired to model the section with more than 200 fibers, the fiber generator subroutines will need changing to accommodate more fibers. It is



Side steel = 19.1 in.^{2}

End steel = 11.8 in.^{2}

Inner wall steel = 5 in.^{2}

Total steel = 71.8 in.^{2}

f'_c = 4000 psi (unconfined concrete)

f_y = 40000 psi

P = 11660k

Fig. 5.14 Cellular section analyzed

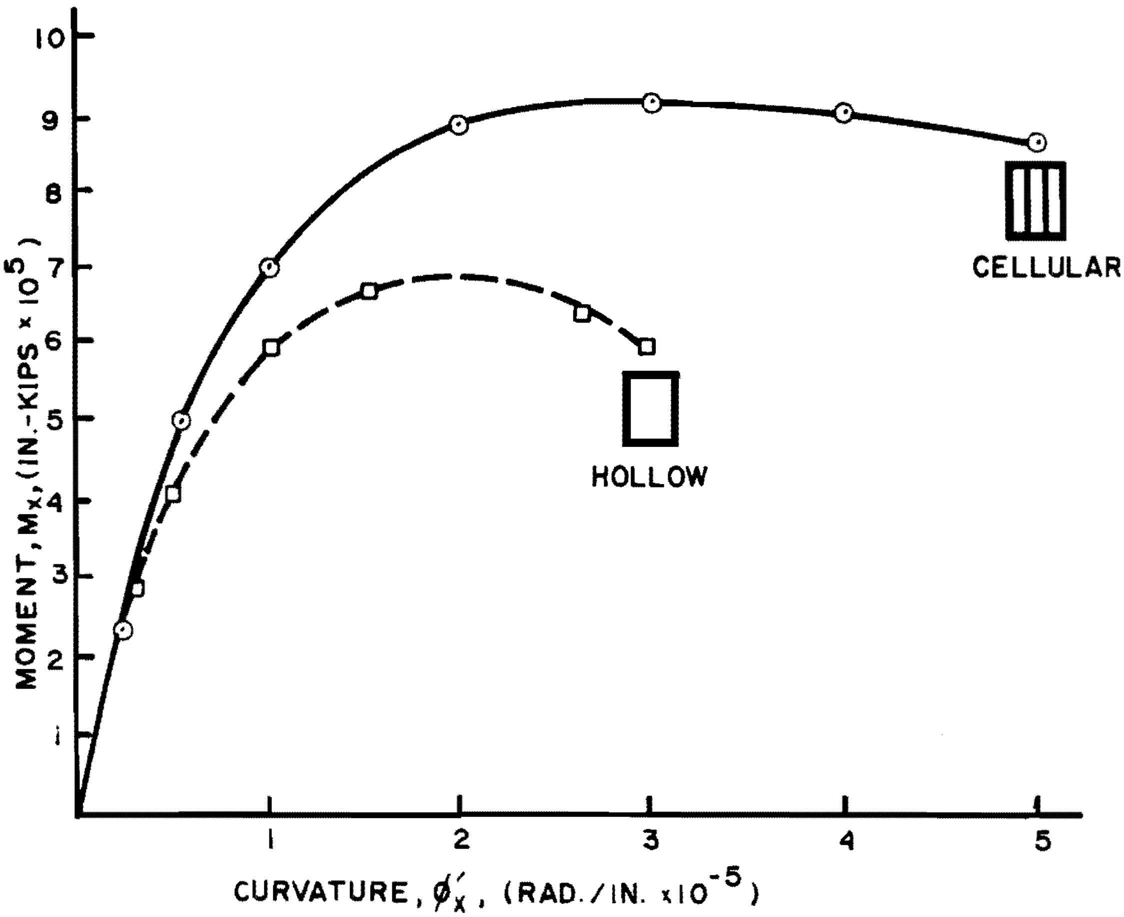


Fig. 5.15 P-M-ϕ' curve for cellular section

believed 200 fibers are more than adequate for modeling most concrete sections and reinforcing steel.

5.3 Program PIER

PIER was originally written in FORTRAN IV for The University of Texas at Austin CDC Cyber 170/750 computer and is adaptable to other systems. It is in conformance with the State Department of Highways and Public Transportation requirement that programs conform to FORTRAN 77. Even though PIER is a comprehensive analysis program, it needs no peripheral computer devices for a solution. A listing of the FORTRAN 77 program and explanation of its use can be found in Appendix B.

5.3.1 Program PIER Details. This program is intended for generalized second order analysis of slender single piers. PIER analyzes a reinforced concrete beam-column subjected to static monotonic biaxial loading. A general flowchart of the program is given in Fig. 5.16. The pier can have any of the cross sections illustrated in Fig. 5.5 and any vertical configuration. The concrete and steel fiber generator subroutines in PIER are the same as in BIMPHI. Input data to describe the pier depend on the vertical configuration.

The pier is divided into a number of individual members or segments as specified. Each segment is divided into ten equal sections for purposes of computing the stiffness matrix of the segment. Each section of a segment has the same fiber properties as the middle section. An example pier is shown in Fig. 5.17. For a pier of constant cross section as shown in Fig. 5.18 only one section needs to be defined. The fiber properties of each segment will be identical. If the cross section of the pier varies linearly from top to bottom as illustrated in Fig. 5.19, two sections are needed to define the pier. The geometry of the middle section of the top and bottom segments are input. The program assumes a

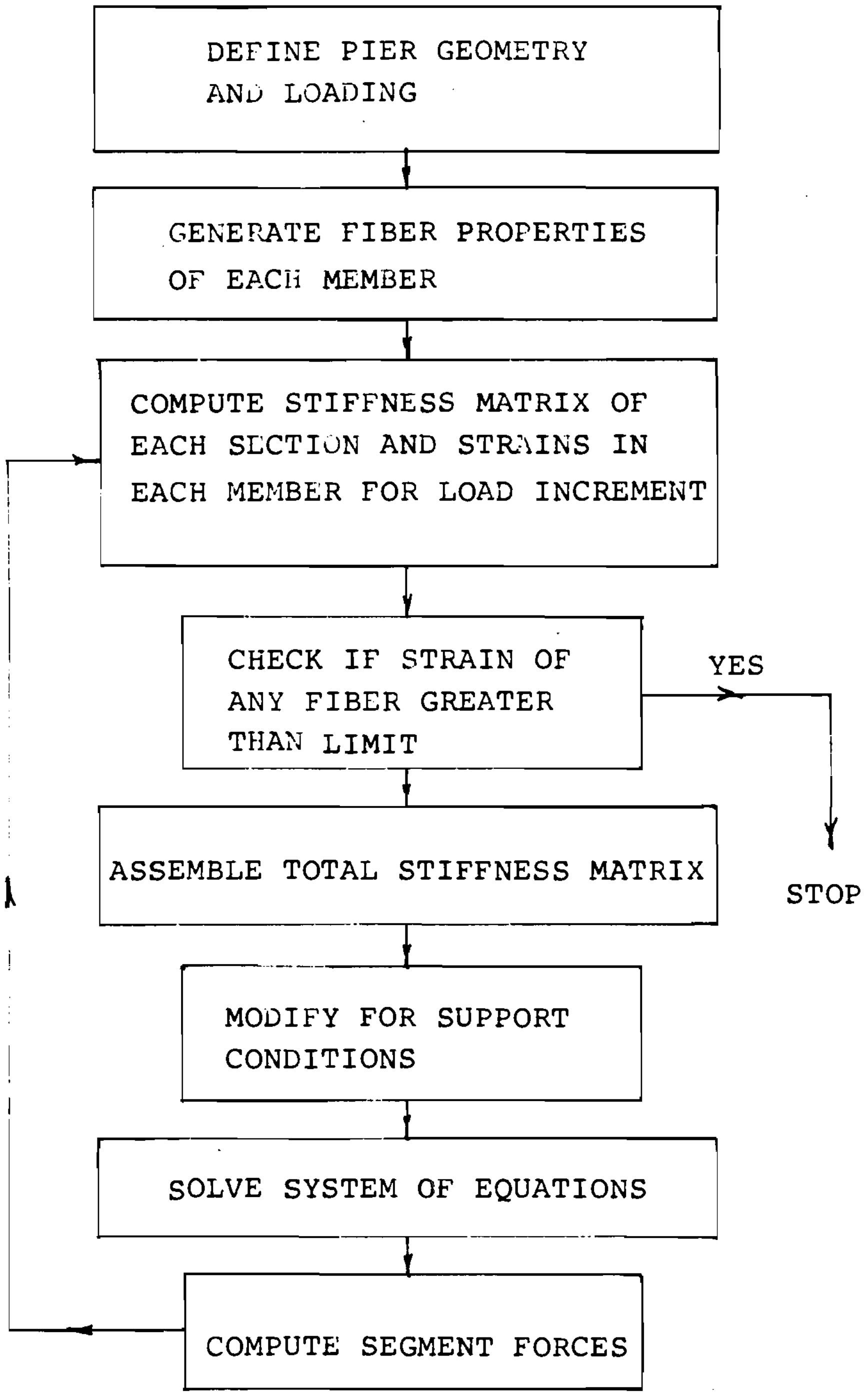


Fig. 5.16 Flowchart of program PIER

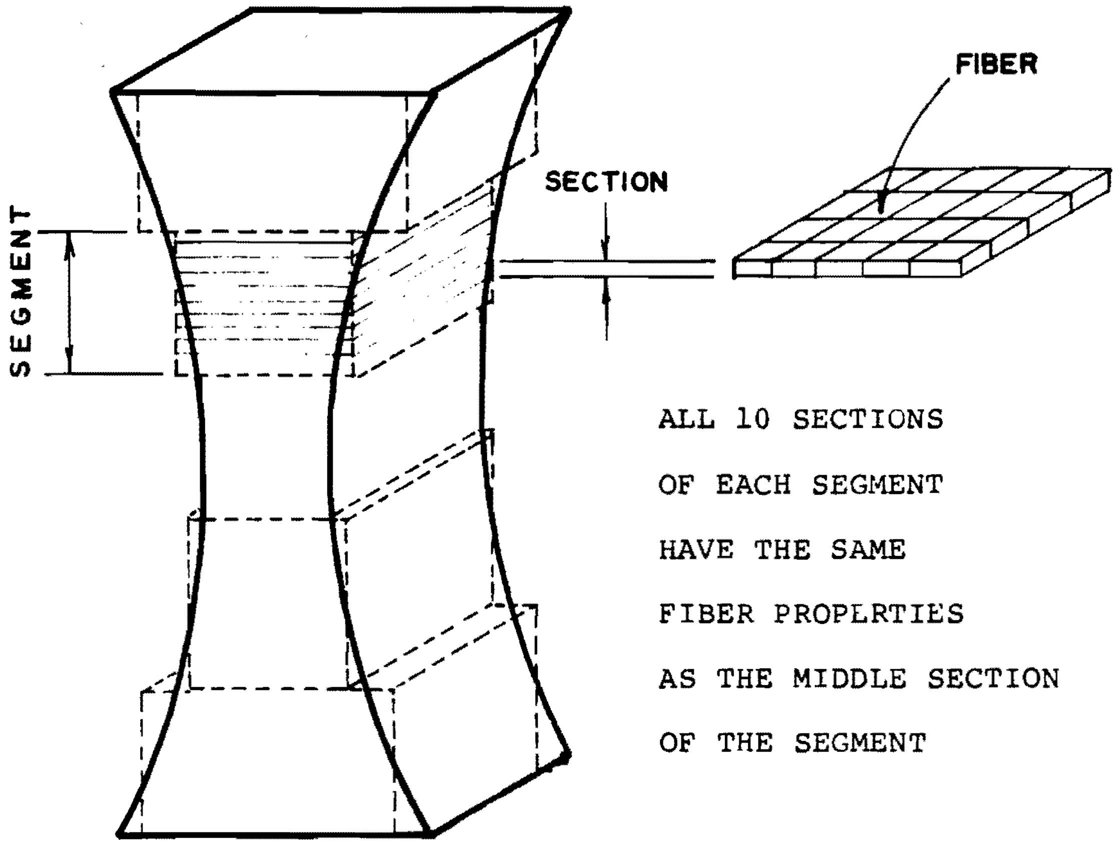


Fig. 5.17 Discrete fiber model representation of a nonprismatic pier

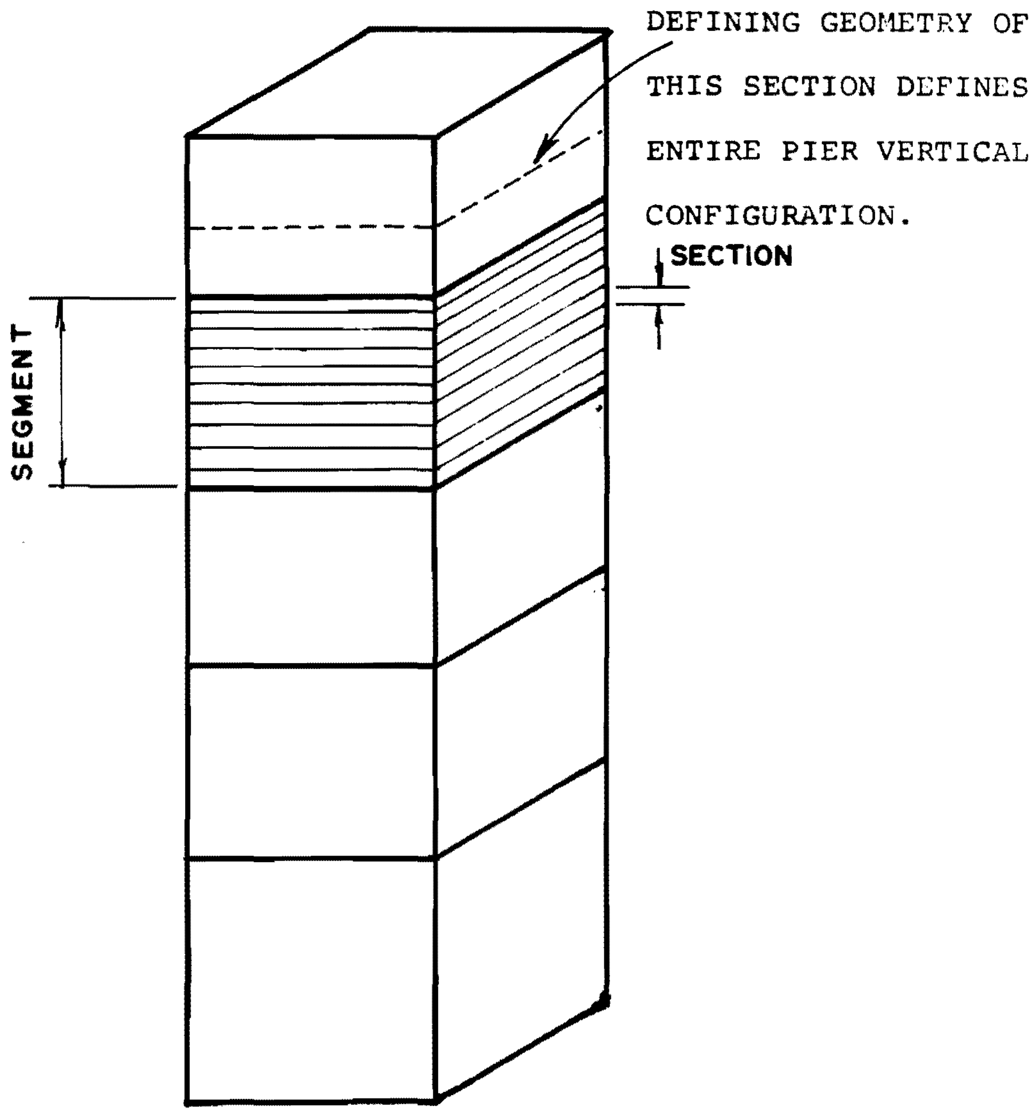
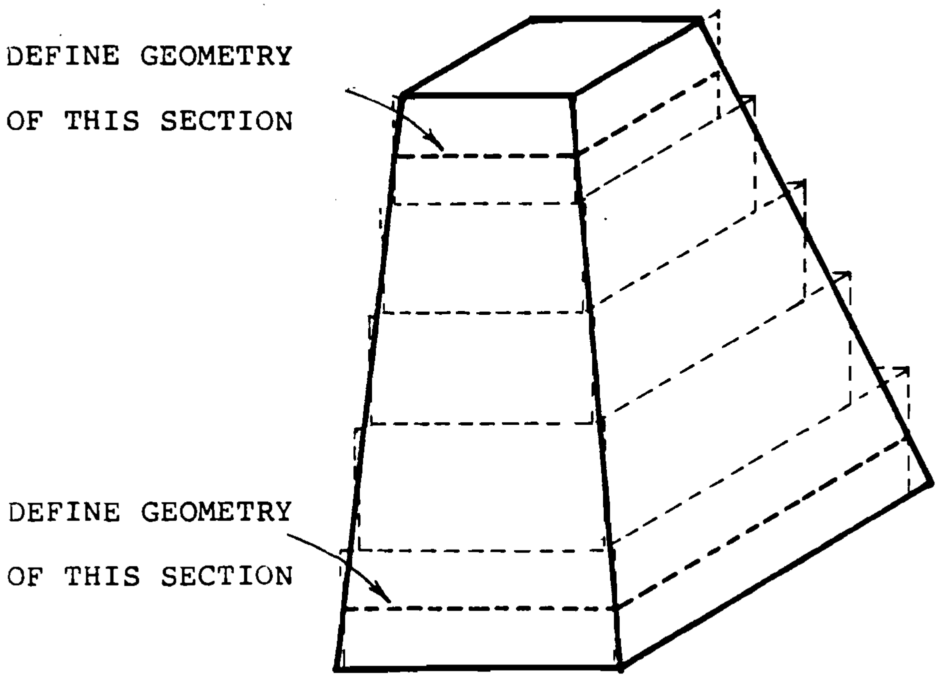


Fig. 5.18 Pier of constant cross section

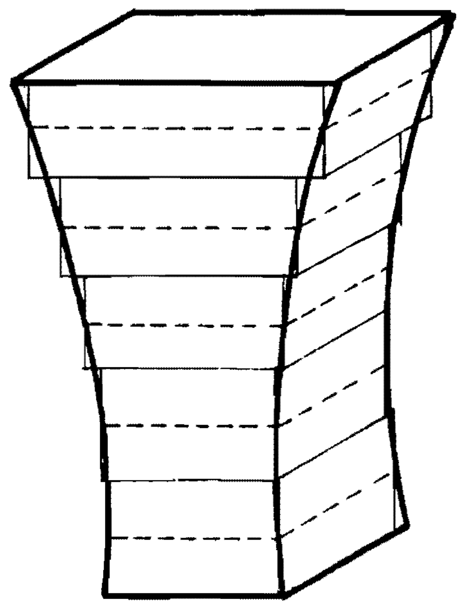


DEFINE THESE TWO SECTIONS TO DEFINE THE ENTIRE PIER LONGITUDINAL CONFIGURATION

Fig. 5.19 Pier of linear cross-sectional variation linear variation between these segments for defining the fibers of the other segments. For a pier of arbitrary vertical configuration as in Fig. 5.20, the individual section geometry for the middle of each segment are input. The ten sections of each segment are assumed to have the same properties as the section input for that segment. There will be the same number of sections input as there are number of segments designated.

The axes of the pier and the numbering scheme for the joints and segments is from bottom to top as shown in Fig. 5.21(a). There are no implied units in the program. The user must use consistent units. The incremental loading is specified at the free joints as forces and moments. The forces are assumed to be positive if in the direction shown in Fig. 5.21(b). The moments are assumed to be positive if given by the right-hand vector rule as in Fig. 5.21(c). There are six degrees of freedom for each joint: (1) x-displacement; (2) y-displacement; (3) z-displacement; (4) rotation about the x-axis; (5) rotation about the y-axis; and (6) rotation about the z-axis. All degrees of freedom of a joint are assumed free unless the joint is specified as a support. For the supports, a release code is specified. A "0" specified for a degree of freedom implies it is free, and a "1" implies it is fixed. Forces may be specified at a support in a particular degree of freedom if the release code is "0" for that degree of freedom. Displacements may be specified at a support if the release code is "1" for a degree of freedom. There cannot be forces and displacements specified for the same degree of freedom at a joint.

However, there may be some applications where it would be desirable to first impose a specified deflection at a joint, and then apply forces to the same joint. This cannot be done directly. What can be done is to perform an analysis with an equivalent static force system which will give the desired specified initial displacement and then continue with the desired load case. As an example, the



DEFINE THE CENTER SECTION (DASHED LINE) GEOMETRY OF EACH SEGMENT TO DEFINE ENTIRE PIER VERTICAL CONFIGURATION

Fig. 5.20 Pier of arbitrary longitudinal configuration

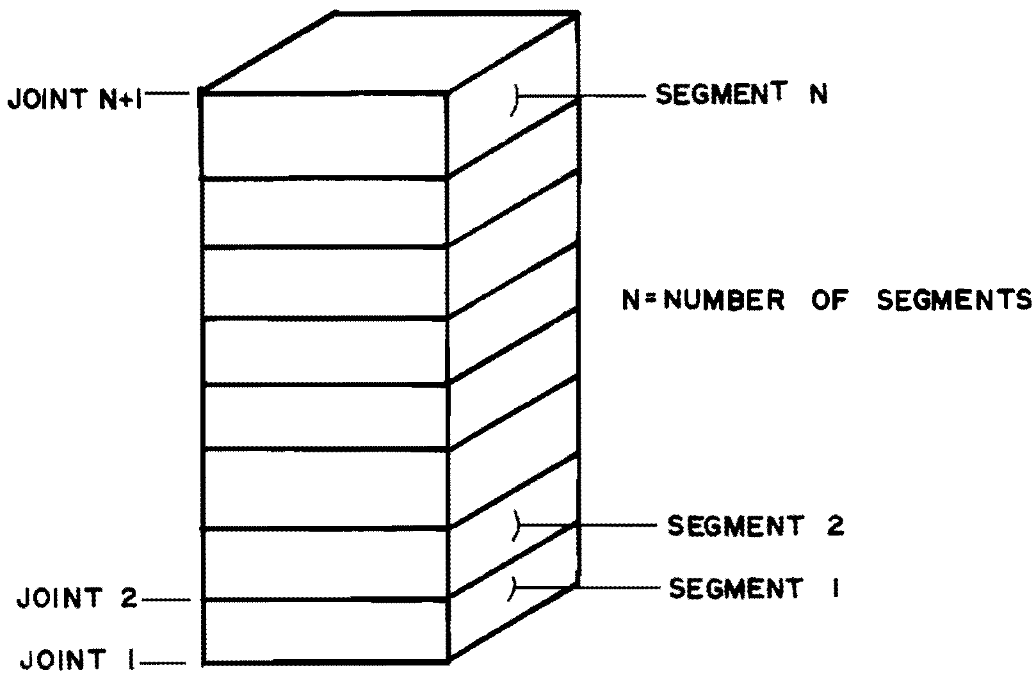


Fig. 5.21a Member and joint numbering

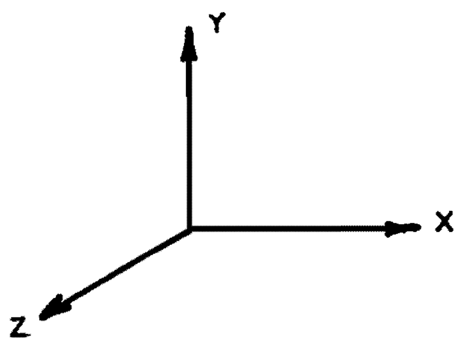


Fig. 5.21b Positive force sign convention

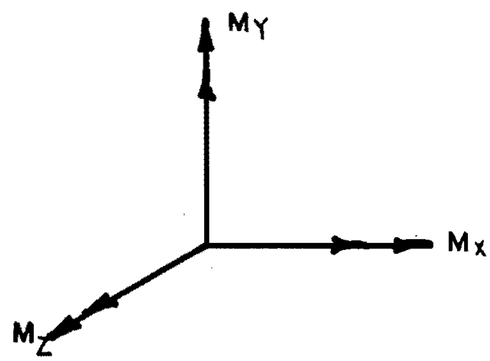


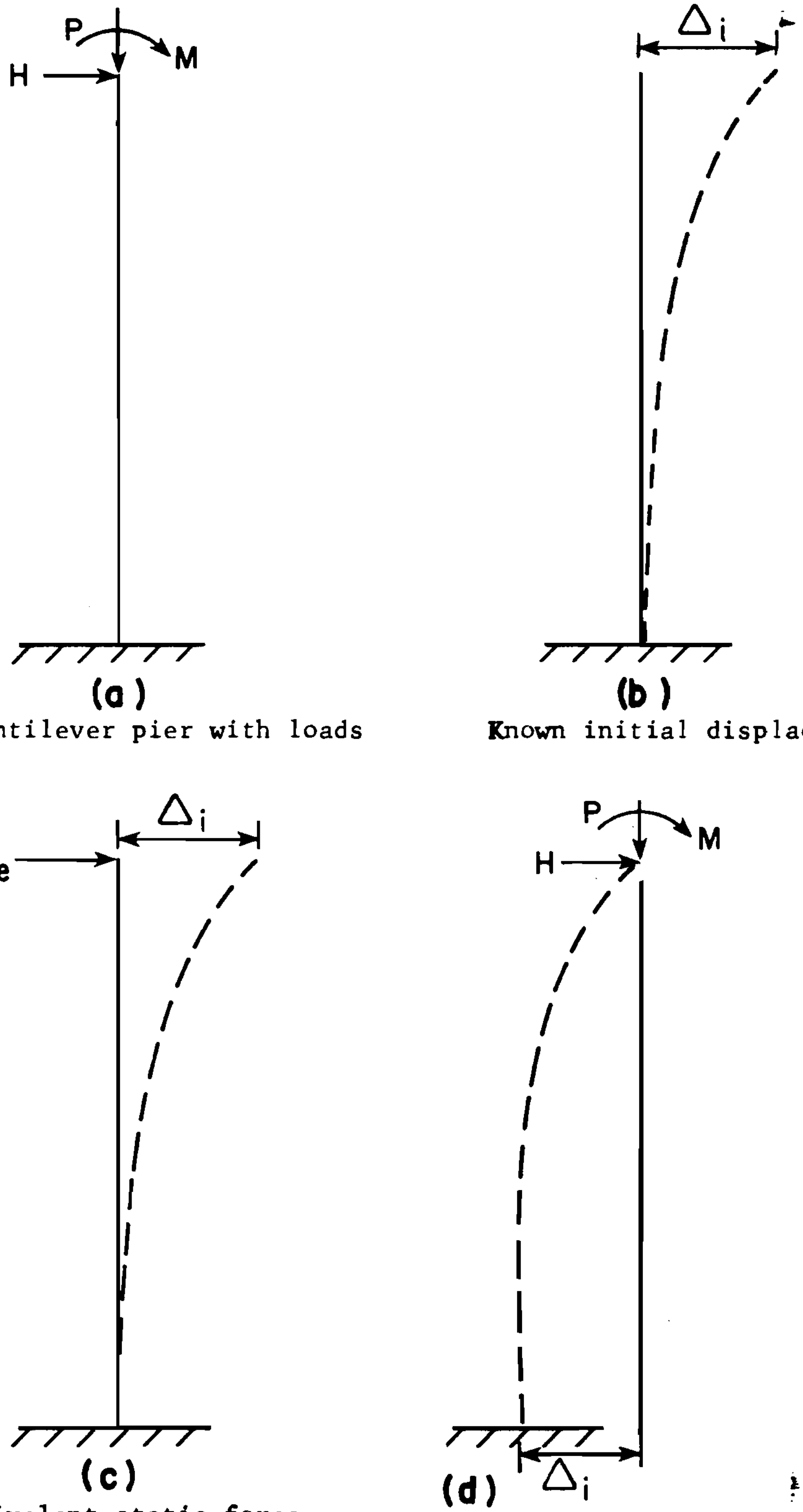
Fig. 5.21c Positive moment sign convention

cantilever pier in Fig. 5.22a is subject to the imposed loads shown. However, before any loads are imposed, the pier tip undergoes a known initial displacement Δ_i due to creep and shrinkage of the superstructure, as in Fig. 5.22b. First, by an iterative procedure, an equivalent static force F_e is found, which produces the known specified displacement Δ_i at the pier tip. This force F_e represents load case 1 of the final analysis, as shown in Fig. 5.22c, and the other applied loads represent load case 2 as in Fig. 5.22d. Thus, by this procedure an initial displacement is imposed before the loads are applied.

There can be as many load cases and load increments for each case as desired to determine the pier behavior. The incremental loading is applied in the number of increments specified for that load case. The total load at the end of the load case equals the number of increments multiplied by the incremental load. If another load case is desired, the new loading and number of increments are specified. The new incremental displacements and incremental forces are added algebraically to the displacements and forces of the previous loading and so forth for other load cases.

Loads can be input in a number of ways. If the designer wishes to check a pier for a given set of unfactored or factored loads he can input incremental loads which total to those values. In addition, the program has the capability of converting input unfactored loads into AASHTO factored load combinations if the designer wishes to check a number of load combinations. In this case the designer inputs the unfactored loads for each type loading and specifies the load combinations which he desires to have checked.

The points of load application are restricted to model points which are joints between segments. Distributed loads must be input in terms of statically equivalent concentrated joint loads. Point forces and moment vectors must be resolved into their components in the major axes of the global coordinate system shown in Fig. 5.21.



Cantilever pier with loads

Known initial displacement

Equivalent static force system - Load Case 1

Load Case 2

Fig. 5.22 Illustration of specifying displacement and force at same point

The user can input loads either as specified incremental loading or as total unfactored applied loads. In the first option the user decides the number of load cases he wishes to investigate and must also indicate the magnitude and number of load increments desired. This option is of primary use in determining the capacity of a given bent under a given loading configuration. In the second option the program will factor the applied loads using the appropriate AASHTO load factors (see Table 5.2) for those loading combinations specified by the user. This allows the computer code to manipulate the various load combinations and load factors and to check the given bent for those cases. For each of the combinations specified the program first factors the applied loads using the appropriate AASHTO load factors. A target maximum load is computed as 132% of the factored loads. The purpose of the excess is to provide a margin so that the program can choose loading increments which have a good likelihood of bringing the frame to full capacity. The program will signal the actual capacity reached. The actual analysis is always incremental but this mode of input relieves the user of inputting detailed incremental load values. The program calculates proportional load increments for each applied force or moment and applies them automatically in three load cases until failure is reached. The first load case applies 80% of the total factored loads divided into 50 increments. The second load case then applies 20% of the total factored loads divided into 30 increments. The third load case then applies the extra 32% of the total factored loads in 30 increments. The computed failure load is determined and given as output. In addition, the program computes the ratio of the total computed failure load applied to the design factored load. Information messages are printed to indicate the degree of under or over design of the bent. These messages only appear when using this AASHTO load factor option of load application. The bounds for the information messages are indicated in Table 5.3 as a function of a ratio PER which is the computed failure load divided by the factored design load. The same load treatment and information

TABLE 5.2 AASHTO LOAD FACTORS

		β Values														
Load Type		1	2	3	4	5	6	7	8	9	10	11	12	13	14	
Loading Group		D*	D*	L+I	CF	E*	E*	B	SF	W	WL	LF	R+S+T	EQ	ICE	γ Values
Program	AASHTO															
1	I	β_d=1.0	β_d=0.75	1.67	1.0	1.3	0.5	1.0	1.0	0	0	0	0	0	0	1.3
2	IA			2.20	0	0	0	0	0	0	0	0	0	0	0	1.3
3	II			0	0	1.3	0.5	1.0	1.0	1.0	0	0	0	0	0	1.3
4	III			1.0	1.0			1.0	1.0	0.3	1.0	1.0	0	0	0	1.3
5	IV			1.0	1.0			1.0	1.0	0	0	0	1.0	0	0	1.3
6	V			0	0			1.0	1.0	1.0	0	0	1.0	0	0	1.25
7	VI			1.0	1.0			1.0	1.0	0.3	1.0	1.0	1.0	0	0	1.25
8	VII			0	0			1.0	1.0	0	0	0	0	1.0	0	1.3
9	VIII			1.0	1.0			1.0	1.0	0	0	0	0	0	1.0	1.3
10	IX			0	0	1.3	0.5	1.0	1.0	1.0	0	0	0	0	1.0	1.20

*See AASHTO Manual, Section 1.2.22; only 1 value of dead load a 1 value of earth pressure may be used per problem.

TABLE 5.3 FPIER INFORMATION MESSAGES

PER = $\frac{\text{Computed failure load}}{\text{Factored design load}}$	Information Message
PER < 70%	Gross underdesign
70% ≤ PER < 85%	Underdesign
85% ≤ PER < 95%	Slight underdesign
95% ≤ PER ≤ 105%	Correct
105% < PER ≤ 115%	Slight overdesign
115% < PER ≤ 130%	Overdesign
130% < PER	Gross overdesign

messages are given in program FPIER. The treatment of the capacity reduction factor (ϕ factors) must be handled by the user. Since factors for pure flexure and for combined axial compression and flexure vary, and vary in a complex fashion, the automated programming is prohibitive. The user generally knows the range of loads and nature of the members. Since $P_u = \phi P_n$ or $M_u = \phi M_n$, it is easiest if the loads and moments input are P/ϕ or M/ϕ. By preapplying the appropriate ϕ factor in this way the design will have a proper factor of safety. For investigations of existing structures with known material properties, or for checking of laboratory specimens where properties and dimensions are known very well, values of ϕ approaching 1 may be appropriate. In this way the program can be used in a wide variety of applications.

All member joints are assumed to be free to translate and rotate unless the boundary conditions are specified to impose translational and/or rotational restraint. Any desired displacement values may be specified at supports. However, in the AASHTO load factor option, specified displacements or rotations are not directly permitted. To achieve specified displacements, the user must input an equivalent static force system that produces the desired displacements.

Translational springs and rotational springs may be specified at joints for the degrees of freedom desired (see Fig. 5.23). The stiffness of a translational spring is given in force per length, and the stiffness of a rotational spring in force per radian. Springs are extremely important at the foundation of the pier since the pile cap and soil usually do not provide full restraint. Springs may be used at the top of the pier to model the restraint imposed by the bridge girders.

The appropriate value of degree of fixity for various foundation conditions which should be input as the spring stiffness

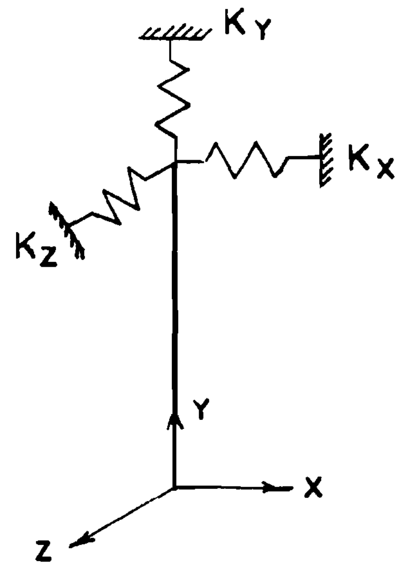


Fig. 5.23a Translational springs at a joint

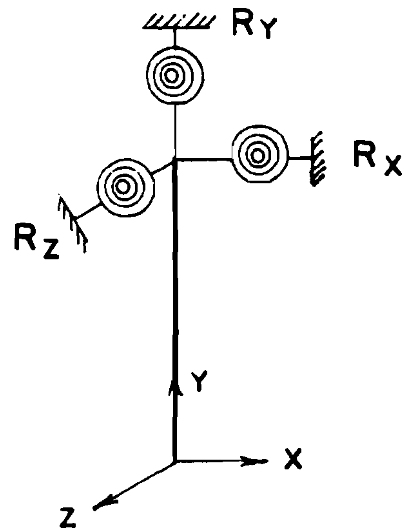


Fig. 5.23b Rotational springs at a joint

requires a great deal of judgment. This area of soil-structure interaction has been addressed by various authors [20-24]. Calculation of appropriate spring stiffnesses for shallow foundations usually considers the general system shown in Fig. 5.24(a) and represents it by the analog system shown in Fig. 5.24(b). This analog is widely used for dynamic analysis. There are an infinite number of combinations of spring stiffness, damping, and added mass which model the response. However, the static spring stiffness is normally used. These static values seem most desirable for static loading of bridge piers. Suggested calculation procedures and values are available in Refs. 20-22 for circular and rectangular footings. An illustrative example of computations is shown in Fig. 5.25 for a circular footing. Because these approaches model the soil as an elastic half space and because of general uncertainties involved with assumed soil properties, it is advisable to examine the sensitivity of slender pier designs to the substructure restraint. This can be done by halving and then doubling the assumed restraint values. If the pier forces change less than 10-20% this approach is probably adequate. If the forces in the pier change radically, a more rigorous approach like the use of a finite element analysis should be used to determine base restraint. A special case exists where the base restraint changes at higher load levels.

The output from PIER at the end of each load increment includes displacements and rotations at each joint, and the forces and moments at the end of each member. The analysis is completed either when all specified input load cases and load increments are completed with no failure, or when an assumed material failure occurs. A concrete compression failure is assumed if any concrete fiber strain exceeds the specified ultimate strain of concrete. A message indicates if such a condition occurs. A tensile failure is assumed if any steel fiber strain exceeds 1%. A message indicates if such a condition is reached. If the pier undergoes an assumed material failure, the displacements and forces for the last increment failure will be printed.

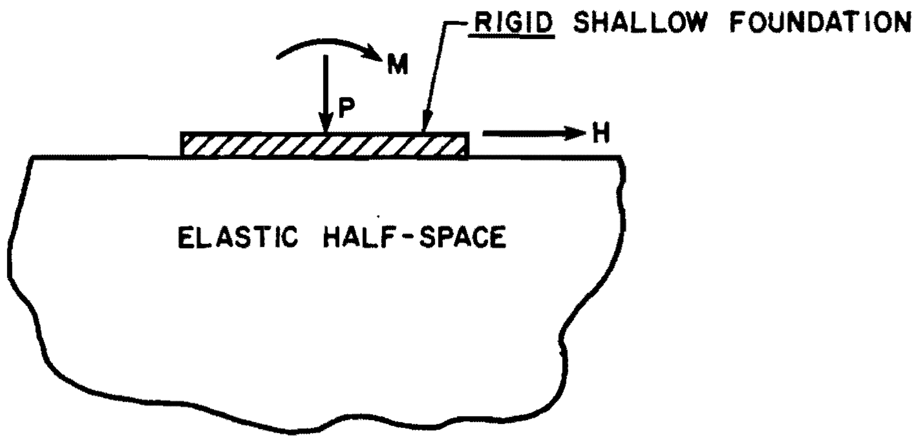


Fig. 5.24a Shallow foundation

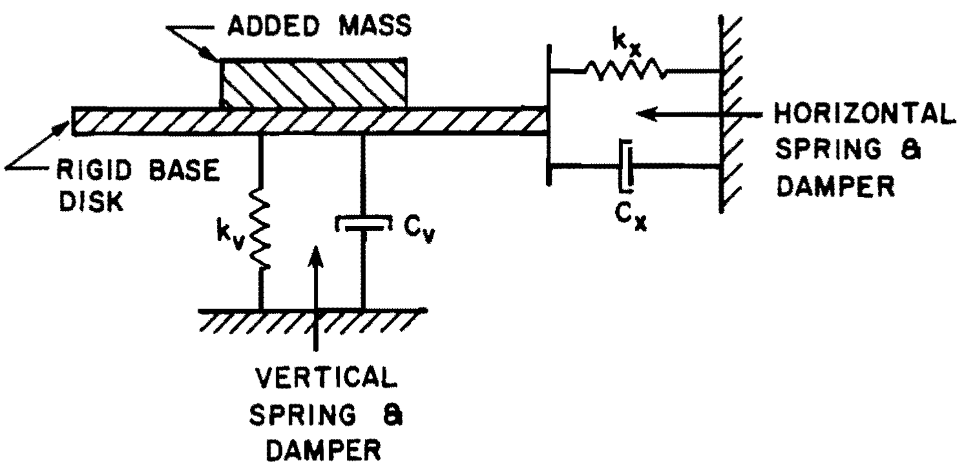
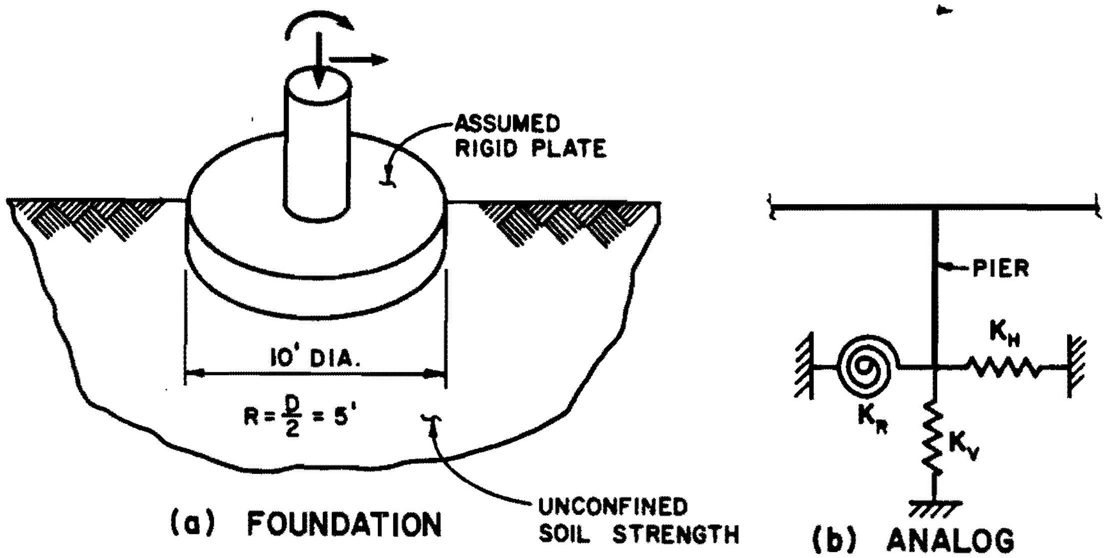


Fig. 5.24b Equivalent analog



C_u = 2000 psf

ν = Poisson's ratio = 0.40
(Saturated soil)

$$E_{static} = 400\ C_u \quad \text{(Ref. 23)}$$

$$= (400)(2000\ \text{psf}) = 800000\ \text{psf}$$

$$G = \frac{E}{2(1+\nu)}$$

$$= \frac{800000}{2(1+0.4)} = 285{,}700\ \text{psf}$$

$$K_v = \frac{4GR}{1-\nu} \quad \text{(Ref. 21)}$$

$$K_v = \frac{(4)(285{,}700)(5)}{1-0.4} = 9.5 \times 10^6\ \text{lb/ft}$$

$$K_H = \frac{18.2\ GR(1-\nu^2)}{(2-\nu)^2} \quad \text{(Ref. 21)}$$

$$K_H = \frac{(18.2)(285{,}700)(5)(1-(0.4)^2)}{(2-0.4)^2} = 8.5 \times 10^6\ \text{lb/ft}$$

$$K_R = 2.7\ GR^3 \text{ for } \nu = 0 \quad \text{(Ref. 24)}$$

$$K_R = (2.7)(285{,}700)(5)^3 = 9.6 \times 10^7\ \text{lb-ft/radian}$$

Fig. 5.25 Example of pier restraint

In addition to the material failure conditions there are two other possible failure modes. If the member is restrained and a highly stressed section attains a maximum moment value, it is possible to form a negative tangent stiffness since increasing curvatures require a reduction in moment. This negative tangent stiffness causes a mathematical instability in the computation procedure and is usually signalled by a reversal in sign in the change in deflection. Such a reversal in the trend of deflections is a computational signal that a "plastic hinge" has been formed and should be considered as the useful or "hinging" capacity of the section. Use of the recommended maximum usable strain of 0.003 given in AASHTO Art. 1.5.31(A)(2) will minimize such failures by putting a tighter limit on concrete compression strains. The output should always be carefully examined for such indications. In addition, formation of a negative stiffness matrix signals a stability failure. An actual member instability failure is signalled by rapidly increasing displacements as shown in Fig. 5.26 Such instability failures may occur before the assumed critical strains are reached. Again, their occurrence may be easily seen by the output. They usually are terminated by achieving a material failure condition. The user should always check for a hinging or stability failure even if the pier did not have a material failure and has completed all loading. In order to give the designer guidance on how close the design level factored loads were to the actual pier capacity, both programs PIER and FPIER give output messages indicating the general ratio of applied load to capacity. The messages only appear when using the AASHTO load factor option of load application.

5.3.2 Examples Using Program PIER. Several example piers are shown to demonstrate the use of this program. The first example is a solid prismatic pier with dimensions, cross-sectional properties, supports, and loading as shown in Fig. 5.27. The pier was subjected

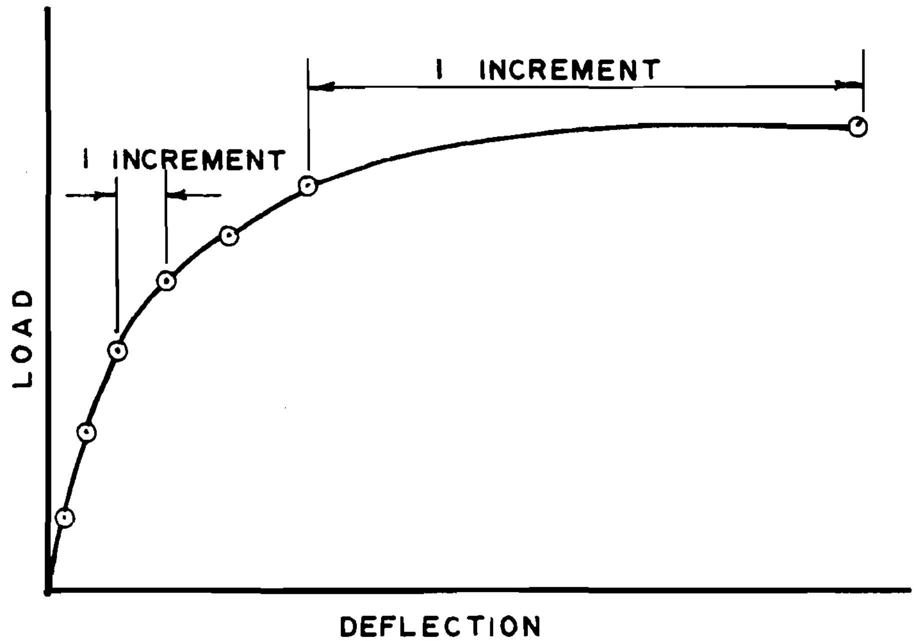
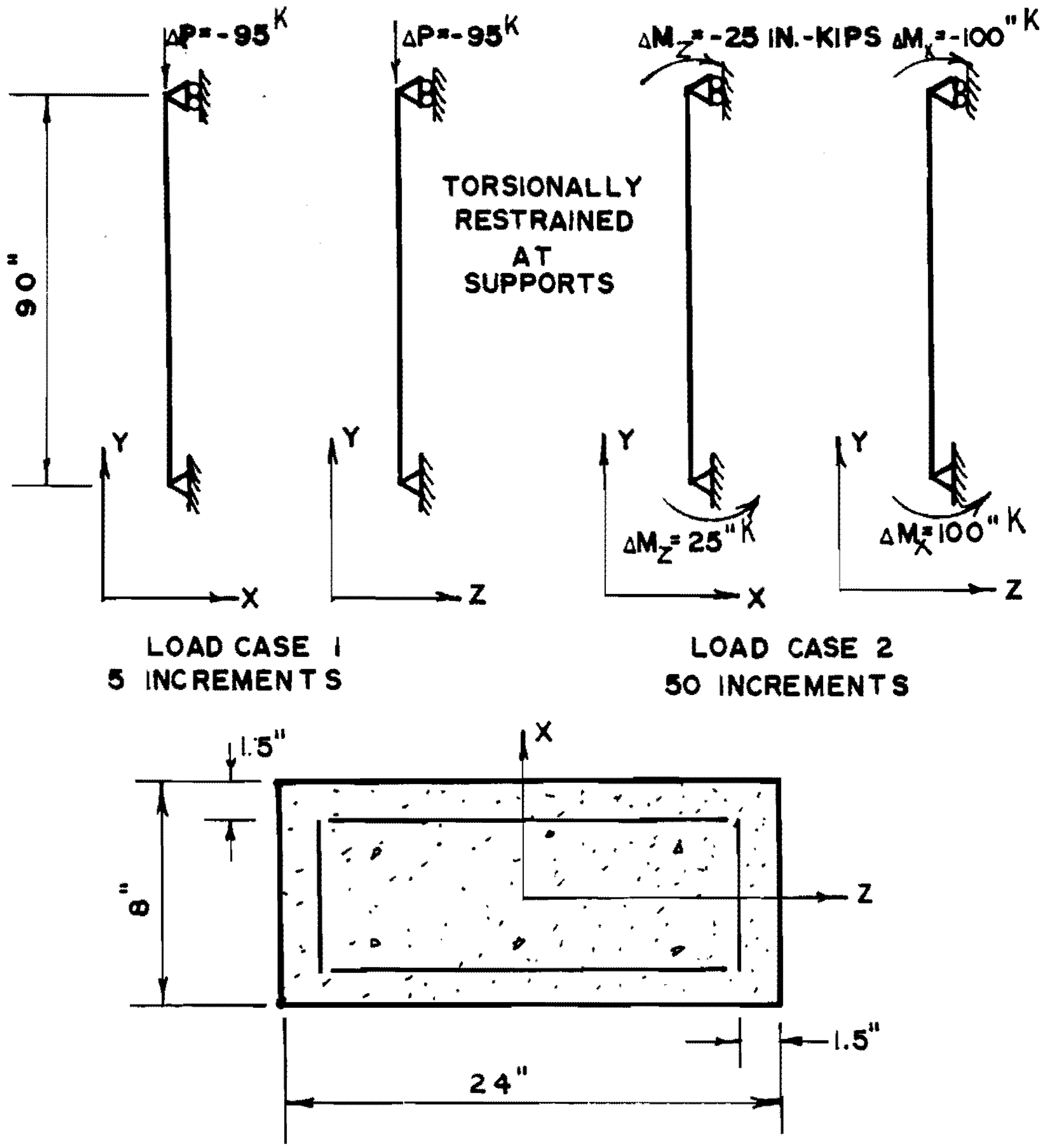


Fig. 5.26 Rapidly increasing deflection signalling a stability failure



f'_c = 4000 psi (unconfined concrete)

f_y = 40000 psi

Side steel = 0.48 in.^{2}

End steel = 1.44 in.^{2}

Total steel = 3.84 in.^{2}

$E_c = 3.6 \times 10^6$ psi

$E_s = 29 \times 10^6$ psi

10 segments

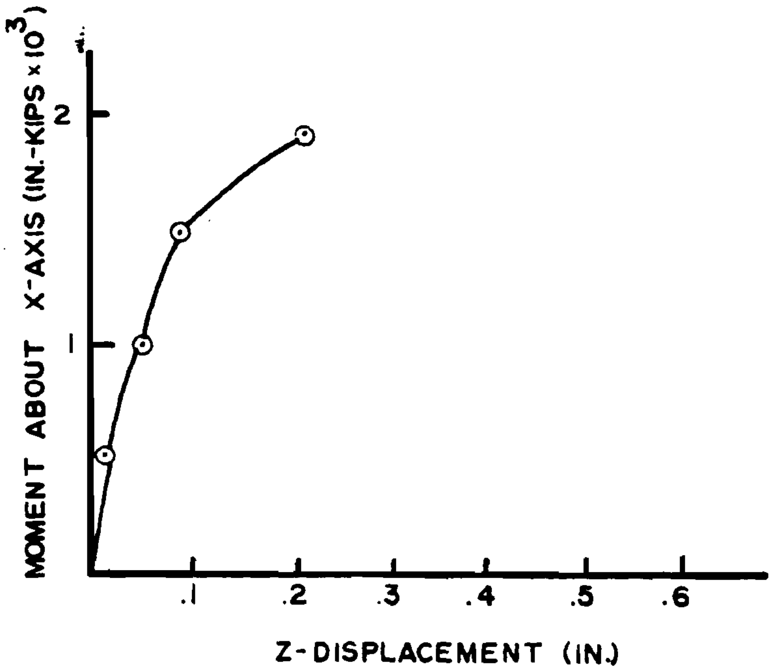
11 joints

Fig. 5.27 Example of solid pier

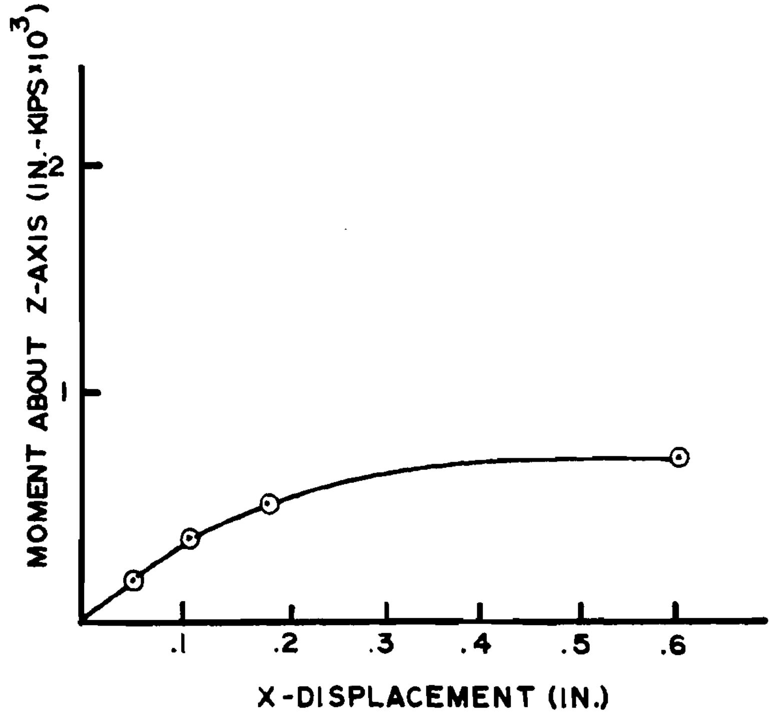
to two load cases. The first was a compressive axial load which totalled $0.6P_o$ after 5 increments. The second loading applied simultaneous incremental moments about the x- and z-axes. The pier had a concrete compression failure in segment 2 in load increment 19 of load case 2. A graph of the moment about the x-axis versus the z-displacement at the center joint is presented in Fig. 5.28(a). A graph of the moment about the z-axis versus the x-displacement of the center joint is shown in Fig. 5.28(b). Using the Bresler reciprocal thrust equation [15], the computer ultimate biaxial thrust is 466k. At the time of the assumed concrete compression failure, PIER predicted an ultimate biaxial thrust of 475k. This value is within 2% of the Bresler value. This shows excellent agreement between the program and Bresler's equation.

The second example is a solid tapered pier on an assumed rigid foundation. Since it varied linearly from top to bottom, only two section geometries need defining. The pier is shown in Fig. 5.29. It was loaded with an incremental compressive axial load and incremental moments about the x- and z-axes. Graphs of the axial load versus tip displacements are shown in Fig. 5.30. The program results indicated a concrete compression failure in segment 10 at load increment 41. Due to the nature of the vertical configuration of the pier and the complex support conditions, conventional methods such as the moment-magnifier procedures and the Bresler equation cannot be used to predict the ultimate biaxial thrust.

The last example analyzed by PIER is the flared pier shown in Fig. 5.31. It consisted of seven members, thus seven sections were input to describe the pier (Fig. 5.32). The loading consisted of a compressive axial load applied in 5 increments, and x- and z-forces applied at each free joint applied in 50 increments. The pier was modeled as a fixed-fixed structure except axial deformation was permitted. Also, the concrete was assumed as confined concrete

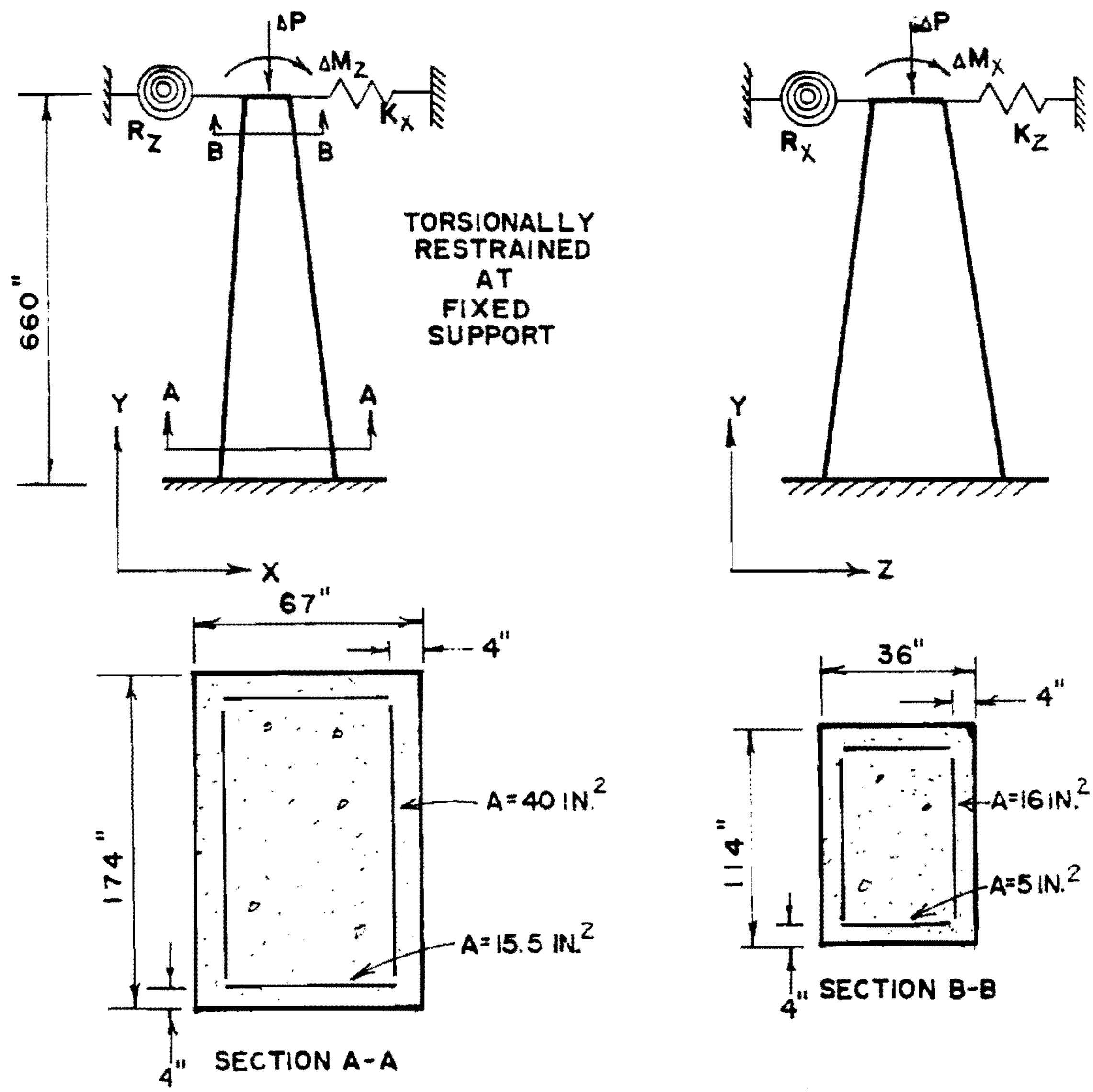


(a)



(b)

Fig. 5.28 Moment versus deflection of middle joint of solid pier



f'_c = 4000 psi (unconfined concrete)	R_x = 20 kip/rad
f_y = 40000 psi	R_z = 40 kip/rad
$E_c = 3.6 \times 10^6$ psi	K_x = 100 kip/in.
$E_s = 29 \times 10^6$ psi	K_z = 500 kip/in.
10 segments	ΔP = -200 kip
	ΔM_x = 1000 in.-kip
	ΔM_z = 2000 in.-kip

Fig. 5.29 Example of tapered pier

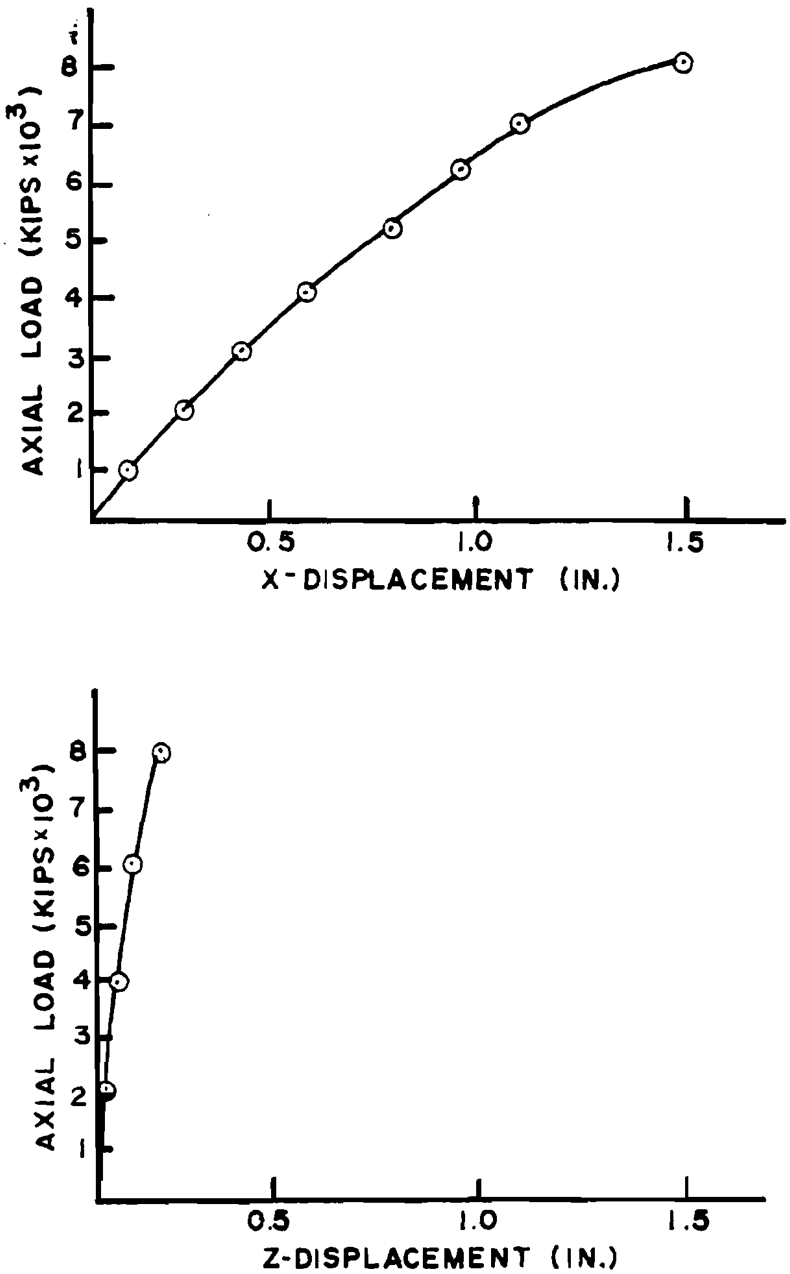


Fig. 5.30 Axial load versus tip displacement for tapered pier

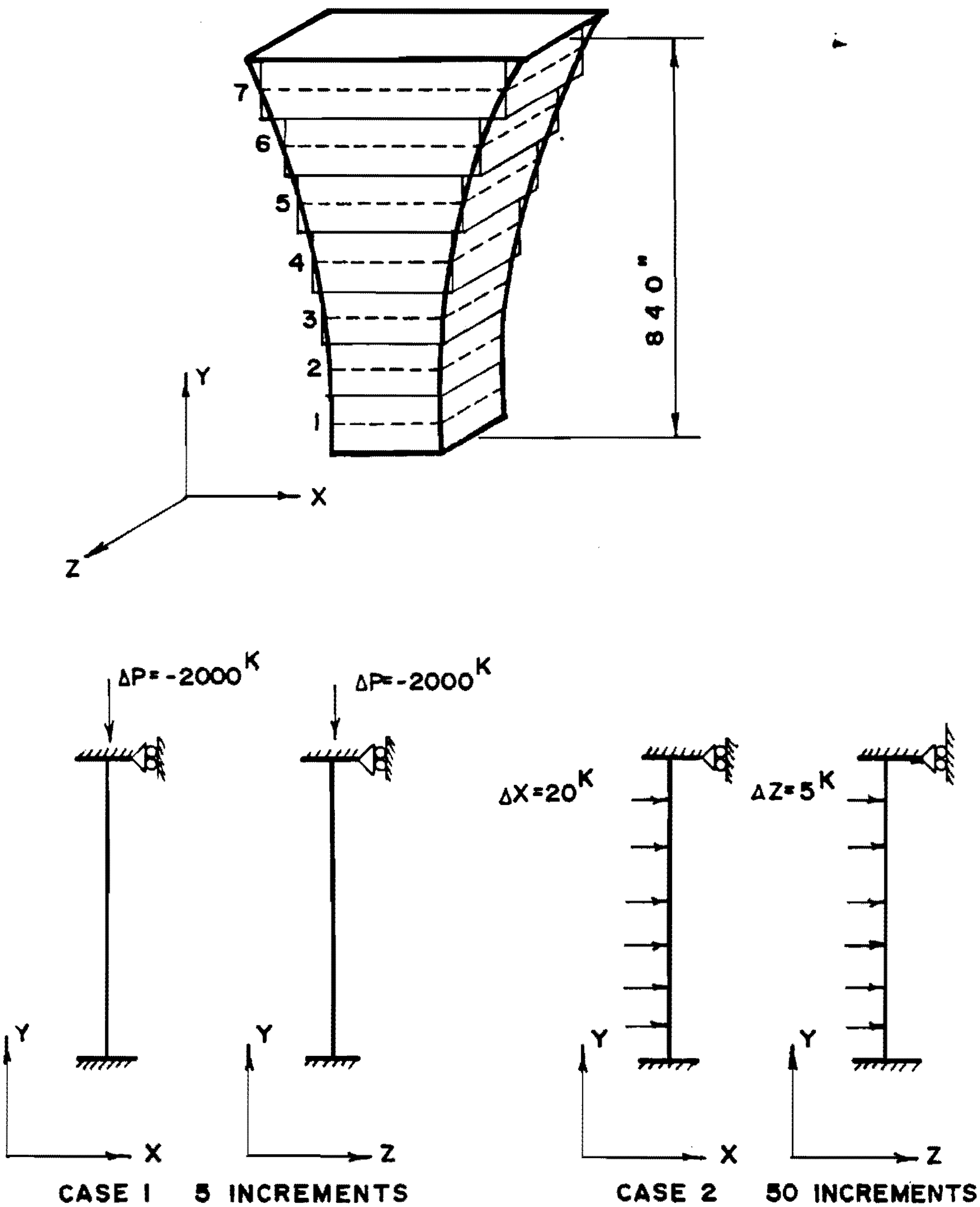
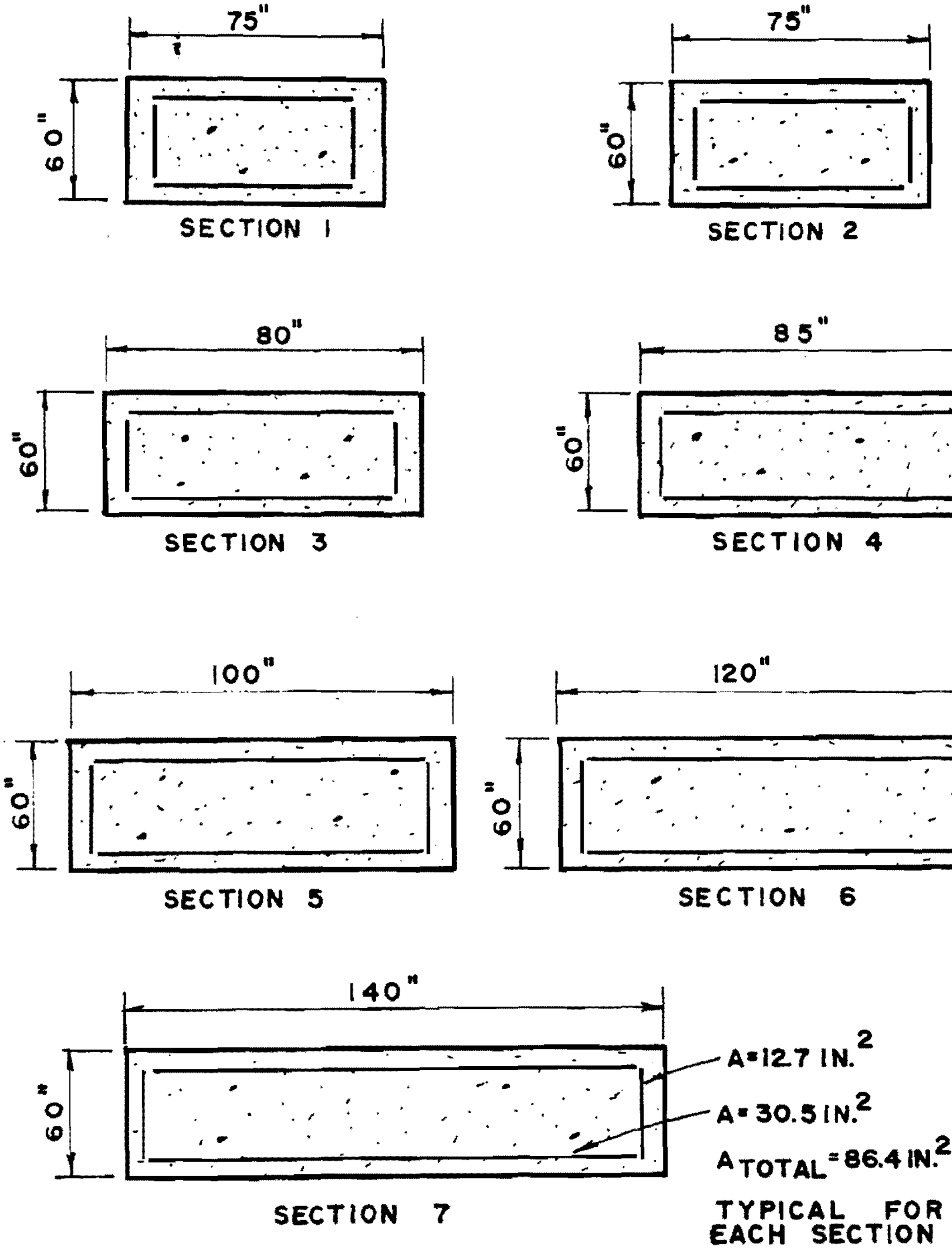


Fig. 5.31 Example of flared pier



f'_c = 4000 psi (confined concrete) $E_c = 3.6 \times 10^6$ psi

f_y = 40000 psi $E_s = 29 \times 10^6$ psi

Fig. 5.32 Sections of flared pier in Fig. 5.31

with an ultimate limiting strain of 0.0045. The pier experienced a concrete compression failure in load increment 35 of load case 2 in segment 1. Graphs of the moment versus the deflection of joint 4 are in Fig. 5.33 and show only a little nonlinear behavior before failure.

The previous two examples of the tapered pier and flared pier would have been difficult to analyze by conventional methods such as the AASHTO moment magnification procedure because of their vertical configurations and the presence of biaxial bending. The examples show the versatility of program PIER. It can handle almost any type of cross section, vertical configuration, support conditions, and loading. There was no way to show all the results obtained from each analysis, so only a typical result from each was reproduced.

5.3.3 Program PIER Limitations and Extensions. All the limitations associated with program BIMPHI also apply to program PIER. One additional limit to input variables is the maximum number of segments to define the pier which is ten. Presently, the program treats the concrete as either all confined or unconfined. It would be possible to treat individual concrete fibers in the same pier as either confined or unconfined. Each fiber would need a code designating its status when the tangent modulus is calculated.

5.4 Program FPIER

FPIER was originally written in FORTRAN IV for The University of Texas at Austin CDC Cyber 170/750 computer and is adaptable to other systems. It is in conformance with the State Department of Highways and Public Transportation requirement that programs conform to FORTRAN 77. A listing of the FORTRAN 77 program and explanation of its use can be found in Appendix C.

5.4.1 Program FPIER Details. FPIER was specifically developed to analyze rectangular pier bents while incorporating all of the versatility and variety of cross section shape and variation

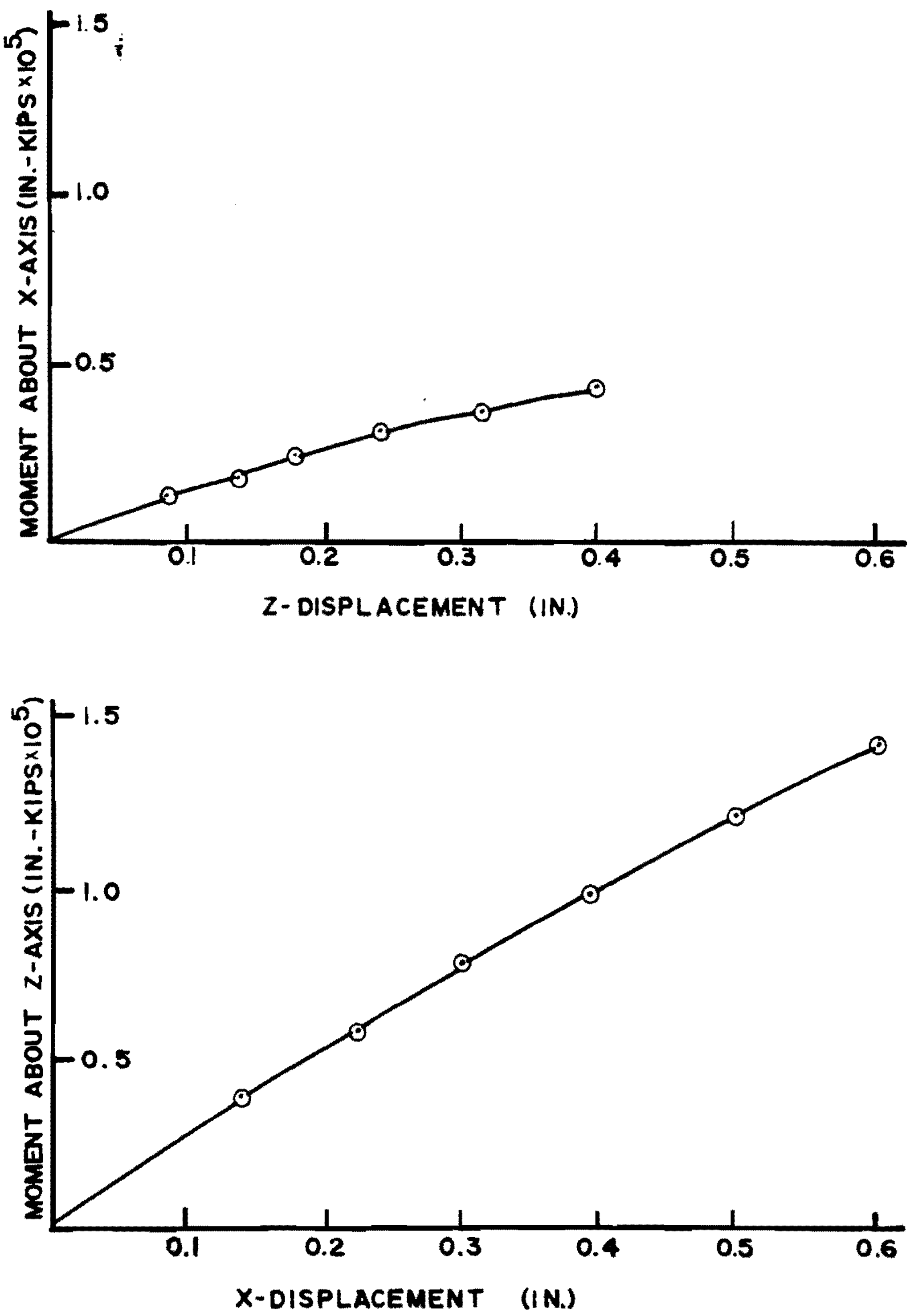


Fig. 5.33 Moment versus deflection of joint 4 of flared pier

along the column axis which is available in PIER. Thus this program can treat assemblages of straight, tapered or flared columns tied together with constant or variable depth bent cap guiders. The only constraint on the assemblage is that the pier bent must be rectangular and geometrically symmetrical in elevation with a maximum of two bays and three levels. The joint constraints and loadings do not need to be symmetric. Thus the program can readily handle involved multistory bents such as those shown in Fig. 5.34.

Each member of the bent is divided into segments which can be of unequal lengths and different cross sections as long as the overall geometric symmetry is maintained. For example, if the left column of a single bay portal frame is divided into eight segments then the right column is assumed to be divided into the same eight segments.

The points of load application are restricted to model points which are the joints between members or joints between segments. Distributed loads must be input in terms of statically equivalent concentrated joint loads. Point forces and moment vectors must be resolved into their components in the major axes of the global coordinate system shown in Fig. 5.35.

As in program PIER, the user can input loads in FPIER either as specified incremental loading or as total unfactored applied loads. A detailed description of these types of input loads is given in the previous section on PIER, which has the same capability.

All frame joints are assumed to be free to translate and rotate unless the boundary conditions are specified to impose translational and/or rotational restraint. Any desired displacement values may be specified at supports.

In order to obtain the total pier bent stiffness matrix the fiber model used in program PIER is also used in program FPIER. Each member is divided into a series of cross sections and each

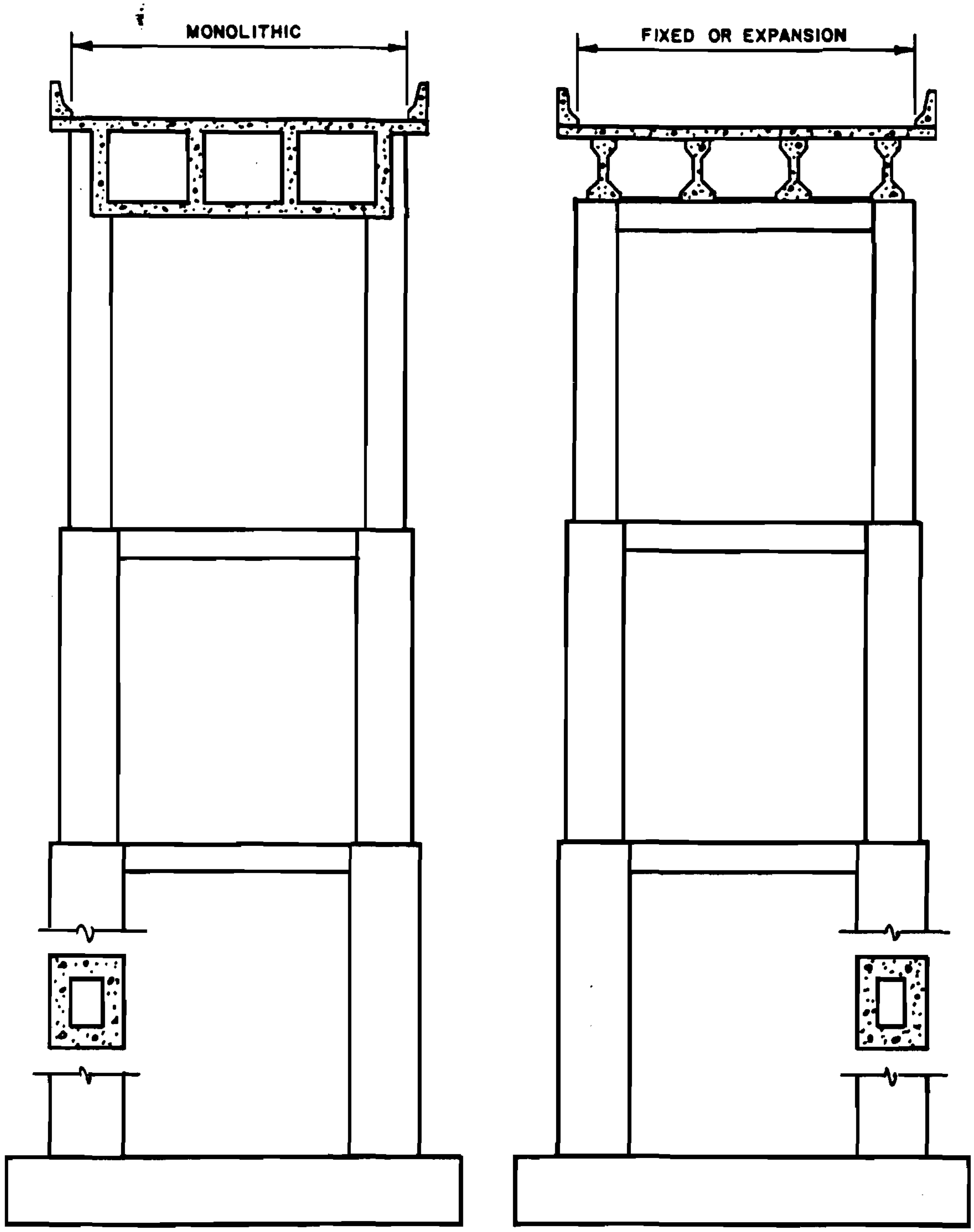


Fig. 5.34 Multistory bent (voided rectangular section)

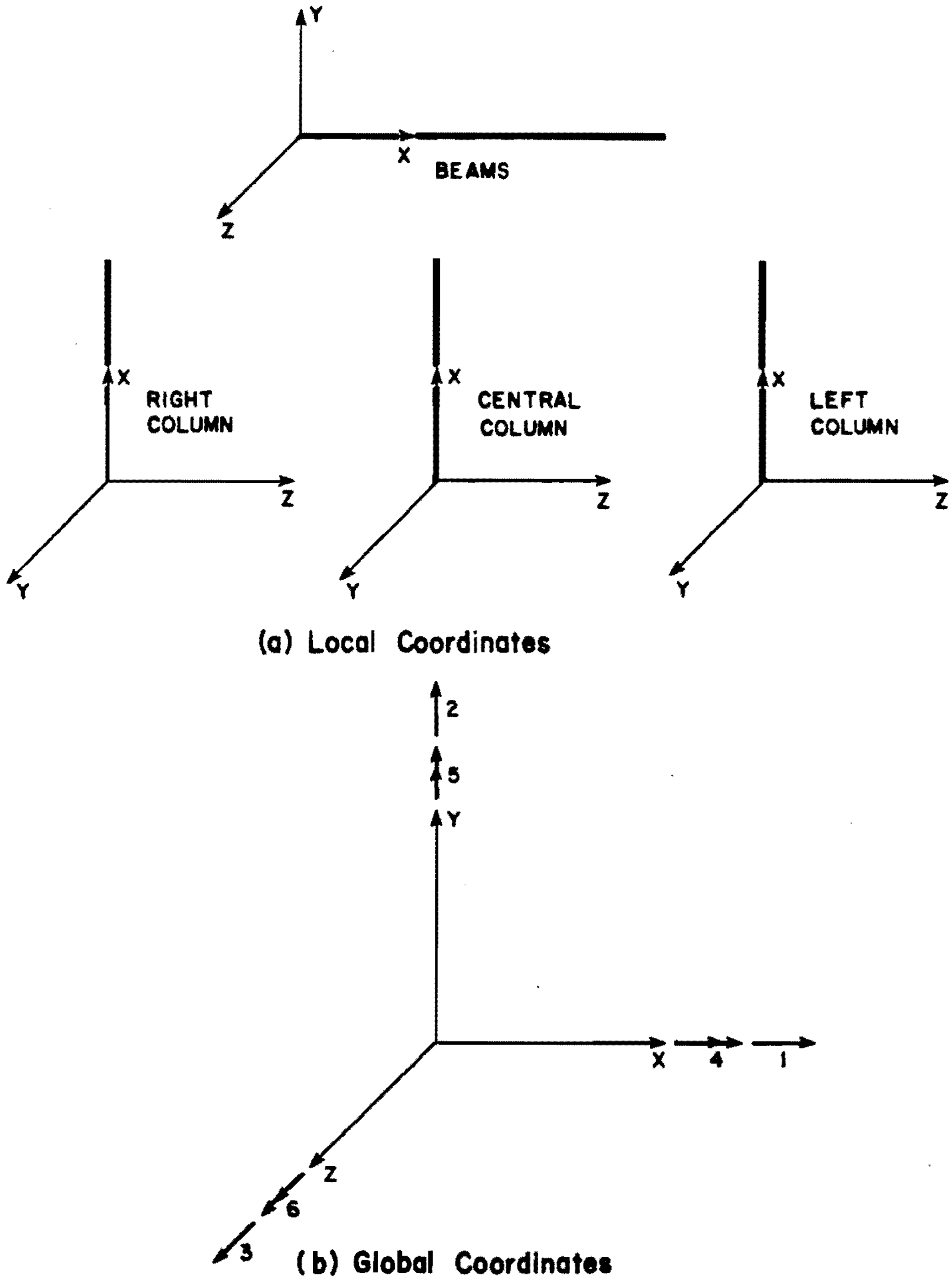


Fig. 5.35 FPIER coordinate system

cross section into fibers. The stresses are monitored at every fiber of each section for each member. In the first step, the calculation of flexibility coefficients from the fiber dimensions and stress state of each cross section is performed. The integration over the cross section is accomplished by summing each fiber of a section.

Once the flexibility coefficients for each cross section have been computed the segment flexibility matrix is determined by summing the appropriate flexibility term for each section over all the sections in the segment. The segment flexibility matrix is then inverted to get the segment stiffness matrix. The segment stiffness matrix is used to assemble the member stiffness matrix which is then rotated into global coordinates and added to the pier-bent stiffness matrix.

Knowing the new joint displacements and forces computed by the stiffness method, the inverse of the original member force-strain assumption (see Eqs. 4.14 to 4.16) can be used to calculate the incremental strains at each section. From these strains, the state of stress, and hence the strain of each fiber, can be calculated at the intersection of each section and each fiber. This now makes it possible to recalculate the stiffness coefficients and the cycle begins again. The stiffness matrix is reassembled for every load increment and the cycle is continued until all increments of load have been applied to the pier-bent or until failure occurs.

All geometric and material nonlinearities are included in the formulation. $P\Delta$ effects are considered for the piers of the pier-bent. As in program PIER the change in geometry is handled by changing the joint coordinates of each segment of the member at every increment.

Material nonlinearity follows the same approach used in program PIER. The steel stress-strain relationship indirectly takes into account the influence of the concrete around the bars under tension. Compressive stress-strain curves may be the Hognestad curve for unconfined concrete and the Ford curve for confined concrete.

The PΔ effect in the piers of the pier-bent is treated in the same way as in program PIER. These corrections in the member stiffness matrix can cause the displacement to become negative, because of the incremental change in stiffness once the critical load is exceeded. The results then are no longer valid. To prevent the program from continuing to run beyond this point the product of the diagonal values of the stiffness is checked for a negative value. If a negative value is found, a warning statement is printed to indicate that the matrix has become negative and further calculations are no longer valid. The forces and displacements from one step before are printed as a final solution.

The behavior under sustained load is predicted as in program PIER by modifying the short-time stress-strain relationships for concrete. The modification consists of shifting the stress-strain curves by increasing the value of E_o as a function of time as given in Eqs. 4.72 through 4.74. It is also suggested [17] that the maximum compressive stress for concrete, f_c'', should be expressed as a function of t. This does not seem to be very meaningful (in Ref. 17 a variation of about 10% is shown for a period of 25 years). Therefore, it has not been considered explicitly in the program. However, the user can do so by specifying the appropriate value of the reduction factor (k).

CHAPTER 6

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

6.1 Summary of the Study

The objective of this research was to develop improved design and analysis techniques for a wide range of slender bridge piers.

In order to determine the current state-of-the-art, a comprehensive survey questionnaire was sent to all states as well as a representative group of consultants, railroads, and foreign jurisdictions. Results of the survey are summarized in Chapter 2. The results indicated a growing trend towards slender pier usage and a perceived need for information to simplify application of present AASHTO design procedures for slender compression members. The survey indicated a need for treating a variety of cross sections, longitudinal configurations, and both single pier bents and multiple pier bents.

The needed ranges of variables were established based on the survey results. Programs developed were required to handle circular, oval and rectangular cross sections. Circular and rectangular sections had to be able to be solid or hollow. In addition, the rectangular cross section had to be able to be cellular. The programs had to be able to handle longitudinal cross section profiles such as uniform, tapered, flared, or stepped. Program PIER was developed for analysis of single pier bents. Program FPIER was developed for analysis of multiple pier bents of up to two-bay, three-level configurations. The analysis programs met all requirements of AASHTO Specifications Article 1.5.34(A)(1) and included realistic material properties, axial load and cracking effects on stiffness, creep effects, geometric and material nonlinearities, and were verified by comparison with a wide range of experimental data as reported in a companion report (Ref. 18).

From the many available analysis models, the fiber model was deemed most appropriate for adaptation. Development of the fiber model was presented in Chapter 4. Development of programs BIMPHI, PIER, and FPIER is summarized in Chapter 5. The latter two programs are the basic analysis programs for single pier bents and multiple pier bents, respectively. Both programs have versatile load inputs so that the designer can input a single load case or can input generalized loads and have the program apply AASHTO load factors and combine the loadings for specified load groups. In order to demonstrate the practical applications and capabilities of BIMPHI and PIER, some numerical examples are considered in Chapter 5. Several example problems are solved and some typical results presented. Other cases, including a FPIER example, are given in Ref. 18.

Several other design procedures such as the reduction factor (R) method, the moment magnification (δ) method, and the stability index (Q) method are summarized in Chapter 3. Suggestions for improvement in the moment magnification procedures, including proposed changes to AASHTO Specification Article 1.5.34(B), are given in Section 3.3.2. Illustrations of possible applications of the stability index method to bridge piers are given in Section 3.3.3.

6.2 Conclusions

6.2.1 General Conclusions. The main conclusion of this research is that BIMPHI, PIER, and FPIER are useful tools for the analysis of biaxially loaded bridge piers. BIMPHI is used for studying section behavior. PIER is used for analyzing single pier behavior. FPIER is used for analyzing multiple pier bent behavior. Program BIMPHI allows the user to compute the theoretical biaxial moment-curvature relationships of reinforced concrete sections of various shapes and reinforcement patterns. Program PIER allows the user to predict the space behavior of a single reinforced concrete pier of varying longitudinal section subject to arbitrary loading and restraint conditions. Program FPIER does the same thing for multiple pier bents. Both PIER and FPIER can be used to study biaxially loaded piers of various cross sections and longitudinal configurations.

6.2.2 Specific Conclusions Regarding Accuracy. The analytical models used agreed well with data from previous physical tests as reported in detail in Ref. 18.

BIMPHI was checked against several uniaxial bending cases for both confined and unconfined concrete and one biaxial bending case. The sections had various levels of constant axial load. The results from these comparisons were excellent. BIMPHI closely predicted the experimental results in each case.

PIER was checked with both a uniaxial bending case and several biaxial bending cases. The analytical predictions were very close to the observed experimental data in each case.

FPIER was checked against a number of frame results. The analytical predictions were in good agreement with experimental data up to the development of full plastic hinging.

For the cases investigated, the Hognestad stress-strain curve for unconfined concrete and the Ford stress-strain curve for confined concrete seem appropriate. A maximum compressive stress of $0.85f'_c$ for vertically cast members and $0.95f'_c$ for horizontally cast members appears reasonable. Also, the approach accounting for the cracking of concrete seems reasonable.

There was no possible way to check all the cases the programs will handle because of limited experimental data. BIMPHI was checked for both solid and hollow rectangular sections. PIER was checked for a solid rectangular prismatic cantilever subject to uniaxial bending, and several pin-ended, solid, rectangular, and oval prismatic columns subject to biaxial bending. The biaxial loading produced no moment gradient along the column and was for only a few skewed loading angles. FPIER was checked for solid rectangular sections only. The many capabilities of the programs could not be verified because no experimental data exist.

6.3 Recommendations

6.3.1 Use of BIMPHI, PIER and FPIER. There is a definite disadvantage accompanying the neatly printed output from a "canned" computer program. All too frequently the user unconsciously accepts whatever is printed as unquestionable fact. The strongest recommendation the authors can make to a potential user is to avoid this pitfall by using engineering judgment to constantly ensure that the computed results appear reasonable. The user is advised to carefully scrutinize the echo print of the input data to ensure that the input resulted in a model of the structure intended. In order to minimize input errors, a sketch of the pier and loading would be valuable. In order to properly interpret the computed results, the user is urged to become familiar with all the assumptions upon which the analysis is based.

6.3.2 Improvements to BIMPHI, PIER and FPIER. BIMPHI should not need much refinement. At present it handles the very general cases associated with biaxial moment-curvature relationships of a section. Possible improvement would be expansion to include other sections not presently treated such as cruciform, L or T shapes. These are relatively simple programming changes.

At present PIER is a general analysis tool for the single column reinforced concrete bridge pier. Possible improvements would be the capability of handling prestressed or partially prestressed concrete, as well as pier cross sections other than those considered. There have been few prestressed piers built but more can be expected in the future. In addition, feedback from practical usage should suggest that much can be done to make PIER into an even more comprehensive design-oriented package. Similar improvements could be prepared for FPIER.

6.3.3 Research Needs. For the range of variables examined, the analytical models seem appropriate. Additional tests on various cross sections and loadings which occur in bridge piers would be useful to fully validate the analytical model.

6.5 Concluding Remarks

It should be recognized that the examples and verifications presented do not fully demonstrate the versatility of the BIMPHI, PIER and FPIER programs. The examples presented are limited to simple loading and support conditions to demonstrate the application of the programs without unnecessary complications. As with other analytical techniques, the range of problems to which these programs can be applied depends partially upon the user's familiarity with the technique and his ingenuity.

A P P E N D I C E S

A P P E N D I X   A

PROGRAM BIMPHI

DMX	=	incremental moment about x-axis
DMY	=	incremental moment about y-axis
DN	=	incremental axial load
DPHIX	=	incremental rotational strain about x-axis
DPHIY	=	incremental rotational strain about y-axis
DU(3)	=	contains either calculated forces or displacements
EC	=	initial modulus of concrete
ECON	=	tangent modulus of a concrete fiber for a given strain
ENO	=	strain corresponding to maximum stress on concrete compression stress-strain curve
ESTL	=	tangent modulus of a steel fiber for a given strain
EU	=	ultimate strain of concrete
EY	=	steel modulus
EZC(200)	=	strain of concrete fibers
EZS(200)	=	strain of steel fibers
FY	=	yield stress of steel
GAMMAX(3)	=	x-distance from side edge of section to centroid of side steel
GAMMAY(3)	=	y-distance from end edge of section to centroid of top or bottom steel
ICODE	=	code to indicate if hollow section has interior walls
ICON	=	code to define if concrete is unconfined or confined
INO	=	code to indicate if there are interior walls in hollow section
IOR	=	orientation of interior walls, either horizontal or vertical
ISTAT	=	code to indicate if program will generate steel fibers or if the user will read in individual reinforcement properties
ITITLE(20)	=	title of problem
ITYPE	=	type of section (solid, hollow, oval or circular hollow)

JR(3)	= joint release code
NFIB	= number of concrete fibers
NRING	= number of rings of steel
NSTOT	= number of steel fibers
NW	= number of interior walls of a hollow section
PDIS(3)	= contains specified displacements of section
PFOR(3)	= contains specified forces of section
RATIO	= ratio of reinforcement area to section area
RK	= reduction factor applied to f'_c
RMX	= summation of moments about x-axis of section
RMY	= summation of moments about y-axis of section
SA11, SA12, . . . SA33	= stiffness parameters for correction procedure
STF(3,3)	= auxiliary matrix
STK(3,3)	= contains stiffness parameters of section
STLA(200)	= area of steel fibers
STLX(200)	= x-centroid of steel fibers
STLY(200)	= y-centriod of steel fibers
STR	= stress on a concrete fiber
STRANO	= any initial axial strain on section
STS(3)	= area of steel of one side of section
STT(3)	= area of top or bottom steel of section
STTR	= stress on a steel fiber
SUM	= dummy variable to add forces
TF	= flange thickness of hollow or cellular section
TW	= web thickness of hollow or cellular section
TWALL(5)	= thickness of interior walls
WLSTL(5)	= area of steel in interior walls
XCURV	= total curvature about x-axis of section

XI = width of cross section (dimension along x-axis)

XIX = moment of inertia of section about x-axis

YCURV = total curvature about y-axis of section

YI = depth of cross section (dimension along y-axis)

YIY = moment of inertia of section about y-axis

A.3 Guide to Input Data

The program data are input by means of data cards. No units are implied. The user must use consistent units. See Figure A.1 for section descriptions. F-fields are for real numbers and data can be placed anywhere within the field. A decimal point must be used. I-fields are for integers and are right justified. A-fields are for alphanumeric designations and are left justified.

1. Title Card (20A4)

   Column 1-80--ITITLE(20), title of the problem. Any FORTRAN characters are acceptable.

2. Material Properties (8F10.0)

   Column 1-10--CYL, compressive cylinder strength of concrete

   Column 11-20--FY, yield stress of reinforcing steel

   Column 21-30--RK, strength reduction factor for concrete (i.e., $f_c'' = 0.85f_c'$; see Sec. 4.5.3)

   Column 31-40--EC, initial modulus of concrete

   Column 41-50--EY, modulus of reinforcing steel

   Column 51-60--EU, ultimate strength of concrete (if left blank program assumes EU = 0.0038)

3. Section Card (2A4,2X,3I10)

   Column 1- 8--ITYPE, type of section (type either SOLID, HOLLOW, OVAL, or CIRHOLLO [circular hollow section], starting in column 1 for type of section desired) (see Fig. A.1)

   Column 11-20--ICON, status of concrete (if ICON = 0, unconfined concrete; if ICON = 1, confined concrete) To use confined concrete option, user must ensure enough lateral ties are provided to meet the equivalent of ACI Building Code 318-83 Appendix A Sec. 4.4.4 for compression members.

   Column 21-30--ISTAT, status of reinforcing steel (if ISTAT = 0, steel fibers generated by the program; if ISTAT =1, steel fibers read in by the user)

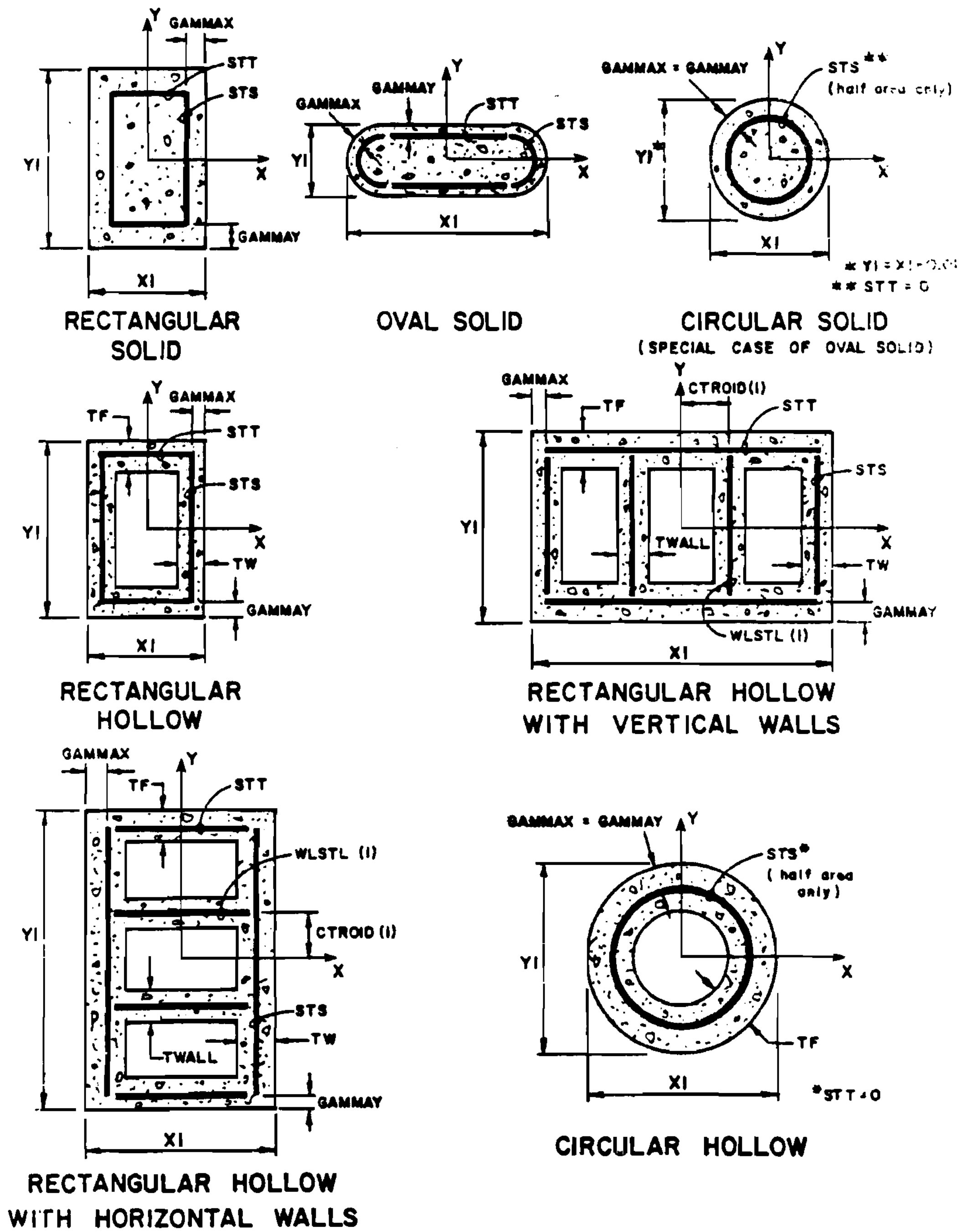


Fig. A.1 Section characteristics

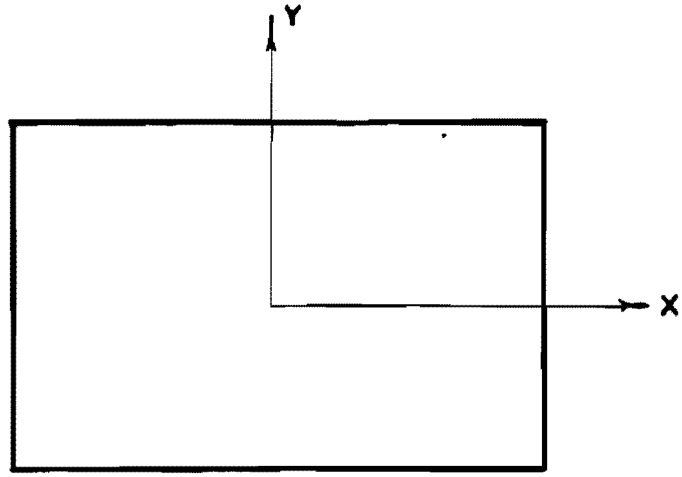


Fig. A.2a Positive axes of section

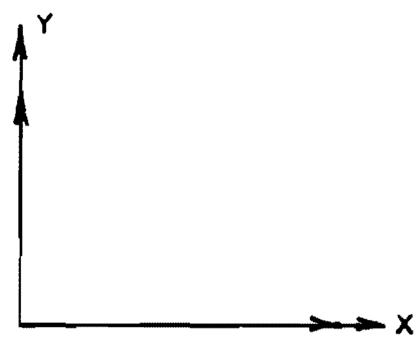


Fig. A.2b Positive curvatures and moments given by right-hand rule

4. Geometry of Section (8F10.0)

(a) If ITYPE = SOLID (rectangular solid)

Column 1-10--XI, width of section (dimension along x-axis)

Column 21-30--YI, depth of section (dimension along y-axis)

(b) If ITYPE = HOLLOW (rectangular hollow with or without interior walls)

Column 1-10--XI, width of section (exterior dimension along x-axis)

Column 11-20--TF, thickness of exterior wall which is parallel to the x-axis

Column 21-30--YI, depth of section (exterior dimension along y-axis)

Column 31-40--TW, thickness of exterior wall which is parallel to the y-axis

4.b.1 Status of Interior Walls of Hollow Section (2A4,2X,3I10)

Column 1-40--ICODE (Are there any interior walls? Type YES or NO beginning in Column 1)

If ICODE = YES

4.b.2 Description of Interior Walls (3A4,8X,I10)

Column 1-12--IOR, orientation of walls (type VERTICAL if walls are parallel to y-axis or HORIZONTAL if walls are parallel to x-axis beginning in column 1)

Column 21-30--NW, number of interior walls (maximum of 5)

If ICODE = YES

4.b.3 Geometry of Interior Walls (8F10.0)

One or more data cards alternately giving values for pairs of CTROID(I), distance from centroid of section to centroid of interior wall, and TWALL(I), thickness of wall. Pairs repeat for I = 1, NW (one data card can contain pairs of CTROID(I) and TWALL(I) for 4 interior walls. Therefore in case there is a fifth interior wall an additional card is required.

(c) If ITYPE = OVAL (also used for solid circular*)

Column 1-10--XI, width of section (dimension along x-axis)

Column 11-20--YI, depth of section (dimension along y-axis)

*The solid circular section special case of the oval section with YI = diameter and XI = YI + 0.01

(d) If ITYPE = CIRHOLLO (circular hollow)

Column 1-10--XI, exterior diameter of section

Column 11-20--TF, thickness of wall section (thickness must be less than 1/2 the diameter)

5. <u>Reinforcing Steel Description</u>

(a) If ISTAT = 0

5.a.1 <u>Description of Steel Rings (I10,7F10.0)</u>

Column 1-10--NRING, number of rings of steel (at least 1 ring, maximum of 3 rings). The rings are symmetric.

Column 11-70--Starting with STT(I), alternate STT(I), area of end steel in ring (parallel to x-axis), with STS(I), area of side steel in ring (parallel to y-axis), in fields of F10.0 for I=1 to NRING (for solid circular and hollow circular sections, STT =0, and STS is half the total area of steel)

5.a.2 <u>Concrete Cover (8F10.0)</u>

Column 1-60--Starting with GAMMAX(I), alternate GAMMAX(I), distance (parallel to x-axis) from side edge of section to center of side steel, with GAMMAY(I), distance (parallel to y-axis) from end edge of section to center of end steel, in fields of F10.0 for I = 1 to NRING

5.a.3 <u>Steel in Interior Walls (8F10.0)</u>

Only if ITYPE = HOLLOW and ICODE = YES

Column 1-50--WLSTL(I), area of steel in each interior wall in fields of F10.0 for I = 1 to NW

(b) If ISTAT = 1

5.b.1 Steel Fibers (I10)

Column 1-10--NSTOT, number of reinforcing bars to be read (maximum of 200)

5.b.2 Description of Reinforcing Bars (2(3F10.2))

Column 1-60--Starting with STLX(I), alternate three dimension sets of STLX(I) (x-distance from centroid of section to reinforcing bar) with STLY(I) (y-distance from centroid of section to reinforcing bar) and STLA(I) (area of reinforcing bar) in fields of F10.0 for I = 1 to NSTOT (A set of three numerical values (STLX, STLY, and STLA) describes a single bar. There are 2 bar descriptions per card)

6. Initial Conditions Card (8F10.0)

Column 1-10--AXIAL, initial axial load (positive if tensile)

Column 11-20--STRANO, any other initial axial strain on section (i.e., preset strain)

7. Joint Release Code (3I1)

Column 1--JR(1),

0 to specify an incremental axial load

1 to specify an incremental axial strain

Column 2--JR(2),

0 to specify an incremental moment about the x-axis

1 to specify an incremental curvature about the x-axis

Column 3--JR(3),

0 to specify an incremental moment about the y-axis

1 to specify an incremental curvature about the y-axis

8. Specified Forces (8F10.0)

If a degree of freedom had a 1 specified in the release code (step 7), the specified force for that degree of freedom should be 0 or left blank.

Column 1-10--PFOR(1), specified incremental axial load

Column 11-20--PFOR(2), specified incremental moment about the x-axis

Column 21-30--PFOR(3), specified incremental moment about the y-axis

9. Specified Displacements (8F10.0)

If a degree of freedom had a 0 specified in the release code, the specified displacement for that degree of freedom should be 0 or left blank.

Column 1-10--PDIS(1), specified incremental axial strain

Column 11-20--PDIS(2), specified incremental curvature about the x-axis

Column 21-30--PDIS(3), specified incremental curvature about the y-axis

A.4 Computer Output

BIMPHI computes the complete biaxial load-moment-curvature relationship of a reinforced concrete section. The output consists of:

A.4.1 Title and echo print of all section data including fiber properties.

A.4.2 Echo print of initial conditions, release code, and incremental forces or displacements specified.

A.4.3 Printout of completed results which includes the axial load, moment about the x-axis, moment about the y-axis, curvature about the x-axis, and curvature about the y-axis for each increment until an assumed failure. The sign convention for the results is given in Fig. A.2.

A.5 Listing of BIMPHI

```
C          DATA SET T224169    AT LEVEL 005 AS OF 01/20/84
      PROGRAM BIMPHI
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      CHARACTER*4,ICHOLL,ICHOLT,ICODE,IH,IHOLL,IHOLL1,IH1,
     . INO,INO1,IOR(3),IOVAL,IOVAL1,ISOLID,ISOLI1,ITITLE(20),
     . ITYPE(2),IV,IV1
      DIMENSION CONPX(200),CONPY(200),CPA(200)
      DIMENSION STLX(200),STLY(200),STLA(400)
      DIMENSION EZC(200),EZS(200)
      DIMENSION STP(3,3),STK(3,3),JR(3),PPOR(3),PDIS(3),DU(3),DF(3)
      COMMON /L1/ CYL,RK,EC,ENC,EU
      COMMON /L2/ FY,EY,RATIO
      COMMON /ONE/ XI,TF,YI,TW
      COMMON /TWO/ CTROID(5),TWALL(5),NW,NWX,NWY
      COMMON /THREE/ IOR,INO,IH,IV
      COMMON /FOUR/ STT(3),STS(3),GAMMAX(3),GAMMAY(3),WLSTL(5),NRING
      COMMON /ZZ/ ISOLID,IHOLL,IOVAL
      EQUIVALENCE(DU(1),DEO),(DU(2),DPHIX),(DU(3),DPHIY)
      EQUIVALENCE(DF(1),DN),(DF(2),DPX),(DF(3),DMY)
      DATA INO1/'NO  '/,IH1/'HORI'/,IV1/'VERT'/,ISOLI1/'SOLI'/,
     .     IHOLL1/'HOLL'/,IOVAL1/'OVAL'/,ICHOL1/'CIRH'/
      DATA ICNF/1/,IZERO/0/
      JT=1
      CALL TICTOC (JT)
      INO=INO1
      IH=IH1
      IV=IV1
      ISOLID=ISOLI1
      IHOLL=IHOLL1
      IOVAL=IOVAL1
      ICHOLL=ICHOL1
      READ (*,1501,END=10000)(ITITLE(I),I=1,20)
 1501 FORMAT(5X,20A4)
10000 PRINT 1502,(ITITLE(I),I=1,20)
 1502 FORMAT('1',6X,20A4)
      READ (*,103,END=10001)CYL,FY,RK,EC,EY,EU
 103  FORMAT (5X,6F10.0)
10001 IF(EU.EQ.0.0) EU=.0038
      PRINT 205,CYL,FY,RK,EC,EY,EU
 205  FORMAT(///,3X,'CONCRETE STRENGTH          =',F10.2,/,
     1              3X,'STEEL YIELD POINT          =',F10.2,/,
     2              3X,'STRENGTH REDUCTION FACTOR=',F10.2,/,
     3              3X,'CONCRETE MODULUS           =',1PE12.4,/,
     4              3X,'STEEL MODULUS              =',1PE12.4,/,
     5              3X,'CONCRETE ULTIMATE STRAIN =',1PE12.4)
      READ (*,301,END=10002)ITYPE,ICON,ISTAT
 301  FORMAT(5X,2A4,4X,3I10)
10002 PRINT 207,ITYPE
 207  FORMAT(///,3X,'THE PIER SECTION IS',3X,2A4,//)
      IF(ICON.EQ.0) PRINT 3333
      IF(ICON.EQ.1) PRINT 3334
 3333 FORMAT('0',4X,'CONCRETE FIBERS WILL BE MODELLED AS',
     1' UNCONFINED CONCRETE')
 3334 FORMAT('0',4X,'CONCRETE FIBERS WILL BE MODELLED AS',
     1' CONFINED CONCRETE')
      IF(ISTAT.EQ.IZERO) PRINT 212
      IF(ISTAT.EQ.ICNF) PRINT 213
 212  FORMAT('0',4X,'STEEL POINTS ARE GENERATED BY THE PROGRAM')
 213  FORMAT('0',4X,'STEEL POINTS ARE READ IN BY THE USER')
      IF(ITYPE(1).EQ.ISOLID) GO TO 151
      IF(ITYPE(1).EQ.IHOLL) GO TO 152
      IF(ITYPE(1).EQ.IOVAL) GO TO 153
      PRINT 107
 107  FORMAT ('0',4X,'MISTAKE MADE IN INPUT OF SECTION TYPE')
      STOP 3000
 151  READ (*,103,END=10003)XI,YI
10003 PRINT 103,XI,YI
 133  FORMAT('0',3X,'X-DIMENSION=',F10.2,/,
     1              3X,'Y-DIMENSION=',F10.2)
      GO TO 402
 152  READ (*,103,END=10004)XI,TF,YI,TW
10004 PRINT 204,XI,TF,YI,TW
 204  FORMAT('0',3X,'X-DIMENSION=',F10.2,/,
     1              3X,'FLANGE THICKNESS=',F10.2,/,
     2              3X,'Y-DIMENSION=',F10.2,/,
     3              3X,'WEB THICKNESS    =',F10.2)
      READ (*,301,END=10005)ICODE
10005 IF(ICODE.EQ.INO) GO TO 402
      READ (*,304,END=10006)IOR,NW
 304  FORMAT(5X,3A4,3X,I10)
10006 READ (*,103,END=10007)(CTROID(J),TWALL(J),J=1,NW)
10007 PRINT 211,NW,IOR
 211  FORMAT(///,3X,'NUMBER OF INNER WALLS=',I5,/,
     1              3X,'WALLS ARE ORIENTED',2X,3A4)
      PRINT 440
 440  FORMAT(//,3X,'NO.',5X,'X OR Y CENTROID',4X,'THICKNESS')
      DO 405 J=1,NW
 405  PRINT 450,J,CTROID(J),TWALL(J)
 450  FORMAT(1X,I5,3X,F10.2,8X,F10.2)
 402  CONTINUE
      IF(ITYPE(1).EQ.ISOLID)CALL SOLSCR(CONPX,CONPY,
     1CPA,NFIB)
      IF(ITYPE(1).EQ.IHOLL)CALL HSCRIB(CONPX,CONPY,
     1CPA,ICODE,NFIB)
      IF(ITYPE(1).EQ.IOVAL)CALL OSCRIB(CONPX,CONPY,
     1CPA,NFIB)
      IF(ITYPE(1).EQ.ICHOLL) CALL CIRSCR(CONPX,CONPY,CPA,NFIB)
      XIX=0.0
      YIY=0.0
      DO 8001 KAY=1,NFIB
      XIX=XIX+CONPY(KAY)**2*CPA(KAY)
      YIY=YIY+CONPX(KAY)**2*CPA(KAY)
 8001 CONTINUE
      PRINT 8002,XIX,YIY
 8002 FORMAT('0',2X,'MOMENT OF INERTIA ABOUT X-AXIS=',1PE10.3,/,
     1              3X,'MOMENT OF INERTIA ABOUT Y-AXIS=',1PE10.3,/)
      IF(ISTAT.EQ.IZERO) GO TO 307
      IF(ISTAT.EQ.ICNF) GO TO 502
 307  READ (*,102,END=10008)NRING,(STT(I),STS(I),I=1,NRING)
 102  FORMAT(5X,I10,7F10.0)
10008 PRINT 705,NRING
 705  FORMAT(///,3X,'NUMBER OF STEEL RINGS=',I5,/,3X,
     1'NO.',5X,'AREA OF END STEEL',5X,'AREA OF SIDE STEEL')
      PRINT 706,(I,STT(I),STS(I),I=1,NRING)
 706  FORMAT(1X,I3,7X,F10.2,14X,F10.3)
      READ (*,103,END=10009)(GAMMAX(J),GAMMAY(J),J=1,NRING)
10009 PRINT 710
 710  FORMAT(//,3X,'NO.',8X,'X COVER',7X,'Y COVER')
      PRINT 715,(K,GAMMAX(K),GAMMAY(K),K=1,NRING)
 715  FORMAT(1X,I3,5X,F10.2,2X,F10.2)
      IF(ITYPE(1).EQ.IHOLL.AND.ICODE.NE.INO) GO TO 255
      GO TO 502
 255  READ (*,103,END=10010)(WLSTL(J),J=1,NW)
10010 PRINT 725
 725  FORMAT(///,3X,'NO.',5X,'AREA OF STEEL IN WALL')
      PRINT 750,(L,WLSTL(L),L=1,NW)
```

```
733  FORMAT(1X,I3,1X,F10.3)
504  CONTINUE
     IF(ITYPE(1).EQ.ISCLID)CALL SSCRIS(STLX,STLY,
    1STLA,ICODE,NSTOT,ITYPE(1))
     IF(ITYPE(1).EQ.INCLL) CALL SSCRIS(STLX,STLY,
    1STLA,ICODE,NSTOT,ITYPE(1))
     IF(ITYPE(1).EQ.IOVAL) CALL OSSTL(STLX,STLY,
    1STLA,NSTOT)
     IF(ITYPE(1).EQ.ICNCLL) CALL CIRSTL(STLX,STLY,STLA,NSTOT)
     GO TO 8
302  CALL RECS(STLX,STLY,STLA,NSTOT)
8    CONTINUE
     AST=0.0
     AG=0.0
     DO 82 J=1,NPIE
82   AG=AG+CPA(J)
     DO 84 J=1,NSTOT
84   AST=AST+STLA(J)
     RATIO=AST/AG
     DO 20 I=1,3
     PPOR(I)=0.
20   POIS(I)=0.
     READ (*,103,END=10011)AXIAL,STRAND
10011 PRINT 745,AXIAL,STRAND
745  FORMAT('1',3X,'BEGINNING AXIAL LOAD      =',1PE12.4,/,
    &            3X,'INITIAL PRESET STRAIN    =',1PE12.4)
     READ (*,125,END=10012)(JR(I),I=1,3)
125  FORMAT(E2,3I1)
10012 PRINT 129,(JR(I),I=1,3)
129  FORMAT('0',2X,'RELEASE CODE',/,2X,3(I1))
     READ (*,105,END=10013)(PPOR(I),I=1,3)
10013 PRINT 131,(PPOR(I),I=1,3)
131  FORMAT('0',3X,'AXIAL LOAD',4X,'MOMENT-X',4X,'MOMENT-Y',/,
    14X,1PE10.3,3X,1PE10.3,2X,1PE10.3)
     READ (*,105,END=10014)(POIS(I),I=1,3)
10014 PRINT 132,(POIS(I),I=1,3)
132  FORMAT('0',3X,'AXIAL STRAIN',4X,'CURVATURE-X',4X,'CURVATURE-Y',/,
    13X,1PE10.3,6X,1PE10.3,5X,1PE10.3)
     ENDRK=2.0*CYL/EC
     DO 40 I=1,NPIE
40   EIC(I)=0.0
     DO 50 J=1,NSTOT
50   EIS(J)=0.0
     IF(AXIAL.EQ.0.0) GO TO 721
     CALL BEGIN(AXIAL,CPA,STLA,EIC,EIS,NPIE,NSTOT,ICON)
721  PRINT 633,EIC(1)
633  FORMAT(/,3X,'INITIAL STRAIN=',1PE12.4)
     DO 60 K=1,NPIE
60   EIC(K)=EIC(K)+STRAND
     DO 62 J=1,NSTOT
62   EIS(J)=EIS(J)+STRAND
     KCT=0
     XCURV=0.0
     YCURV=0.0
     RMX=0.0
     RMY=0.0
     JAT=3
     PRINT 617
617  FORMAT ('1',4X,'N.',4X,'AXIAL LOAD',7X,'MOMENT-X',4X,'MOMENT-Y',
    15X,'CURVATURE-X',4X,'CURVATURE-Y')
775  CALL UNSTR(CC,FA,CI,FY,CFS,STLX,STLY,STLA,FCFS,NPIE,NSTOT,
    1CA11,CA12,CA13,CA21,CA22,CA23,CA31,ACD,CA32,ICON)
     KCT=KCT+1
776  STK(1,1)=CA11
     STK(1,2)=CA12
     STK(1,3)=-CA13
     STK(2,1)=CA21
     STK(2,2)=CA22
     STK(2,3)=-CA23
     STK(3,1)=-CA31
     STK(3,2)=-CA32
     STK(3,3)=CA33
     DO 21 I=1,3
     DU(I)=PPOR(I)
     DO 21 J=1,3
21   STP(I,J)=STK(I,J)
     DO 22 I=1,3
     IF(JR(I).EQ.0) GO TO 22
     DO 23 J=1,3
     DU(J)=DU(J)-STK(J,I)*POIS(I)
23   STK(J,I)=0.
     DO 24 J=1,3
24   STK(I,J)=0.
     STK(I,I)=1.
     DU(I)=POIS(I)
22   CONTINUE
     CALL FSINV(STK,3,3)
     CALL PSMULT(STK,DU,3,3,3,1)
     DO 25 I=1,3
     SUM=0.
     DO 26 J=1,3
26   SUM=SUM+STP(I,J)*DU(J)
25   DF(I)=SUM
     XCURV=XCURV+DPHIX
     YCURV=YCURV+DPHIY
     AXIAL=AXIAL+DN
     RMX=RMX+DMX
     RMY=RMY+DMY
     ANF=0.
     SMX=0.
     SMY=0.
     DO 71 I=1,NPIE
     DEI=DEO-CONFX(I)*DPHIY+CONFY(I)*DPHIX
     EIC(I)=EIC(I)+DEI
     IF(ICON.EQ.0) CALL FC(EIC(I),STR,ECON)
     IF(ICON.EQ.1) CALL FCON(EIC(I),STR,ECON)
     ANF=ANF+STR*CPA(I)
     SMX=SMX+CONFY(I)*STR*CPA(I)
     SMY=SMY-CONFX(I)*STR*CPA(I)
     IF(EIC(I).LT.-EU) GO TO 2001
71   CONTINUE
     DO 73 J=1,NSTOT
     DEI=DEO-STLX(J)*DPHIY+STLY(J)*DPHIX
     EIS(J)=EIS(J)+DEI
     IF(ICON.EQ.0) CALL FC(EIS(J),STR,ECON)
     IF(ICON.EQ.1) CALL FCON(EIS(J),STR,ECON)
     CALL FS(EIS(J),STTR,FSTL)
     STR=STTR-STR
     ANF=ANF+STR*STLA(J)
     SMX=SMX+STLY(J)*STR*STLA(J)
     SMY=SMY-STLX(J)*STR*STLA(J)
     IF(EIS(J).GT.ESU) GO TO 2001
73   CONTINUE
     IF(KAT.GT.0 ) PRINT 617
     IF(KAT.GT. ) JAT=0
```

```
      PRINT 751,ACT,AXIAL,RMX,RMY,XCURV,YCURV
      PRINT 752,ACT,AXP,DMX,DMY
  751 FORMAT('0',I7,1P,5E13.5)
  752 FORMAT(1X,1P,5E15.5)
      JAY=JAY+1
      IF(MARY.EQ.2) AXIAL=AXP
      IF(MARY.EQ.2) GO TO 775
      IF(AXP.EQ.0.0.AND.AXIAL.EQ.0.0) GO TO 775
      DEN=AXP
      IF(DEN.EQ.0.0) DEN=AXIAL
      ERR=ABS(AXP-AXIAL)/ABS(DEN)
      IF(ERR.LT.0.05) GO TO 775
      CALL CNSTP(CONFX,CONFY,CPA,STLX,STLY,STLA,EIC,EIS,NFIB,NSTOT,
     1SA11,SA12,SA13,SA21,SA22,SA23,SA31,SA32,SA33,ICON)
      MARY=2
      DA11=0.5*(DA11+SA11)
      DA12=0.5*(DA12+SA12)
      DA13=0.5*(DA13+SA13)
      DA21=0.5*(DA21+SA21)
      DA22=0.5*(DA22+SA22)
      DA23=0.5*(DA23+SA23)
      DA31=0.5*(DA31+SA31)
      DA32=0.5*(DA32+SA32)
      DA33=0.5*(DA33+SA33)
      XCURV=XCURV-DPHIX
      YCURV=YCURV-DPHIY
      AXIAL=AXIAL-DN
      RMX=RMX-DMX
      RMY=RMY-DMY
      DO 74 I=1,NFIB
      DEI=DEO-CONFX(I)*DPHIY+CONFY(I)*DPHIX
   74 EIC(I)=EIC(I)-DEI
      DO 75 I=1,NSTOT
      DEI=DEO-STLX(I)*DPHIY+STLY(I)*DPHIX
   75 EIS(I)=EIS(I)-DEI
      GO TO 776
 2001 CONTINUE
      STOP
      END
C*PASV1PO*
      SUBROUTINE DSCRIS(CONFX,CONFY,CPA,ICODE,NFIB)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      CHARACTER*4,ICODE,IOR(2),INC,IH,IV
      DIMENSION CONFX(200),CONFY(200),CPA(200)
      COMMON /ONE/ XI,TF,YI,TW
      COMMON /TWO/ CTROID(5),THALL(5),KW,KWX,KWY
      COMMON /THREE/ IOR,INC,IH,IV
      REAL INNERX,INNERY
      IXI=XI
      IYI=YI
      KFX=2
      KWY=2
      FACT=16.
      IF(ICODE.NE.INC) FACT=16.*KW*10.
      XKFY=FACT/(4.*(1.+YI/XI))
      KFY=XKFY
      XKWX=YI/XI*XKFY
      KWX=XKWX
      XFINC=(XI-2.0*TW)/KFY
      YFINC=TF/KFX
      XWINC=TW/KWY
      YWINC=(YI-2.0*TF)/KWX
      NFIB=2*KFX*KFY+2*KWX*KWY+2*KFX*KWY
      INNERX=XI-2.0*TW
      INNERY=YI-2.0*TF
      MIX=INNERX/2.0
      MIY=INNERY/2.0
      STATX=-XI/2.0+XWINC/2.0
      STATY=YI/2.0-YFINC/2.0
      KC=1
      Y=STATY
      IT=0
   98 DO 10 I=1,KFX
      X=STATX
      DO 20 J=1,KWY-1
      CONFX(KC)=X
      CONFY(KC)=Y
      CPA(KC)=XWINC*YFINC
      X=X+XWINC
   20 KC=KC+1
      CONFX(KC)=X
      CONFY(KC)=Y
      CPA(KC)=XWINC*YFINC
      KC=KC+1
      X=X+XFINC/2.0+XWINC/2.0
      CONFX(KC)=X
      CONFY(KC)=Y
      CPA(KC)=XFINC*YFINC
      DO 30 K=1,KFY-1
      X=X+XFINC
      KC=KC+1
      CPA(KC)=XFINC*YFINC
      CONFX(KC)=X
   30 CONFY(KC)=Y
      X=X+XFINC/2.0+XWINC/2.0
      KC=KC+1
      CONFX(KC)=X
      CONFY(KC)=Y
      CPA(KC)=XWINC*YFINC
      KC=KC+1
      DO 40 JK=1,KWY-1
      X=X+XWINC
      CONFX(KC)=X
      CONFY(KC)=Y
      CPA(KC)=XWINC*YFINC
   40 KC=KC+1
   10 Y=Y-YFINC
      IF(IT.NE.0) GO TO 99
      Y=Y+YFINC/2.0-YWINC/2.0
      DO 50 L=1,KWX
      X=STATX
      DO 60 M=1,KWY-1
      CONFX(KC)=X
      CONFY(KC)=Y
      CPA(KC)=XWINC*YWINC
      X=X+XWINC
   60 KC=KC+1
      CONFX(KC)=X
      CONFY(KC)=Y
      CPA(KC)=XWINC*YWINC
      KC=KC+1
      X=X+INNERX+XWINC
      CONFX(KC)=X
      CONFY(KC)=Y
      CPA(KC)=XWINC*YWINC
      KC=KC+1
```

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      DO 70 N=1,NWY-1
      X=X+XINC
      CONFX(KC)=X
      CONFY(KC)=Y
      CPA(KC)=XINC*YINC
 70   KC=KC+1
 50   Y=Y-YINC
      Y=Y+YINC
      Y=Y-YINC/2.0-YINC/2.0
      IT=1
      GO TO 93
 99   CONTINUE
      NFF=KFX+KFY+KPX+KWY+2
      NWP=KWX+KWY
      PRINT 100
100   FORMAT (///,10X,'CONCRETE FIBER PROPERTIES',
     1//,3X,'NO.',8X,'X',12X,'Y',12X,'AREA')
      DO 110 NO=1,NFIB
      PRINT 102,NO,CONFX(NO),CONFY(NO),CPA(NO)
102   FORMAT (I5,3X,F10.2,2X,F10.2,3X,F10.2)
110   CONTINUE
      IF(ICODE.EQ.INO) GO TO 199
      IF(IOR(1).EQ.IH) GO TO 130
      IF(IOR(1).EQ.IV) GO TO 140
130   NWX=2
      NWY=5
      WALLX=INNERX/NWT
      ANWX=NWX
      DO 150 L=1,NW
      WALLY=TWALL(L)/ANWX
      TSTWL=CTRDIO(L)+(ANWX-1.0)*.5*WALLY
      WY=YSTWL+WALLY
      DO 160 K=1,NWX
      WY=WY-WALLY
      WX=-XI/2.0+TW-WALLX/2.0
      DO 170 M=1,NWT
      WX=WX+WALLX
      CONFX(KC)=WX
      CONFY(KC)=WY
      CPA(KC)=WALLX*WALLY
170   KC=KC+1
160   CONTINUE
150   CONTINUE
      NWLFIB=NW*NWX*NWY
      INDEX=NFIB+1
      NFIB=NFIB+NWLFIB
      DO 150 I=INDEX,NFIB
      PRINT 102,I,CONFX(I),CONFY(I),CPA(I)
150   CONTINUE
      GO TO 199
140   NWX=5
      NWY=2
      ANWY=NWY
      ANWX=NWX
      WALLY=INNERY/ANWX
      DO 210 L=1,NW
      WALLX=TWALL(L)/ANWY
      XSTWL=CTRDIO(L)-(ANWY-1.0)*.5*WALLX
      WX=XSTWL-WALLX
      DO 220 K=1,NWY
      WX=WX+WALLX
      WY=YI/2.0-TW+WALLY/2.0
      DO 230 M=1,NWX
      WY=WY-WALLY
      CONFX(KC)=WX
      CONFY(KC)=WY
      CPA(KC)=WALLX*WALLY
230   KC=KC+1
220   CONTINUE
210   CONTINUE
      NWLFIB=NW*NWX*NWY
      INDEX=NFIB+1
      NFIB=NFIB+NWLFIB
      DO 250 I=INDEX,NFIB
      PRINT 102,I,CONFX(I),CONFY(I),CPA(I)
250   CONTINUE
199   CONTINUE
      RETURN
      END
C=PASV1PC=
      SUBROUTINE SSCRIB(STLX,STLY,STLA,ICODE,NSTOT,ITYP)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      CHARACTER*4,ICODE,ITYP,ISCLID,IHOLL,IOVAL,IOR(3),INO,IH,IV
      DIMENSION STLX(200),STLY(200),STLA(200)
      COMMON /ONE/ XI,TF,YI,TW
      COMMON /TWO/ CTRDIO(5),TWALL(5),NW,NWX,NWY
      COMMON /THREE/ IOR,INO,IH,IV
      COMMON /FOUR/ STT(3),STS(3),GAMMAX(3),GAMMAY(3),WLSTL(5),NRING
      COMMON /ZE/ ISCLID,IHOLL,IOVAL
      REAL INNERX,INNERY
      NWX=10
      NWY=10
      KC=1
      VAL=200./NRING
      IF(ITYP.EQ.IHOLL.AND.ICODE.NE.INO) VAL=(200.-NW*10.)/NRING
      IVAL=VAL
      NNX=VAL/(2.*(1.+YI/XI))
      NX=NNX
      NY=YI/XI*NX
      NSTOT=(2*(NX+NY))*NRING
      DO 10 I=1,NRING
      GX=XI-2.0*GAMMAX(I)
      GY=YI-2.0*GAMMAY(I)
      NT=STT(I)/GX
      NS=STS(I)/GY
      XINC=GX/NX
      YINC=GY/NY
      STRTX=-XI/2.0+GAMMAX(I)
      STRTY=YI/2.0-GAMMAY(I)
      X=STRTX-XINC/2.0
      Y=STRTY
      DO 20 J=1,NX
      X=X+XINC
      STLX(KC)=X
      STLY(KC)=Y
      STLA(KC)=XINC*NT
 20   KC=KC+1
      X=STRTX-XINC/2.0
      Y=-STRTY
      DO 30 K=1,NX
      X=X+XINC
      STLX(KC)=X
      STLY(KC)=Y
      STLA(KC)=XINC*NT
 30   KC=KC+1
      X=STRTX
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      Y=STRTY+YINC/2.0
      DO 40 L=1,NY
      Y=Y-YINC
      STLX(KC)=X
      STLY(KC)=Y
      STLA(KC)=YINC*HS
40    KC=KC+1
      X=-STRTX
      Y=STRTY+YINC/2.0
      DO 50 M=1,NY
      Y=Y-YINC
      STLX(KC)=X
      STLY(KC)=Y
      STLA(KC)=YINC*HS
50    KC=KC+1
10    CONTINUE
      PRINT 100
100   FORMAT (///,10X,'STEEL FIBER PROPERTIES'
     1//,3X,'NO.',6X,'X',13X,'Y',10X,'AREA')
      PRINT 110,(K,STLX(K),STLY(K),STLA(K),K=1,NSTOT)
110   FORMAT (I5,3X,F10.2,2X,F10.2,3X,F10.3)
      IF(ITYP.EQ.ISCLD) GO TO 199
      IF(ICODE.EQ.INO) GO TO 199
      IF(IOR(1).EQ.IH) GO TO 130
      IF(IOR(1).EQ.IV) GO TO 140
130   INNERX=XI-2.0*TW
      WALLX=INNERX/NWY
      DO 150 L=1,NW
      NW=WLSTL(L)/INNERX
      YSTWL=CTROID(L)
      WX=-XI/2.0+TW-WALLX/2.0
      DO 160 K=1,NWY
      WX=WX+WALLX
      STLX(KC)=WX
      STLY(KC)=YSTWL
      STLA(KC)=WW*WALLX
160   KC=KC+1
150   CONTINUE
      INDEX=NSTOT+1
      NSTOT=NSTOT+NW*NWY
      PRINT 110,(I,STLX(I),STLY(I),STLA(I),I=INDEX,NSTOT)
      GO TO 199
140   INNERY=YI-2.0*TF
      WALLY=INNERY/NWX
      DO 170 K=1,NW
      WW=WLSTL(K)/INNERY
      XSTWL=CTROID(K)
      WY=YI/2.0-TF+WALLY/2.0
      DO 180 M=1,NWX
      WY=WY-WALLY
      STLX(KC)=XSTWL
      STLY(KC)=WY
      STLA(KC)=WW*WALLY
180   KC=KC+1
170   CONTINUE
      INDEX=NSTOT+1
      NSTOT=NSTOT+NW*NWX
      PRINT 110,(I,STLX(I),STLY(I),STLA(I),I=INDEX,NSTOT)
199   CONTINUE
      RETURN
      END
C*F45V1P0*
      SUBROUTINE SOLSCR(CONFZ,CONFX,CFA,NFIS)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      DIMENSION CONFZ(200),CONFX(200),CFA(200)
      COMMON /ONE/ ZI,TF,XI,TW
      FACTOR=SQRT(200./(XI/ZI))
      KV=FACTOR
      KH=XI/ZI*KV
      NFIS=KV*KH
      IF(NFIS.GT.200) STOP 555
      HINC=ZI/KV
      VINC=XI/KH
      HSTRT=-ZI/2.0+HINC/2.0
      VSTRT=XI/2.0-VINC/2.0
      KC=1
      V=VSTRT
      DO 10 I=1,KH
      H=HSTRT
      DO 20 J=1,KV
      CONFZ(KC)=H
      CONFX(KC)=V
      CFA(KC)=HINC*VINC
      H=H+HINC
20    KC=KC+1
10    V=V-VINC
      PRINT 100
100   FORMAT(///,10X,'CONCRETE FIBER PROPERTIES',
     1//,3X,'NO.',6X,'X',12X,'Y',10X,'AREA')
      DO 110 M=1,NFIS
      PRINT 102,M,CONFZ(M),CONFX(M),CFA(M)
102   FORMAT(I5,3(3X,F10.2))
110   CONTINUE
      RETURN
      END
C*F45V1P0*
      SUBROUTINE DSCRIS(CONFZ,CONFX,CFA,NFIS)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      DIMENSION CONFZ(200),CONFX(200),CFA(200)
      COMMON /ONE/ ZI,TF,XI,TW
      SI=ZI-XI
      IF(SI.LE.0.) GO TO 15
      UI=XI
      VI=ZI
      GO TO 35
15    UI=ZI
      VI=XI
35    SI=ABS(SI)
      IF(SI.LE.1.0) GO TO 50
      IF(KCTYPE.GT.0) GO TO 25
      FACTOR=SQRT(125./(UI/SI))
      KV=FACTOR
      KH=UI/SI*KV
25    NFIS=KV*KH
      HINC=SI/KV
      VINC=UI/KH
      HSTRT=-SI/2.0+HINC/2.0
      VSTRT=UI/2.0-VINC/2.0
      KC=1
      V=VSTRT
      DO 10 I=1,KH
      H=HSTRT
      DO 20 J=1,KV
      CONFZ(KC)=H
      CONFX(KC)=V
      CFA(KC)=HINC*VINC
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      H=H+HINC
2C    KC=KC+1
10    V=V-VINC
      RADIUS=UI/2.0
      PI=3.1415927
      H=1.0/6.0*RADIUS
      ANG=75.0*PI/180.
      C1=.5*30.*PI/180.*H
      ALPHA=15.*PI/180.
      CON=2.*SIN(ALPHA)/(3.*ALPHA)
      DO 50 I=1,3
      R1=RADIUS
      R2=5./6.*RADIUS
      DO 40 J=1,6
      A=C1*(R1+R2)
      ONE=CON*R1*ALPHA*R1**2
      TWO=CON*R2*ALPHA*R2**2
      RAD=(ONE-TWO)/A
      Z=VI/2.0-RADIUS+RAD*COS(ANG)
      X=RAD*SIN(ANG)
      CONFZ(KC)=Z
      CONFX(KC)=X
      CPA(KC)=A
      KC=KC+1
      CONFZ(KC)=-Z
      CONFX(KC)=X
      CPA(KC)=A
      KC=KC+1
      CONFZ(KC)=-Z
      CONFX(KC)=-X
      CPA(KC)=A
      KC=KC+1
      CONFZ(KC)=Z
      CONFX(KC)=-X
      CPA(KC)=A
      KC=KC+1
      R1=R1-H
40    R2=R2-H
30    ANG=ANG-30.*PI/180.
      NFIB=NFIB+72
      GO TO 90
50    KC=1
      RADIUS=UI/2.0
      PI=3.1415927
      H=1.0/8.0*RADIUS
      ANG=82.5*PI/180.
      C1=.5*15.*PI/180.*H
      ALPHA=7.5*PI/180.
      CON=2.*SIN(ALPHA)/(3.*ALPHA)
      DO 70 I=1,6
      R1=RADIUS
      R2=7./8.*RADIUS
      DO 60 J=1,5
      A=C1*(R1+R2)
      ONE=CON*R1*ALPHA*R1**2
      TWO=CON*R2*ALPHA*R2**2
      RAD=(ONE-TWO)/A
      Z=RAD*COS(ANG)
      X=RAD*SIN(ANG)
      CONFZ(KC)=Z
      CONFX(KC)=X
      CPA(KC)=A
      KC=KC+1
      CONFZ(KC)=-Z
      CONFX(KC)=X
      CPA(KC)=A
      KC=KC+1
      CONFZ(KC)=-Z
      CONFX(KC)=-X
      CPA(KC)=A
      KC=KC+1
      CONFZ(KC)=Z
      CONFX(KC)=-X
      CPA(KC)=A
      KC=KC+1
      R1=R1-H
60    R2=R2-H
70    ANG=ANG-15.*PI/180.
      NFIB=192
   90 IF(NFIB.GT.200) STOP 560
      IF(IPRNT.EQ.0) GO TO 113
      PRINT 100
100   FORMAT(///,10X,'CONCRETE FIBER PROPERTIES',
     1//,3X,'NO.',5X,'Z',12X,'X',12X,'AREA')
      DO 110 M=1,NFIB
      PRINT 102,M,CONFZ(M),CONFX(M),CPA(M)
102   FORMAT(I5,3(3X,F10.2))
110   CONTINUE
113   CONTINUE
      KCTYPE=KCTYPE+1
      RETURN
      END
C*PASYIPG*
      SUBROUTINE DSSTL(STLZ,STLX,STLA,NSTOT)
      DIMENSION STLZ(200),STLX(200),STLA(200)
      COMMON /ONE/ ZZ,TP,XI,TW
      COMMON /FCU4/ STT(3),STS(3),GAMMAZ(3),GAMMAX(3),NLSTL(5),NRING
      KC=1
      IF(KSTTPE.GT.0) GO TO 25
      VAL=200.-24.*NRING
      PI=3.141593
      ZZ=(VAL/NRING)/2.0
      NZ=ZZ
25    NSTOT=(2*NZ)*NRING+24*NRING
      DO 10 I=1,NRING
      GI=ZI-XI
      RAD=XI/2.0-GAMMAZ(I)
      GCIR=PI*RAD
      NT=STT(I)/GI
      NCIR=STS(I)/GCIR
      ZINC=GI/NZ
      CIRINC=GCIR/12.0
      STRTZ=-ZI/2.0+XI/2.0+ZINC/2.0
      STRTX=XI/2.0-GAMMAX(I)
      Z=STRTZ
      X=STRTX
      DO 20 J=1,NZ
      STLZ(KC)=Z
      STLX(KC)=X
      STLA(KC)=ZINC*NT
      KC=KC+1
      STLZ(KC)=Z
      STLX(KC)=-X
      STLA(KC)=ZINC*NT
      Z=Z+ZINC
20    KC=KC+1
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      ANG=90.0*PI/180.
      DO 30 K=1,6
      Z=ZI/2.0-XI/2.0+RAD*COS(ANG)
      X=RAD*SIN(ANG)
      A=CIRINC*HCIR
      STLZ(KC)=Z
      STLX(KC)=X
      STLA(KC)=A
      KC=KC+1
      STLZ(KC)=-Z
      STLX(KC)=X
      STLA(KC)=A
      KC=KC+1
      STLZ(KC)=-Z
      STLX(KC)=-X
      STLA(KC)=A
      KC=KC+1
      STLZ(KC)=Z
      STLX(KC)=-X
      STLA(KC)=A
      KC=KC+1
 30   ANG=ANG-15.*PI/180.
 10   CONTINUE
      IF(IPRNT.EQ.0) GO TO 111
      PRINT 100
 100  FORMAT(///,10X,'STEEL FIBER PROPRTIES'
     1//,3X,'NO.',9X,'Z',13X,'X',10X,'AREA')
      PRINT 110,(K,STLZ(K),STLX(K),STLA(K),K=1,NSTOT)
 110  FORMAT(1X,I4,3X,F10.2,2X,F10.2,3X,F10.2)
 111  CONTINUE
      KSTYPE=KSTYPE+1
      RETURN
      END
C*F45V1P0*
      SUBROUTINE CIRSCR(CONFX,CONFY,CFA,NFIB)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      DIMENSION CONFX(200),CONFY(200),CFA(200)
      COMMON /ONE/ XI,TF,YI,XF
      KC=1
      PI=3.14156
      ANG=9.0*PI/180.0
      RADIUS=XI/2.0
      H=YF/5.0
      STANG=ANG/2.0
      R1=RADIUS
      R2=RADIUS-H
      DO 10 I=1,5
      AREA=0.5*ANG*H*(R1+R2)
      C1=2.0/3.0*SIN(STANG)/STANG
      C2=R1**3-R2**3
      C3=R1**2-R2**2
      RCEN=C1*C2/C3
      TANG=STANG
      DO 20 J=1,10
      X=RCEN*COS(TANG)
      Y=RCEN*SIN(TANG)
      CFA(KC)=AREA
      CONFX(KC)=X
      CONFY(KC)=Y
      KC=KC+1
      CFA(KC)=AREA
      CONFX(KC)=X
      CONFY(KC)=-Y
      KC=KC+1
      CFA(KC)=AREA
      CONFX(KC)=-X
      CONFY(KC)=Y
      KC=KC+1
      CFA(KC)=AREA
      CONFX(KC)=-X
      CONFY(KC)=-Y
      KC=KC+1
      TANG=TANG+ANG
 20   CONTINUE
      R1=R1-H
      R2=R2-H
 10   CONTINUE
      NFIB=KC-1
      IF(NFIB.GT.200) STOP 560
      PRINT 100
 100  FORMAT(///,10X,'CONCRETE FIBER PROPERTIES',
     1//,3X,'NO.',6X,'X',12X,'Y',10X,'AREA')
      DO 110 M=1,NFIB
      PRINT 102,M,CONFX(M),CONFY(M),CFA(M)
 102  FORMAT(I5,3(3X,F10.2))
 110  CONTINUE
      RETURN
      END
C*F45V1P0*
      SUBROUTINE CIRSTL(STLX,STLY,STLA,NSTOT)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      DIMENSION STLX(200),STLY(200),STLA(200)
      COMMON /ONE/ XI,TF,YI,T4
      COMMON /FOUR/ STT(3),STS(3),GAMMAX(3),GAMMAY(3),NLSTL(5),NRING
      PI=3.14156
      VAL=200.0/NRING
      NVAL=VAL
      KVAL=NVAL/4
      ANG=(PI/2.0)/KVAL
      RADIUS=XI/2.0
      STANG=ANG/2.0
      KC=1
      DO 10 I=1,NRING
      R=RADIUS-GAMMAX(I)
      ST=0.5*STS(I)
      AREA=ST/KVAL
      RCEN=R*SIN(STANG)/(STANG)
      TANG=STANG
      DO 20 J=1,KVAL
      X=RCEN*COS(TANG)
      Y=RCEN*SIN(TANG)
      STLA(KC)=AREA
      STLX(KC)=X
      STLY(KC)=Y
      KC=KC+1
      STLA(KC)=AREA
      STLX(KC)=X
      STLY(KC)=-Y
      KC=KC+1
      STLA(KC)=AREA
      STLX(KC)=-X
      STLY(KC)=Y
      KC=KC+1
      STLA(KC)=AREA
      STLX(KC)=-X
      STLY(KC)=-Y
```

```
      KC=KC+1
      TANG=TANG+ANG
 20   CONTINUE
 10   CONTINUE
      NSTOT=KC-1
      IF(NSTOT.GT.200) STOP 570
      PRINT 100
 100  FORMAT(///,10X,'STEEL FIBER PROPERTIES',
     1//,3X,'NO.',5X,'X',13X,'Y',10X,'AREA')
      PRINT 110,(K,STLX(K),STLY(K),STLA(K),K=1,NSTOT)
 110  FORMAT(1X,I4,3X,F10.2,2X,F10.2,3X,F10.2)
      RETURN
      END
C*F45V1P0*
      SUBROUTINE READ(STLI,STLX,STLA,NSTOT)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      DIMENSION STLI(200),STLX(200),STLA(200)
      READ (*,100,END=10000)NSTOT
 100  FORMAT(8X,I10)
10000 PRINT 150,NSTOT
 150  FORMAT(///,3X,'NUMBER OF READ IN STEEL FIBERS=',I5)
      PRINT 200
 200  FORMAT(///,10X,'STEEL FIBER PROPERTIES',
     1//,3X,'NO.',8X,'X',13X,'Y',10X,'AREA')
      READ (*,500,END=10001)(STLI(I),STLX(I),STLA(I),I=1,NSTOT)
 500  FORMAT(8X,2(3F10.2))
10001 PRINT 110,(K,STLI(K),STLX(K),STLA(K),K=1,NSTOT)
 110  FORMAT(1X,I4,5X,F10.2,2X,F10.2,3X,F10.2)
      RETURN
      END
C*F45V1P0*
      SUBROUTINE BEGIN(AXIAL,CPA,STLA,EIC,EIS,NFIB,NSTOT,ICON)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      DIMENSION CPA(200),STLA(200),EIC(200),EIS(200)
      COMMON /L1/ CYL,RK,EC,ENO,EU
      COMMON /L2/ FY,EY,RATIO
      AG=0.0
      AS=0.0
      KC=0
      DO 10 I=1,NFIB
 10   AG=AG+CPA(I)
      DO 20 I=1,NSTOT
 20   AS=AS+STLA(I)
      AT=AG+(EY/EC+10.0)*AS
      SIGMA=AXIAL/AT
      EI=(EY/EC+10.0)*SIGMA/EY
      IF(ICON.EQ.0) CALL FC(EI,SC,ETC)
      IF(ICON.EQ.1) CALL FCON(EI,SC,ETC)
      CALL FS(EI,SS,ETS)
      PI=SC*(AG-AS)+SS*AS
      EF=EI
      PARE=ABS((AXIAL-PI)/AXIAL)
      IF(PARE.LT.0.005) GO TO 1000
      IF(ABS(AXIAL)-ABS(PI)) 100,1000,300
 300  PCAL=PI
      DE=EI/10.0
      EF=EI
 310  PARE=ABS((AXIAL-PCAL)/AXIAL)
      IF(PARE.LT.0.005) GO TO 1000
 315  ESAVE=EF
      KC=KC+1
      IF(KC.GT.25) STOP 965
      EF=EF+DE
      IF(ICON.EQ.0) CALL FC(EF,SCO,ECON)
      IF(ICON.EQ.1) CALL FCON(EF,SCO,ECON)
      CALL FS(EF,SSTL,ESTL)
      PCAL=SCO*(AG-AS)+SSTL*AS
      IF(ABS(AXIAL).GE.ABS(PCAL)) GO TO 310
      DE=DE/10.0
      PARE=ABS((AXIAL-PCAL)/AXIAL)
      IF(PARE.LT.0.005) GO TO 1000
      EF=ESAVE
      GO TO 315
 100  PCAL=PI
      DE=EI/10.0
      EF=EI
 110  PARE=ABS((AXIAL-PCAL)/AXIAL)
      IF(PARE.LT.0.005) GO TO 1000
 115  ESAVE=EF
      KC=KC+1
      IF(KC.GT.25) STOP 965
      EF=EF-DE
      IF(ICON.EQ.0) CALL FC(EF,SCO,ECON)
      IF(ICON.EQ.1) CALL FCON(EF,SCO,ECON)
      CALL FS(EF,SSTL,ESTL)
      PCAL=SCO*(AG-AS)+SSTL*AS
      IF(ABS(AXIAL).LE.ABS(PCAL)) GO TO 110
      DE=DE/10.0
      PARE=ABS((AXIAL-PCAL)/AXIAL)
      IF(PARE.LT.0.005) GO TO 1000
      EF=ESAVE
      GO TO 115
 1000 EFF=EF
      DO 50 J=1,NFIB
 50   EIC(J)=EFF
      DO 60 J=1,NSTOT
 60   EIS(J)=EFF
      RETURN
C**  THIS PROGRAM VALID ON FTN4 AND FTN5 **
      END
C*F45V1P0*
      SUBROUTINE CNSTF(CONFX,CONFY,CPA,STLX,STLY,STLA,EIC,EIS,
     1NFIB,NSTOT,DA11,DA12,DA13,DA21,DA22,DA23,DA31,DA32,DA33,ICON)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      DIMENSION CONFX(200),CONFY(200),CPA(200),STLX(200),STLY(200),
     1STLA(200),EIC(200),EIS(200)
      DA11=0.0
      DA12=0.0
      DA13=0.0
      DA21=0.0
      DA22=0.0
      DA23=0.0
      DA31=0.0
      DA32=0.0
      DA33=0.0
      DO 10 I=1,NFIB
      ECON=EIC(I)
      IF(ICON.EQ.0) CALL FC(ECON,SC,ETC)
      IF(ICON.EQ.1) CALL FCON(ECON,SC,ETC)
      DA11=DA11+ETC*CPA(I)
      DA12=DA12+ETC*CONFY(I)*CPA(I)
      DA13=DA13+ETC*CONFX(I)*CPA(I)
      DA21=DA21+ETC*CONFY(I)*CPA(I)
      DA22=DA22+ETC*CONFY(I)**2*CPA(I)
      DA23=DA23+ETC*CONFX(I)*CONFY(I)*CPA(I)
      DA31=DA31+ETC*CONFX(I)*CPA(I)
```

```
      DA32=DA32+ETC*CONFX(I)*CONFY(I)*CFA(I)
      DA33=DA33+ETC*CONFX(I)**2*CFA(I)
 10   CONTINUE
      DO 20 J=1,NSTOT
      ESTL=EZS(J)
      CALL PS(ESTL,SS,ETS)
      IF(ICON.EQ.0) CALL PC(ESTL,SSB,ESUB)
      IF(ICON.EQ.1) CALL PCON(ESTL,SSB,ESUB)
      EEPF=ETS-ESUB
      DA11=DA11+EEPF*STLA(J)
      DA12=DA12+EEPF*STLY(J)*STLA(J)
      DA13=DA13+EEPF*STLX(J)*STLA(J)
      DA21=DA21+EEPF*STLY(J)*STLA(J)
      DA22=DA22+EEPF*STLY(J)**2*STLA(J)
      DA23=DA23+EEPF*STLX(J)*STLY(J)*STLA(J)
      DA31=DA31+EEPF*STLX(J)*STLA(J)
      DA32=DA32+EEPF*STLX(J)*STLY(J)*STLA(J)
      DA33=DA33+EEPF*STLX(J)**2*STLA(J)
 20   CONTINUE
      RETURN
C**  THIS PROGRAM VALID ON FTN4 AND FTN5 **
      END
C*FASV1PG*
      SUBROUTINE PS(EP,SS,ETS)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      COMMON /L2/ FY,EY,RATIO
      EP=-EP
C     CALCULATION OF STEEL STRESS AND TANGENT STIFFNESS
      IF(EP.LT.-0.00010) GO TO 4
      SS=EY*EP
      ETS=EY
      IF(SS.GT.FY) SS=FY
      IF(SS.GE.FY) ETS=0.0
      GO TO 10
 4    X=ABS(EP/.0001)
      ADD=EY/508770./RATIO
      ADK=EY/2544.
      IF(RATIO.LT..005) ADD=ADK+(ADD-ADK)*(RATIO/.005)
      SS=EP*EY-ADD*(1.0-1./X)
      ETS=EY+0.0001*ADD/(EP**2)
      IF(SS.LT.-FY) SS=-FY
      IF(SS.LE.-FY) ETS=0.0
 10   EP=-EP
      SS=-SS
      RETURN
C**  THIS PROGRAM VALID ON FTN4 AND FTN5 **
      END
C*FASV1PG*
      SUBROUTINE PC(EP,STRESS,ETC)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      COMMON /L1/ CYL,RK,EC,ENO,EU
C     CONCRETE STRESS AND TANGENT STIFFNESS USING HOGNESTAD STRESS BLOCK
      EUHOG=1.5*ENO
      EP=-EP
      IF(EP) 2,3,4
 2    SC=0.
      ETAN=EC/10.**10
      GO TO 1
 3    SC=0.0
      ETAN=EC
      GO TO 1
 4    SC=RK*CYL*(2.0*EP/ENO-(EP/ENO)**2)
      ETAN=RK*CYL*(2.0/ENO-2.0*(EP/ENO)*(1.0/ENO))
      IF(EP.GT.ENO) SC=RK*CYL*(1.0-.15*(EP-ENO)/(EUHOG-ENO))
      IF(EP.GT.ENO) ETAN=-.15*RK*CYL/(EUHOG-ENO)
 1    STRESS=-SC
      ETC=ETAN
      EP=-EP
      RETURN
C**  THIS PROGRAM VALID ON FTN4 AND FTN5 **
      END
C*FASV1PG*
      SUBROUTINE PCON(EP,STRESS,ETC)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      COMMON /L1/ CYL,RK,EC,ENO,EU
C      CONCRETE STRESS AND TANGENT STIFFNESS FOR CONFINED
C          CONCRETE USING FORD STRESS-STRAIN CURVE
      EP=-EP
      IF(EP) 2,3,4
 2    SC=0.
      ETAN=EC/10.**10
      GO TO 1
 3    SC=0.0
      ETAN=EC
      GO TO 1
 4    SC=RK*CYL*(2.0*EP/ENO-(EP/ENO)**2)
      ETAN=RK*CYL*(2.0/ENO-2.0*(EP/ENO)*(1.0/ENO))
      IF(EP.GT.ENO) SC=RK*CYL*(1.0-20.0*(EP-ENO))
      IF(EP.GT.ENO) ETAN=-20.0*RK*CYL
 1    STRESS=-SC
      ETC=ETAN
      EP=-EP
      RETURN
C**  THIS PROGRAM VALID ON FTN4 AND FTN5 **
      END
C*FASV1PG*
      SUBROUTINE PSINV(A,NDIM,N)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      DIMENSION A(NDIM,NDIM)
      N1=N-1
      DO 10 I=1,N1
      C=1./A(I,I)
      I1=I+1
      DO 11 J=I1,N
 11   A(I,J)=A(I,J)*C
      DO 12 K=I1,N
      DO 12 J=I1,N
 12   A(K,J)=A(K,J)-A(K,I)*A(I,J)
 10   CONTINUE
      RETURN
C**  THIS PROGRAM VALID ON FTN4 AND FTN5 **
      END
C*FASV1PG*
      SUBROUTINE PSMULT(A,B,NDIM,N,MDIM,M)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      DIMENSION A(NDIM,NDIM),B(MDIM,MDIM)
      N1=N-1
      DO 10 I=1,N1
      C=1./A(I,I)
      DO 11 J=1,M
 11   B(I,J)=B(I,J)*C
      I1=I+1
      DO 12 K=I1,N
      DO 12 J=1,M
 12   B(K,J)=B(K,J)-A(K,I)*B(I,J)
 10   CONTINUE
```

```
      C=1./A(N,N)
      DO 13 J=1,M
13    B(N,J)=B(N,J)*C
      DO 14 II=1,N1
      I=N-II
      I1=I+1
      DO 15 K=I1,N
      DO 15 J=1,M
15    B(I,J)=B(I,J)-A(I,K)*B(K,J)
14    CONTINUE
      RETURN
C**   THIS PROGRAM VALID ON FTN4 AND FTN5 **
      END
```

A.6 Example Problem

The following are the input data, explanatory remarks for the input data, and the data output for the example hollow section shown in Ref. 44. The input will vary slightly for each type of section and if the program steel fiber generation option is used.

No. Card	Type Card
1	1
2	2
3	3
4	4b.
5	4b.1
6	5.a.
7	5.a.1
8	6
9	7
10	8
11	9

THE UNIVERSITY OF TEXAS COMPUTATION CENTER FORTRAN - COMPASS PROGRAMMING FORM

NAME Randy Poston PROGRAM BIMPNI

```
SAMPLE PROBLEM USING WALLER SECTION
 1000.      10000.      0.65       3600000.   29000000.   0.0038
WALLER              0          0
 100.       12.         1.14       1.2
14
        1   11.8        19.1
 6.0        6.0
 1161000.    0.
011
 0.0        0.0         0.0
 0.0        0.0000005   0.0
```

A.6.1 Input Data for Example Problem

A.6.2 Explanatory Remarks for Data Input of Example

Card No.	Card Type	Remarks
1	1	Title
2	2	Material properties
3	3	Section Card (type of section, concrete modeled as unconfined, program, steel fiber generation option used)
4	4b	Geometry of section (width, flange thickness, depth, web thickness)
5	4.b.1	No interior walls in hollow section
6	5a	Reinforcing steel description (1 ring, area of steel for one end, area of steel for one side)
7	5.a.1	Concrete cover (x-cover, y-cover)
8	6	Initial conditions (initial axial load, initial preset strain)
9	7	Joint release code (incremental curvature procedure)
10	8	Specified forces (since it is an incremental curvature procedure, forces specified as 0)
11	9	Specified displacements (only curvature about the x-axis is specified)

A.6.3 Selected Data Output

```
SAMPLE HOLLOW SECTION


CONCRETE STRENGTH           =    4000.00
STEEL YIELD POINT           =   40000.00
STRENGTH REDUCTION FACTOR=           .85
CONCRETE MODULUS            =    .3600E+07
STEEL MODULUS               =    .2900E+08
CONCRETE ULTIMATE STRAIN =       .3000E-02


THE PIER SECTION IS  HOLLOW

CONCRETE FIBERS WILL BE MODELLED AS UNCONFINED CONCRETE

STEEL FIBERS ARE GENERATED BY THE PROGRAM

X-DIMENSION=     108.00
FLANGE THICKNESS=      12.00
Y-DIMENSION=     174.00
WEB THICKNESS  =       12.00
```

CONCRETE FIBER PROPERTIES

NO.	X	Y	AREA
1	-51.00	84.00	36.00
2	-45.00	84.00	36.00
3	-39.53	84.00	29.65
4	-34.59	84.00	29.65
5	-29.65	84.00	29.65
6	-24.71	84.00	29.65
7	-19.76	84.00	29.65
8	-14.82	84.00	29.65
9	-9.88	84.00	29.65
10	-4.94	84.00	29.65
11	.00	84.00	29.65
12	4.94	84.00	29.65
13	9.88	84.00	29.65
14	14.82	84.00	29.65
15	19.76	84.00	29.65
16	24.71	84.00	29.65
17	29.65	84.00	29.65
18	34.59	84.00	29.65
19	39.53	84.00	29.65
20	45.00	84.00	36.00
21	51.00	84.00	36.00
22	-51.00	78.00	36.00
23	-45.00	78.00	36.00
24	-39.53	78.00	29.65
25	-34.59	78.00	29.65
26	-29.65	78.00	29.65
27	-24.71	78.00	29.65
28	-19.76	78.00	29.65
29	-14.82	78.00	29.65
30	-9.88	78.00	29.65
31	-4.94	78.00	29.65
32	.00	78.00	29.65

33	4.94	78.00	29.65
34	9.88	78.00	29.65
35	14.82	78.00	29.65
36	19.76	78.00	29.65
37	24.71	78.00	29.65
38	29.65	78.00	29.65
39	34.59	78.00	29.65
40	39.53	78.00	29.65
41	45.00	78.00	36.00
42	51.00	78.00	36.00
43	-51.00	70.59	52.94
44	-45.00	70.59	52.94
45	45.00	70.59	52.94
46	51.00	70.59	52.94
47	-51.00	61.76	52.94
48	-45.00	61.76	52.94
49	45.00	61.76	52.94
50	51.00	61.76	52.94
51	-51.00	52.94	52.94
52	-45.00	52.94	52.94
53	45.00	52.94	52.94
54	51.00	52.94	52.94
55	-51.00	44.12	52.94
56	-45.00	44.12	52.94
57	45.00	44.12	52.94
58	51.00	44.12	52.94
59	-51.00	35.29	52.94
60	-45.00	35.29	52.94
61	45.00	35.29	52.94
62	51.00	35.29	52.94
63	-51.00	26.47	52.94
64	-45.00	26.47	52.94
65	45.00	26.47	52.94
66	51.00	26.47	52.94
67	-51.00	17.65	52.94
68	-45.00	17.65	52.94
69	45.00	17.65	52.94
70	51.00	17.65	52.94
71	-51.00	8.82	52.94
72	-45.00	8.82	52.94
73	45.00	8.82	52.94
74	51.00	8.82	52.94
75	-51.00	-.00	52.94
76	-45.00	-.00	52.94
77	45.00	-.00	52.94
78	51.00	-.00	52.94
79	-51.00	-8.82	52.94
80	-45.00	-8.82	52.94
81	45.00	-8.82	52.94
82	51.00	-8.82	52.94
83	-51.00	-17.65	52.94
84	-45.00	-17.65	52.94
85	45.00	-17.65	52.94
86	51.00	-17.65	52.94
87	-51.00	-26.47	52.94
88	-45.00	-26.47	52.94
89	45.00	-26.47	52.94
90	51.00	-26.47	52.94
91	-51.00	-35.29	52.94
92	-45.00	-35.29	52.94
93	45.00	-35.29	52.94
94	51.00	-35.29	52.94

95	-51.00	-44.12	52.94
96	-45.00	-44.12	52.94
97	45.00	-44.12	52.94
98	51.00	-44.12	52.94
99	-51.00	-52.94	52.94
100	-45.00	-52.94	52.94
101	45.00	-52.94	52.94
102	51.00	-52.94	52.94
103	-51.00	-61.76	52.94
104	-45.00	-61.76	52.94
105	45.00	-61.76	52.94
106	51.00	-61.76	52.94
107	-51.00	-70.59	52.94
108	-45.00	-70.59	52.94
109	45.00	-70.59	52.94
110	51.00	-70.59	52.94
111	-51.00	-78.00	36.00
112	-45.00	-78.00	36.00
113	-39.53	-78.00	29.65
114	-34.59	-78.00	29.65
115	-29.65	-78.00	29.65
116	-24.71	-78.00	29.65
117	-19.76	-78.00	29.65
118	-14.82	-78.00	29.65
119	-9.88	-78.00	29.65
120	-4.94	-78.00	29.65
121	.00	-78.00	29.65
122	4.94	-78.00	29.65
123	9.88	-78.00	29.65
124	14.82	-78.00	29.65
125	19.76	-78.00	29.65
126	24.71	-78.00	29.65
127	29.65	-78.00	29.65
128	34.59	-78.00	29.65
129	39.53	-78.00	29.65
130	45.00	-78.00	36.00
131	51.00	-78.00	36.00
132	-51.00	-84.00	36.00
133	-45.00	-84.00	36.00
134	-39.53	-84.00	29.65
135	-34.59	-84.00	29.65
136	-29.65	-84.00	29.65
137	-24.71	-84.00	29.65
138	-19.76	-84.00	29.65
139	-14.82	-84.00	29.65
140	-9.88	-84.00	29.65
141	-4.94	-84.00	29.65
142	.00	-84.00	29.65
143	4.94	-84.00	29.65
144	9.88	-84.00	29.65
145	14.82	-84.00	29.65
146	19.76	-84.00	29.65
147	24.71	-84.00	29.65
148	29.65	-84.00	29.65
149	34.59	-84.00	29.65
150	39.53	-84.00	29.65
151	45.00	-84.00	36.00
152	51.00	-84.00	36.00

MOMENT OF INERTIA ABOUT X-AXIS= .238E+08
MOMENT OF INERTIA ABOUT Y-AXIS= .108E+08

```
NUMBER OF STEEL RINGS=    1
NO.     AREA OF END STEEL     AREA OF SIDE STEEL
1              11.80                    19.10

NO.       X COVER      Y COVER
1            6.00         6.00
```

STEEL FIBER PROPERTIES

NO.	X	Y	AREA
1	-46.74	81.00	.31
2	-44.21	81.00	.31
3	-41.68	81.00	.31
4	-39.16	81.00	.31
5	-36.63	81.00	.31
6	-34.11	81.00	.31
7	-31.58	81.00	.31
8	-29.05	81.00	.31
9	-26.53	81.00	.31
10	-24.00	81.00	.31
11	-21.47	81.00	.31
12	-18.95	81.00	.31
13	-16.42	81.00	.31
14	-13.89	81.00	.31
15	-11.37	81.00	.31
16	-8.84	81.00	.31
17	-6.32	81.00	.31
18	-3.79	81.00	.31
19	-1.26	81.00	.31
20	1.26	81.00	.31
21	3.79	81.00	.31
22	6.32	81.00	.31
23	8.84	81.00	.31
24	11.37	81.00	.31
25	13.89	81.00	.31
26	16.42	81.00	.31
27	18.95	81.00	.31
28	21.47	81.00	.31
29	24.00	81.00	.31
30	26.53	81.00	.31
31	29.05	81.00	.31
32	31.58	81.00	.31
33	34.11	81.00	.31
34	36.63	81.00	.31
35	39.16	81.00	.31
36	41.68	81.00	.31
37	44.21	81.00	.31
38	46.74	81.00	.31
39	-46.74	-81.00	.31
40	-44.21	-81.00	.31
41	-41.68	-81.00	.31
42	-39.16	-81.00	.31
43	-36.63	-81.00	.31
44	-34.11	-81.00	.31
45	-31.58	-81.00	.31
46	-29.05	-81.00	.31

47	-26.53	-81.00	.31
48	-24.00	-81.00	.31
49	-21.47	-81.00	.31
50	-18.95	-81.00	.31
51	-16.42	-81.00	.31
52	-13.89	-81.00	.31
53	-11.37	-81.00	.31
54	-8.84	-81.00	.31
55	-6.32	-81.00	.31
56	-3.79	-81.00	.31
57	-1.26	-81.00	.31
58	1.26	-81.00	.31
59	3.79	-81.00	.31
60	6.32	-81.00	.31
61	8.84	-81.00	.31
62	11.37	-81.00	.31
63	13.89	-81.00	.31
64	16.42	-81.00	.31
65	18.95	-81.00	.31
66	21.47	-81.00	.31
67	24.00	-81.00	.31
68	26.53	-81.00	.31
69	29.05	-81.00	.31
70	31.58	-81.00	.31
71	34.11	-81.00	.31
72	36.63	-81.00	.31
73	39.16	-81.00	.31
74	41.68	-81.00	.31
75	44.21	-81.00	.31
76	46.74	-81.00	.31
77	-48.00	79.67	.31
78	-48.00	77.02	.31
79	-48.00	74.36	.31
80	-48.00	71.70	.31
81	-48.00	69.05	.31
82	-48.00	66.39	.31
83	-48.00	63.74	.31
84	-48.00	61.08	.31
85	-48.00	58.43	.31
86	-48.00	55.77	.31
87	-48.00	53.11	.31
88	-48.00	50.46	.31
89	-48.00	47.80	.31
90	-48.00	45.15	.31
91	-48.00	42.49	.31
92	-48.00	39.84	.31
93	-48.00	37.18	.31
94	-48.00	34.52	.31
95	-48.00	31.87	.31
96	-48.00	29.21	.31
97	-48.00	26.56	.31
98	-48.00	23.90	.31
99	-48.00	21.25	.31
100	-48.00	18.59	.31
101	-48.00	15.93	.31
102	-48.00	13.28	.31
103	-48.00	10.62	.31
104	-48.00	7.97	.31
105	-48.00	5.31	.31
106	-48.00	2.66	.31
107	-48.00	.00	.31
108	-48.00	-2.66	.31

109	-48.00	-5.31	.31
110	-48.00	-7.97	.31
111	-48.00	-10.62	.31
112	-48.00	-13.28	.31
113	-48.00	-15.93	.31
114	-48.00	-18.59	.31
115	-48.00	-21.25	.31
116	-48.00	-23.90	.31
117	-48.00	-26.56	.31
118	-48.00	-29.21	.31
119	-48.00	-31.87	.31
120	-48.00	-34.52	.31
121	-48.00	-37.18	.31
122	-48.00	-39.84	.31
123	-48.00	-42.49	.31
124	-48.00	-45.15	.31
125	-48.00	-47.80	.31
126	-48.00	-50.46	.31
127	-48.00	-53.11	.31
128	-48.00	-55.77	.31
129	-48.00	-58.43	.31
130	-48.00	-61.08	.31
131	-48.00	-63.74	.31
132	-48.00	-66.39	.31
133	-48.00	-69.05	.31
134	-48.00	-71.70	.31
135	-48.00	-74.36	.31
136	-48.00	-77.02	.31
137	-48.00	-79.67	.31
138	48.00	79.67	.31
139	48.00	77.02	.31
140	48.00	74.36	.31
141	48.00	71.70	.31
142	48.00	69.05	.31
143	48.00	66.39	.31
144	48.00	63.74	.31
145	48.00	61.08	.31
146	48.00	58.43	.31
147	48.00	55.77	.31
148	48.00	53.11	.31
149	48.00	50.46	.31
150	48.00	47.80	.31
151	48.00	45.15	.31
152	48.00	42.49	.31
153	48.00	39.84	.31
154	48.00	37.18	.31
155	48.00	34.52	.31
156	48.00	31.87	.31
157	48.00	29.21	.31
158	48.00	26.56	.31
159	48.00	23.90	.31
160	48.00	21.25	.31
161	48.00	18.59	.31
162	48.00	15.93	.31
163	48.00	13.28	.31
164	48.00	10.62	.31
165	48.00	7.97	.31
166	48.00	5.31	.31
167	48.00	2.66	.31
168	48.00	.00	.31
169	48.00	-2.66	.31
170	48.00	-5.31	.31

171	48.00	-7.97	.31
172	48.00	-10.62	.31
173	48.00	-13.28	.31
174	48.00	-15.93	.31
175	48.00	-18.59	.31
176	48.00	-21.25	.31
177	48.00	-23.90	.31
178	48.00	-26.56	.31
179	48.00	-29.21	.31
180	48.00	-31.87	.31
181	48.00	-34.52	.31
182	48.00	-37.18	.31
183	48.00	-39.84	.31
184	48.00	-42.49	.31
185	48.00	-45.15	.31
186	48.00	-47.80	.31
187	48.00	-50.46	.31
188	48.00	-53.11	.31
189	48.00	-55.77	.31
190	48.00	-58.43	.31
191	48.00	-61.08	.31
192	48.00	-63.74	.31
193	48.00	-66.39	.31
194	48.00	-69.05	.31
195	48.00	-71.70	.31
196	48.00	-74.36	.31
197	48.00	-77.02	.31
198	48.00	-79.67	.31

BEGINNING AXIAL LOAD = -.1166E+00
INITIAL PRESET STRAIN = 0.

RELEASE CODE
011

AXIAL LOAD	MOMENT-X	MOMENT-Y
0.	0.	0.

AXIAL STRAIN	CURVATURE-X	CURVATURE-Y
0.	.508E-06	0.

INITIAL STRAIN= .35663E-03

NO.	AXIAL LOAD	MOMENT-X	MOMENT-Y	CURVATURE-X	CURVATURE-Y
1	-.11661E+08	.33897E+08	-.91553E-09	.50000E-06	0.
1	-.11633E+08	.33897E+08	.11567E-05		
2	-.11661E+08	.66180E+08	-.95316E-08	.10000E-05	0.
2	-.11628E+08	.66165E+08	.15451E-05		
3	-.11661E+08	.99204E+08	-.12851E-07	.15000E-05	0.
3	-.11622E+08	.99168E+08	.15646E-05		
4	-.11661E+08	.13213E+09	-.12058E-07	.20000E-05	0.
4	-.11616E+08	.13200E+09	.15926E-05		
5	-.11661E+08	.16498E+09	-.19108E-07	.25000E-05	0.
5	-.11611E+08	.16476E+09	.11511E-05		
6	-.11661E+08	.19749E+09	-.25574E-07	.30000E-05	0.
6	-.11609E+08	.19727E+09	.15683E-05		
7	-.11661E+08	.22984E+09	-.29345E-07	.35000E-05	0.
7	-.11599E+08	.22953E+09	.11139E-05		
8	-.11661E+08	.26191E+09	-.26748E-07	.40000E-05	0.
8	-.11594E+08	.26158E+09	.98152E-06		
9	-.11661E+08	.29365E+09	-.23885E-07	.45000E-05	0.
9	-.11588E+08	.29312E+09	.11181E-05		
10	-.11661E+08	.32502E+09	-.39258E-07	.50000E-05	0.
10	-.11582E+08	.32436E+09	.89487E-06		
11	-.11661E+08	.35597E+09	-.49599E-07	.55000E-05	0.
11	-.11576E+08	.35516E+09	.10468E-05		
12	-.11661E+08	.38645E+09	-.85092E-07	.60000E-05	0.
12	-.11578E+08	.38548E+09	.12208E-05		
13	-.11661E+08	.41642E+09	-.11968E-06	.65000E-05	0.
13	-.11564E+08	.41526E+09	.11213E-05		
14	-.11661E+08	.44581E+09	-.13553E-06	.70000E-05	0.
14	-.11558E+08	.44446E+09	.15181E-05		
15	-.11661E+08	.47458E+09	-.15346E-06	.75000E-05	0.
15	-.11552E+08	.47382E+09	.11309E-05		
16	-.11661E+08	.50268E+09	-.16818E-06	.80000E-05	0.
16	-.11606E+08	.49581E+09	.15777E-05		
17	-.11661E+08	.52483E+09	-.17883E-06	.85000E-05	0.
17	-.11627E+08	.51577E+09	.77114E-06		
18	-.11661E+08	.54852E+09	-.16158E-06	.90000E-05	0.
18	-.11623E+08	.53126E+09	.13672E-05		
19	-.11661E+08	.55469E+09	-.15438E-06	.95000E-05	0.
19	-.11622E+08	.54503E+09	.11548E-05		

NO.	AXIAL LOAD	MOMENT-X	MOMENT-Y	CURVATURE-X	CURVATURE-Y
20	-.11661E+08	.56822E+09	-.14784E-06	.10000E-04	0.
20	-.11622E+08	.55794E+09	.13672E-05		
21	-.11661E+08	.58063E+09	-.14511E-06	.10500E-04	0.
21	-.11620E+08	.56994E+09	.11846E-05		
22	-.11661E+08	.59219E+09	-.13987E-06	.11000E-04	0.
22	-.11631E+08	.58834E+09	.12219E-05		
23	-.11661E+08	.60181E+09	-.13957E-06	.11500E-04	0.
23	-.11627E+08	.58971E+09	.12591E-05		
24	-.11661E+08	.61091E+09	-.14065E-06	.12000E-04	0.
24	-.11624E+08	.59850E+09	.10245E-05		
25	-.11661E+08	.61944E+09	-.13665E-06	.12500E-04	0.
25	-.11621E+08	.60673E+09	.13746E-05		
26	-.11661E+08	.62733E+09	-.13669E-06	.13000E-04	0.
26	-.11631E+08	.61364E+09	.13113E-05		
27	-.11661E+08	.63379E+09	-.13602E-06	.13500E-04	0.
27	-.11628E+08	.61986E+09	.10505E-05		
28	-.11661E+08	.63971E+09	-.13270E-06	.14000E-04	0.
28	-.11624E+08	.62553E+09	.84937E-06		
29	-.11661E+08	.64587E+09	-.13538E-06	.14500E-04	0.
29	-.11620E+08	.63088E+09	.16583E-05		
30	-.11661E+08	.64899E+09	-.13448E-06	.15000E-04	0.
30	-.11617E+08	.63423E+09	.11864E-05		
31	-.11661E+08	.65250E+09	-.14440E-06	.15500E-04	0.
31	-.11612E+08	.63737E+09	.14981E-05		
32	-.11661E+08	.65473E+09	-.15191E-06	.16000E-04	0.
32	-.11614E+08	.63937E+09	.15606E-05		
33	-.11661E+08	.65645E+09	-.16483E-06	.16500E-04	0.
33	-.11612E+08	.64180E+09	.11511E-05		
34	-.11661E+08	.65797E+09	-.17142E-06	.17000E-04	0.
34	-.11609E+08	.64231E+09	.92015E-06		
35	-.11661E+08	.65903E+09	-.17636E-06	.17500E-04	0.
35	-.11608E+08	.64333E+09	.13374E-05		
36	-.11661E+08	.65995E+09	-.18807E-06	.18000E-04	0.
36	-.11606E+08	.64417E+09	.11995E-05		
37	-.11661E+08	.66067E+09	-.17827E-06	.18500E-04	0.
37	-.11605E+08	.64482E+09	.39488E-06		
38	-.11661E+08	.66097E+09	-.18127E-06	.19000E-04	0.
38	-.11604E+08	.64508E+09	.11362E-05		

NO.	AXIAL LOAD	MOMENT-X	MOMENT-Y	CURVATURE-X	CURVATURE-Y
39	-.11661E+08	.66115E+09	-.18856E-06	.19500E-04	0.
39	-.11602E+08	.64523E+09	.12052E-05		
40	-.11661E+08	.66117E+09	-.19431E-06	.20000E-04	0.
40	-.11601E+08	.64521E+09	.21030E-05		
41	-.11661E+08	.66106E+09	-.19076E-06	.20500E-04	0.
41	-.11600E+08	.64505E+09	.11064E-05		
42	-.11661E+08	.66079E+09	-.19185E-06	.21000E-04	0.
42	-.11597E+08	.64464E+09	.12102E-05		
43	-.11661E+08	.66020E+09	-.19597E-06	.21500E-04	0.
43	-.11596E+08	.64401E+09	.11023E-05		
44	-.11661E+08	.65940E+09	-.19546E-06	.22000E-04	0.
44	-.11595E+08	.64326E+09	.13672E-05		
45	-.11661E+08	.65865E+09	-.19640E-06	.22500E-04	0.
45	-.11594E+08	.64240E+09	.10356E-05		
46	-.11661E+08	.65768E+09	-.19856E-06	.23000E-04	0.
46	-.11593E+08	.64141E+09	.11208E-05		
47	-.11661E+08	.65659E+09	-.19544E-06	.23500E-04	0.
47	-.11590E+08	.64020E+09	.15007E-05		
48	-.11661E+08	.65520E+09	-.19353E-06	.24000E-04	0.
48	-.11589E+08	.63879E+09	.11846E-05		
49	-.11661E+08	.65372E+09	-.20133E-06	.24500E-04	0.
49	-.11588E+08	.63729E+09	.10431E-05		
50	-.11661E+08	.65214E+09	-.21617E-06	.25000E-04	0.
50	-.11587E+08	.63569E+09	.12480E-05		
51	-.11661E+08	.65045E+09	-.22729E-06	.25500E-04	0.
51	-.11586E+08	.63398E+09	.12740E-05		
52	-.11661E+08	.64865E+09	-.23808E-06	.26000E-04	0.
52	-.11585E+08	.63219E+09	.10356E-05		
53	-.11661E+08	.64654E+09	-.26036E-06	.26500E-04	0.
53	-.11584E+08	.63002E+09	.46566E-06		
54	-.11661E+08	.64436E+09	-.27079E-06	.27000E-04	0.
54	-.11583E+08	.62783E+09	.84192E-06		
55	-.11661E+08	.64209E+09	-.29177E-06	.27500E-04	0.
55	-.11582E+08	.62555E+09	.10543E-05		
56	-.11661E+08	.63972E+09	-.31116E-06	.28000E-04	0.
56	-.11584E+08	.62297E+09	.93132E-06		
57	-.11661E+08	.63700E+09	-.32998E-06	.28500E-04	0.
57	-.11583E+08	.62025E+09	.13970E-05		

NO.	AXIAL LOAD	MOMENT-X	MOMENT-Y	CURVATURE-Y	CURVATURE-Y
58	-.11661E+08	.63418E+09	-.30800E-06	.29908E-06	0.
58	-.11582E+08	.61741E+09	.10021E-05		
59	-.11661E+08	.63128E+09	-.36891E-06	.29580E-06	0.
59	-.11581E+08	.61446E+09	.87172E-06		
60	-.11661E+08	.62814E+09	-.38971E-06	.30008E-06	0.
60	-.11580E+08	.61133E+09	.95748E-06		
61	-.11661E+08	.62493E+09	-.40955E-06	.30588E-06	0.
61	-.11579E+08	.60811E+09	.13568E-05		
62	-.11661E+08	.62165E+09	-.83468E-06	.31808E-06	0.
62	-.11579E+08	.60483E+09	.94995E-06		

APPENDIX B

PROGRAM PIER

B.1 General

Identification. The computer program discussed within this appendix, entitled PIER, is written in FORTRAN 77 computer language.

Application. The primary objective of the program is the structural analysis of a reinforced concrete single pier bent beam-column subjected to static loads.

Coordinate System and Sign Convention. The positive axes of a pier are shown in Fig. B.2. Forces and displacements are positive if in the directions shown in Fig. B.4. Moments and rotations are positive if given by the right-hand rule as in Fig. B.4.

B.2 Definition of Main Variables

AG	= gross area of section
AKFX(11)	= vector of translational spring stiffnesses in the x-direction at joints
AKFY(11)	= vector of translational spring stiffnesses in the y-direction at joints
AKFZ(11)	= vector of translational spring stiffnesses in the z-direction at joints
AKX(11)	= vector of rotational spring stiffnesses about the x-axis at joints
AKY(11)	= vector of rotational apring stiffnesses about the z-axis at joints
AL	= length of pier
AMAT(3,3)	= A-matrix for a segment
AST	= area of reinforcing steel
AXF(10)	= auxiliary matrix containing segment axial loads
BETA	= creep factor
BMAT(3,3,100)	= B-matrix of all sections in a member
CFA(200,10)	= matrix containing the area of all concrete fibers in a member
CONFX(200,10)	= matrix containing x-centroid of all concrete fibers in a member

CONFZ(200,10)	= matrix containing z-centroid of all concrete fibers in a member
CTROID(5)	= distance from centroid of section to interior walls
CYL	= compressive cylinder strength of concrete
DA_{11}, DA_{12}, . . ., DA_{13}	= stiffness parameters of section
DAL	= length of a segment
DFORA(6,10), DFORB(6,10)	= auxiliary matrices containing member end forces
DU(66)	= matrix containing the incremental deformations of each joint
DUA(6), DUB(6)	= auxiliary vector containing segment forces
EC	= initial modulus of concrete
ENO	= strain corresponding to maximum stress on concrete compression stress-strain curve
EPS(10,10)	= contains axial strain for a section
EU	= ultimate strength of concrete
EY	= steel modulus
EZC(200)	= array containing total strain of a concrete fiber of a given section
EZS(200)	= array containing total strain of a steel fiber of a given section
FIY(10,10)	= contains rotational strain about the y-axis of a section
FIZ(10,10)	= contains rotational strain about the z-axis of a section
FOR(6)	= auxiliary array to read forces or displacements
FORA(6,10)	= contains member end-A forces
FORB(6,10)	= contains member end-B forces
FY	= yield stress of steel
GAMMAX(3)	= x-distance from end edge of section to centroid of top or bottom steel

GAMMAZ(3)	=	z-distance from side edge of section to centroid of side steel
ICODE	=	code to indicate if hollow section has interior walls
ICON	=	code to define if concrete is unconfined or confined
IOR	=	orientation of interior walls; either horizontal or vertical
IPRNT	=	print option of section fibers
IS	=	code to indicate if segment lengths are equal
ISTAT	=	code to indicate if program will generate steel fibers or if the user will read in individual reinforcement properties
ITITLE(20)	=	title of problem
ITYPE	=	type of section (solid, hollow, oval or circular hollow)
JPRNT	=	code to indicate which incremental results will be printed
JR(11)	=	vector of joint release codes of nodal points
LGI	=	identification of load factor groups
LTI	=	load type
NCJ	=	number of constrained joints
NDF	=	number of degrees of freedom
NFIB	=	number of concrete fibers in a section
NJ	=	number of joints
NJL	=	number of loaded joints
NJSP	=	number of joints with springs
NLFDG	=	number of load factor design groups
NLINC	=	number of load increments
NLJ	=	number of loaded joints
NLOAD	=	number of load cases
NLTYP	=	number of different load types
NRING	=	number of rings of steel
NSECT	=	number of sections to be read
NSEG	=	number of segments

NSTOT	= number of steel fibers in a section
NW	= number of interior walls in hollow section
PDIS(66)	= vector of applied displacement increments
PFOR(66)	= vector of applied force increments
RATIO(10)	= ratio of reinforcement area to section area
RK	= reduction factor applied to f'_c
ROT(6,6)	= rotation matrix
SA(6,6,11), SB(6,6,11), SC(6,6,11)	= auxiliary matrices containing stiffnesses of members
SKAA(6,6,10), SKAB(6,6,10), SKBA(6,6,10), SKBB(6,6,10)	= flexibility or stiffness of matrices of segments
STBB(6,6), STAA(6,6), STAB(6,6), STBA(6,6)	= flexibility matrices or stiffnesses matrices of a segment at a given load increment
STK(720)	= total stiffness of matrix of a member
STLA(200,10)	= matrix containing the area of all steel fibers
STLX(200,10)	= matrix containing the x-centroid of all steel fibers in a member
STLZ(200,10)	= matrix containing the z-centroid of all steel fibers in a member
STS(3)	= area of steel of one side of section
STT(3)	= area of top or bottom steel of section
TF	= thickness of flange on hollow section
TRAT	= torsional constant
TW	= thickness of web on hollow section

TWALL(5) = thickness of interior walls

U(66) = vector of joint deformations

WLSTL(5) = area of steel interior walls

XI = depth of section, dimension along x-axis

XIX = moment of inertia about the x-axis

ZI = width of section, dimension along z-axis

ZIZ = moment of inertia about the z-axis

B.3 Guide to Input Data

The program data are input by means of data cards. No units are implied. The user must use consistent units. See Figs. B.1 through B.4 for sign conventions and section descriptions. Note that axes are different than in BIMPHI. F-fields are for real numbers and data can be placed anywhere within the field. A decimal point must be used. I-fields are for integers and are right justified. A-fields are for alphanumeric designations and are left justified.

1. Title Card (20A4)

   Column 1-80--ITITLE(20), title of the problem. Any FORTRAN characters are acceptable.

2. Member Description (F10.0,3I10)

   Column 1-10--AL, length of member

   Column 11-20--NSEG, number of segments the pier is divided into (limit is 10)

   Column 21-30--NSECT, number of different types of sections required to define the pier geometry
   NSECT = 0 or 1 for prismatic pier with uniform cross section throughout
   NSECT = 2 for linearly tapered pier
   NSECT = NSEG for pier of arbitrary vertical configuration or for a prismatic pier with varying cross sections (limit is 10)

   Column 31-40--IS, parameter for equal segment lengths
   IS = 0 or blank if all the segments lengths are the same
   IS = 1 if the segment lengths are different

3. Segment Lengths (8F10.0)

(Use only if IS = 1)

Column 1-10--DAL(1), length of first segment

Column 11-20--DAL(2), length of second segment

Continue for as many segments as required using extra cards as needed.

4. Support Conditions (2I10)

Column 1-10--NCJ, total number of constrained joints (supports)

Column 11-20--NJSP, total number of joints with springs (Note--all joints are at segment ends)

5. Release Code of Joints (2I10)

Only if NCJ $\neq$ 0; one card for each constrained joint

Column 1-10--J, the number of the joint which is a support (see Fig. B.1 for joint numbering scheme)

Column 11-20--JR(J), joint release code for joint J; 1 for a degree of freedom which is fixed and 0 for a degree of freedom which is not fixed (Column 15--x-translation; Column 16--y-translation; Column 17--z-translation; Column 18--rotation about x-axis; Column 19--rotation about y-axis; Column 20--rotation about z-axis)

6. Spring Information (I10,7F10.0)

Only if NJSP $\neq$ 0; one card for each joint with springs

Column 1-10--J, number of the joint which has springs (see Fig. B.1 for joint numbering scheme)

Column 11-20--AKX(J), stiffness of x-translational spring

Column 21-30--AKY(J), stiffness of y-translational spring

Column 31-40--AKZ(J), stiffness of z-translational spring

Column 41-50--AKFX(J), stiffness of x-rotational spring

Column 51-60--AKFY(J), stiffness of y-rotational spring

Column 61-70--AKFZ(J), stiffness of z-rotational spring

7. Material Properties (8F10.0)

Column 1-10--CYL, compressive cylinder strength of concrete

Column 11-20--FY, yield stress of reinforcing steel

Column 21-30--RK, strength reduction factor for concrete (i.e., $f''_c = 0.85f'_c$; see Sec. 4.5.3)

Column 31-40--EC, initial modulus of concrete

Column 41-50--EY, modulus of reinforcing steel

Column 51-60--EU, ultimate strain of concrete; if left blank, program assumes EU = 1.9 (1.0 + BETA)(RK)(2.0 CYL/EC) (See Sec. 4.5.3)

Column 61-70--BETA, creep factor (see Table B.1)

8. Section Card (2A4,2X,3I10)

Column 1- 8--ITYPE, type of section (type either SOLID, HOLLOW, OVAL or CIRHOLLO (circular hollow section), starting in column 1 for type of section desired)

Column 11-20--ICON, status of concrete (if ICON = 0, unconfined concrete; if ICON = 1, confined concrete). To use confined concrete option, user must ensure enough lateral ties are provided to meet the equivalent of ACI Building Code 318-83 Appendix A Sec. A.4.4 for compression members.

Column 21-30--ISTAT, status of reinforcing steel (if ISTAT = 0, steel fibers generated by the program; if ISTAT = 1, steel fibers read in by the user)

Column 31-40--IPRNT, print parameter (if IPRNT = 0, print out of fiber properties will be suppressed)

Card Group 9-10 should be repeated for the number of sections to describe the pier (i.e., once if NSECT = 0; twice if NSECT = 2; NSEG times if NSECT = NSEG)

9. Geometry of Section J (8F10.0) (see Figs. B.2 and B.3)

(a) If ITYPE = SOLID (rectangular solid)

Column 1-10--ZI, width of section (dimension along z-axis)

Column 11-20--XI, depth of section (dimension along x-axis)

(b) If ITYPE = HOLLOW (rectangular hollow with or without interior walls)

Column 1-10--ZI, width of section (exterior dimension along z-axis)

Column 11-20--TF, thickness of exterior wall which is parallel to z-axis

Column 21-30--XI, depth of section (exterior dimension along x- axis

Column 31-40--TW, thickness of exterior wall which is parallel to x-axis)

9.b.1 <u>Status of Interior Walls of Hollow Section (2A4,3I10)</u>

Column 1- 4-- ICODE (Are there any interior walls? Type YES or NO beginning in column 1.)

If ICODE = YES

9.b.2 <u>Description of Interior Walls (3A4,8X,I10)</u>

Column 1-20--IOR, orientation or walls (type VERTICAL if the walls are parallel to the x-axis; type HORIZONTAL if the walls are parallel to the z-axis; begin in column 1)

Column 21-30--NW, number of interior walls (maximum of 5)

If ICODE = YES

9.b.3 <u>Geometry of Interior Walls (8F10.0)</u>

One or more data cards alternately giving values for pairs of CTROID(I), distance from centroid of interior wall, and TWALL(I), thickness of wall. Pairs repeat for I = 1, NW (one data card can contain CTROID(I) and TWALL(I) for four interior walls. Therefore, in case there is a fifth interior wall an additional card is required).

(c) If ITYPE = OVAL (also used for solid circular)*

Column 1-10--ZI, width of section (dimension along z-axis)

Column 11-20--XI, depth of section (dimension along x-axis)

* The solid circular section is a special case of the oval section with XI = diameter and ZI = XI + 0.01.

(d) If ITYPE = CIRHOLLO (circular hollow)

Column 1-10--ZI, exterior diameter of section

Column 11-20--TF, thickness of wall section (thickness must be less than 1/2 the diameter)

10. Reinforcing Steel Description (see Fig. B.3)

(a) If ISTAT = 0

10.a.1 Description of Steel Rings (I10,7F10.0)

Column 1-10--NRING, number of rings of steel (at least 1 ring, maximum of 3 rings). The rings are symmetric.

Column 11-70--Starting with STT(I), alternate STT(I), area of end steel in ring (parallel to the z-axis), with STS(I), area of side steel in ring (parallel to the x-axis), in fields of F10.0 for I=1 to NRING (for solid circular and hollow circular sections, STT=0 and STS is half the total area of steel)

10.a.2 Concrete Cover (8F10.0)

Column 1-60--Starting with GAMMAZ(I), alternate GAMMAZ(I), distance (parallel to z-axis) from side edge of section to center of side steel, with GAMMAX(I), distance (parallel to x-axis) from end edge of section to center of end steel, in fields of F10.0 for I = 1 to NRING

10.a.3 Steel in Interior Walls (8F10.0)

(Only if ITYPE = HOLLOW and ICODE = YES)

Column 1-50--WLSTL(I), area of steel in each interior wall in fields of F10.0 for I = 1 to NW

(b) If ISTAT = 1

10.b.1 Steel Fibers (I10)

Column 1-10--NSTOT, number of reinforcing bars to be read (maximum of 200)

10.b.2 Description of Reinforcing Bars (2(3F10.2))

Column 1-60--Starting with STLZ(I), alternate three dimension sets of STLZ(I), (z-distance from centroid of section to reinforcing bar), with STLX(I) (x-distance from centroid of section to reinforcing bar) and STLA(I)(area of reinforcing bar) in fields of F10.0 for I-1 to NSTOT. (A set of three numerical values (STLZ, STLX, and STLA) describes a single bar. There are 2 bar descriptions per card.)

11. Type of Analysis (I10)

Column 1-10--NLFDG, number of load factor design groups.

If NLFDG > 0, a number equal to NLFDG of AASHTO load groups will be considered in the analysis (maximum number of load factor design groups is 10).

If NLFDG $\leq$ 0, the analysis will be carried out without factoring loads or using factored load groups.

12. Loading System

(a) If NLFDG $\leq$ 0

12.a.1 Load Cases (2I10)

Column 1-10--NLOAD, number of load cases for the problem (load cases represent the number of different load groups; all pier behavior is cumulative from one load case to the next).

Column 11-20--JPRNT, print parameter (results from every JPRNTth incremental analysis will be printed)

Cards 12.a.2 to 12.a.3 are repeated for the number of load cases, NLOAD.

12.a.2 Load Description (2I10)

Column 1-10--NLJ, total number of loaded joints

Column 11-20--NLINC, number of load increments (the number of load increments for the total number of load cases should not be less than 30 to obtain an accurate solution).

12.a.3 Loads (I10,7F10.0) (Repeat NLJ times) (See Fig. B.4 for sign convention)

One card for loads at each loaded joint. If a joint was specified as a support, and has a 1 specified in the release code for a degree of freedom, and if the user specifies a value at the support for that degree of freedom, he is specifying an incremental displacement or rotation, not a load. For factored load design, all loads should be factored loads and should be divided by proper strength reduction factors of AASHTO Sec. 1.5.30(B)(2).

Column 1-10-- J, number of the loaded joint

Column 11-20--FOR(1), incremental force in the x-direction

Column 21-30--FOR(2), incremental force in the y-direction

Column 31-40--FOR(3), incremental force in the z-direction

Column 41-50--FOR(4), incremental moment about the x-axis

Column 51-60--FOR(5), incremental moment about the y-axis

Column 61-70--FOR(6), incremental moment about the z-axis

(b) If NLFDG > 0

12.b.1 Load Description (8I10)

Column 1-10--JPRNT, print parameter (results from every JPRNTth incremental analysis will be printed)

Column 11-20--NLTYP, number of different load types to be input (D, L, ICE, etc.) (maximum of 14)

Column 21-30--LGI(1), identification of the AASHTO load factor design group combination to be considered first (see Table B.2)

Column 31-40--LGI(2), identification of the AASHTO load factor design group combination to be considered second

Continue in fields of I10 until NLFDG load factor design group combinations have been completed. If NLFDG > 6 begin a new card with LGI(7).

Card 12.b.2 to 12.b.3 are repeated NLTYP times.

12.b.2 Type of Load (8I10)

Column 1-10--LTI, load type (see Table D.2) (for example, load type ICE is input as 14)

Column 11-20--NJL, number of total loaded joints (for example, all joints where ice load would act)

12.b.3 Loads (I10,6F10.0) (Repeat NJL times) (See Fig. B.4)

One card for loads at each loaded joint. In this option (AASHTO Load Group) specified displacements or rotations are not directly permitted. To achieve specified displacements the user must input an equivalent static force system that produces the desired displacements. All loads should be divided by proper strength reduction factor Ø of AASHTO Sec. 1.5.30(B)(2).

Column 1-10--J, number of the loaded joint

Column 11-20--FOR(1), total unfactored force in the x-direction corresponding to load type LTI

Column 21-30--FOR(2), total unfactored force in the y-direction corresponding to load type LTI.

Column 31-40--FOR(3), total unfactored force in the z-direction corresponding to load type LTI.

Column 41-50--FOR(4), total unfactored moment about the x-axis corresponding to load type LTI

Column 51-60--FOR(5), total unfactored moment about the y-axis corresponding to load type LTI.

Column 61-70--FOR(6), total unfactored moment about the z-axis corresponding to load type LTI.

B.3.1 User-Friendly Input Form

The following input form was developed by the Texas State Department of Highways and Public Transportation for use with Program PIER.

PIER PROGRAM

TABLE 1. TITLE CARD

10 20 30 40 50 60 70 80

TABLE 2. MEMBER DESCRIPTION (one card each problem)

LENGTH OF MEMBER	NO. OF SEGMENTS (MAX. 10)	① NO. OF SECTIONS TO DEFINE PIER GEOMETRY	② SEGMENT LENGTH OPTION
10	20	30	40

① Enter 1 for a prismatic pier with a uniform cross section.
Enter 2 for a linearly tapered pier.
For a pier of arbitrary configuration, the number of sections required is equal to the number of segments the pier is divided into.

② Enter 0 or blank if all segments are the same length and 1 if the segment lengths are different.

TABLE 3. SEGMENT LENGTHS (use only if Seg. Lgth. Option = 1)

1st Seg.	2nd Seg.	3rd Seg.	4th Seg.	5th Seg.	6th Seg.	7th Seg.	8th Seg.
10	20	30	40	50	60	70	80

③ 9th Seg.	③ 10th Seg.
10	20

③ If the number of segments ≤ 8, the 2nd card for this table is not used.

TABLE 4. SUPPORT CONDITIONS (one card each problem)

TOTAL NO. OF CONSTRAINED JOINTS (SUPPORTS)	TOTAL NO. OF JOINTS WITH SPRINGS
10	20

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TABLE 5. RELEASE CODE FOR SUPPORTS (number of cards according to Table 4)

LOCATION OF SUPPORT (JOINT NO.)	④ RELEASE CODE TRANSLATION X Y Z	ROTATION X Y Z
10	15	20

④ Enter 1 to fix a support against translation in the x, y, or z direction, or rotation about the x, y, or z axis. See fig. B.2 for orientation of axes.

TABLE 6. SPRING VALUES (number of cards according to table 4)

Joint Number	STIFFNESS OF TRANSLATIONAL SPRING			STIFFNESS OF ROTATIONAL SPRING		
	In X direc.	In Y direc.	In Z direc.	about X axis	about Y axis	about Z axis
10	20	30	40	50	60	70

TABLE 7. MATERIAL PROPERTIES (one card each problem)

Concrete Comp. strength f'c	Steel Yield Stress fy	⑤ Concrete Srength Reduc. Factor	⑥ Initial Concrete Modulus Ec	Steel Modulus Es	⑦ Ultimate Concrete Strain Eu	⑧ Creep Factor Beta
10	20	30	40	50	60	70

⑤ Normal value is .85

⑥ $Ec = Wc^{1.5}\, 33 \sqrt{f'c}$ (If units are lbs. and inches)

⑦ Normal value is .0038, If left blank, program assumes $Eu = 1.9(1.0 + Beta) \left(\frac{\text{Conc. Str.}}{\text{Reduc. Factor}}\right) (2.0) \left(\frac{f'c}{Ec}\right)$

⑧ May be left blank. To consider creep effects, enter value from table B.1

TABLE 8. SECTION CARD (one card each problem)

Field	Description	Columns
(9)	Section Type left justified	1–8
(10)	Status of Concrete	20
(11)	Status of Reinf. Steel	30
(12)	Fiber Properties Print Parameter	40

(9) Enter section type. (Enter SOLID, HOLLOW, OVAL or CIRHOLLO. See fig. B.3)

(10) Enter 0 or blank for unconfined concrete, or 1 for confined concrete. To use confined option, user must ensure enough lateral ties are provided to meet the equivalent of ACI Building Code 318-83 Appendix A3 Sec. A.4.4 for compression members.

(11) Enter 0 or blank if reinf. steel is modeled as solid rings. In this case the program will automatically generate the steel fibers. Enter 1 if individual steel fibers are to be input by the user.

(12) Enter 0 or blank to suppress print out of fiber properties, or 1 for full print out.

TABLE 9. GEOMETRY OF THE SECTION & TABLE 10. REINFORCING STEEL

(Repeat Tables 9 thru 10 for the number of sections required to define pier geometry according to Table 2)

TABLE 9.a. For SOLID Section Type

Dimensions Along Z Axis	Dimensions Along X Axis
Columns 1–10	Columns 11–20

TABLE 9.b. For HOLLOW Section Type (with or without Interior Walls)

Dimension Along Z Axis	Thickness of Wall parallel to Z Axis	Dimension Along X Axis	Thickness of Wall parallel X Axis
Columns 1–10	Columns 11–20	Columns 21–30	Columns 31–40

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9.b.1. Status of Interior Walls

Enter NO (Left Justified) if there are no interior Walls. If there are interior Walls, enter YES and use cards 9.b.2 and 9.b.3.

1

9.b.2. Description of Interior Walls

(13) Orientation of Interior Walls (Left Justified)

10

Number of Interior Walls (Max. 5)

30

(13) Enter VERTICAL if Walls are parallel to the X Axis or HORIZONTAL if Walls are parallel to the Z Axis.

9.b.3. Geometry of Interior Walls

Distance From Centroid	Wall Thickness	Distance From Centroid	Wall Thickness	Distance From Centroid	Wall Thickness	Distance From Centroid	Wall Thickness
10	20	30	40	50	60	70	80

(14) Distance From Centroid	(14) Wall Thickness
10	20

(14) If the number of Interior Walls is ≤ 4, the 2nd card for this table is not used.

TABLE 9.c. For OVAL Section Type (also used for (15) solid circular sections)

Dimension Along Z Axis	Dimension Along X Axis
10	20

(15) A solid circular section is a special case of the OVAL section type. It is defined by entering the diameter of the section as the dimension along the X Axis and the diameter + .01 as the dimension along the Z Axis.

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TABLE 9.d. For CIRHOLLO (circular hollow) Section Type

Diameter	(16) Wall Thickness

Columns: 1, 10, 20

(16) Wall thickness must be less than one-half the diameter.

TABLE 10.a. REINFORCING STEEL MODELED AS RINGS (use only if Status of Reinf. Steel = 0 in Table 8)

10.a.1.

Number of Rings of Steel (Max. 3)	(17) 1st Ring Area of Steel Parallel to Z Axis (STT)	 X Axis (STS)	(17) 2nd Ring Area of Steel Parallel to Z Axis (STT)	 X Axis (STS)	(17) 3rd Ring Area of Steel Parallel to Z Axis (STT)	 X Axis (STS)

Columns: 10, 20, 30, 40, 50, 60, 70

(17) See Fig. B.3 For SOLID, HOLLOW & OVAL sections, STT and STS are areas of Steel per column face. The total area of steel in the section would be 2 times the sum of STT and STS. For circular solid (special case of OVAL) and CIRHOLLO sections STT=0 and STS is one-half the total area of steel.

10.a.2. CONCRETE COVER (edge of concrete to center of steel)

1st Ring Cover to STS (GAMMA Z)	 STT (GAMMA X)	2nd Ring Cover to STS (GAMMA Z)	 STT (GAMMA X)	3rd Ring Cover to STS (GAMMA Z)	 STT (GAMMA X)

Columns: 10, 20, 30, 40, 50, 60

10.a.3. STEEL IN INTERIOR WALLS (use this card only for HOLLOW sections with interior walls)

Enter the area of steel for each interior wall in the same sequence as the walls were defined in TABLE 9.b.

1st Wall	2nd Wall	3rd Wall	4th Wall	5th Wall

Columns: 10, 20, 30, 40, 50

TABLE 10.b. STEEL FIBERS INPUT BY USER (use only if status of reinf. steel = 1 in TABLE 8)

10.b.1

Number of Reinf. Bars (Max. 200)

Columns 8–10

10.b.2. AREA AND LOCATION OF REINFORCING BARS (use as many cards as necessary to define all reinf. bars. There are 2 bar description per card.)

Z Coordinate ~~Area~~	X Coordinate ~~Z Coordinate~~	Area ~~X Coordinate~~	Z Coordinate ~~Area~~	X Coordinate ~~Z Coordinate~~	Area ~~X Coordinate~~
10	20	30	40	50	60

TABLE 11. TYPE OF LOADING (one card per problem)

Loading System Option (column 10)

Enter 0, and the program will analyze the pier without automatically factoring loads or considering AASHTO group loadings. For this type analysis use TABLE 12.c.

Enter a value equal to the number of AASHTO group loads to be considered (max. 10), and the program will apply load factors and group the loads according to AASHTO group loading criteria. See Table B.2. For this type analysis use TABLE 12.b.

TABLE 12.a. JOINT LOADS (use only if value in TABLE 11. = 0)

12.a.1.

Represents the number of ~~load~~ different load groups. ~~and~~ All pier behavior is cumulative from one load case to the next.

Number of Load Cases (column 10)

Print (17) Parameter (column 20)

(17) This value is the frequency of the printout of the incremental analysis. (Example: If this value is 5, the results of every 5th increment of applied load will be printed.)

12.a.2.

(20)

Number of Loaded Joints	Number of Load Increments
10	20

(20) A minimum of 30 load increments should be used to obtain an accurate solution.

12.a.3. (one card for each loaded joint)

(21) Incremental force in the / (21) Incremental moment about the

Joint Number	X Direction	Y Direction	Z Direction	X AXIS	Y AXIS	Z AXIS
10	20	30	40	50	60	70

(21) The program arrives at its solution by applying incremental loads to the member. For this loading option, the total design joint loads must be divided by an appropriate number of increments (min. 30). Since this program is an ulimate design procedure, the load increments should be mulipiled by the appropriate load factors (β) and devided by the proper strength reduction factors (ϕ).

This card may also be used to specify an incremental displacement or rotation of the joint. If the joint was specified as a support in TABLE 5, and has a 1 specified in the release code for a degree of freedom, and if a value is input for that degree of freedom, the user is specifing an incremental displacement or rotation, not a load.

TABLE 12.b. JOINT LOADS (use only if value in TABLE 11. > 0)

12.b.1.

(24) AASHTO Load Factor Design Group Combination to be considered.

(22) Print Parameter	(23) Number of Load Types	1st	2nd	3rd	4th	5th	6th
10	20	30	40	50	60	70	80

(22) This value is the frequency of the printout of the results of the incremental analysis. (Example: If this value is 5, the results of every 5th increment will be printed.)

(23) The total number of different load types to be input. (D, L, ICE, etc) (maximum of 14)

(24) If more than 6 group load combinations are to be considered, begin a second card.

REPEAT CARDS 12.b.2. & 12.b.3. FOR EACH LOAD TYPE.

12.b.2.

25 Load Type	26 Number of Loaded Joints
10	20

25 See Table B.2 (Example: load type ICE is input as 14)

26 Total number of joints where load type acts

12.b.3. (one card for each loaded joint)

Joint Number	27 Total Unfactored Force in the			27 Total Unfactored moment about the		
	X Direction	Y Direction	Z Direction	X AXIS	Y AXIS	Z AXIS
10	20	30	40	50	60	70

27 The program arrives at its solution by applying incremental loads to the member. For this loading option, enter the total unfactored design loads. The program will then automatically divide these loads by 50 to establish the load increment. The program will also automatically multiply these loads by the load factors indicated in Table B.2 ; however the user must divide the design loads by the appropriate strength reduction factors (ϕ).

In the AASHTO Load Group option, specified displacements or rotations are not directly permitted. To achieve specified displacements the user must input an equivalent static force system that produces the desired displacements.

B.4 Computer Output

PIER computes the load-deflection analysis of a member. The output consists of:

B.4.1 Title and echo print of all section and member properties.

B.4.2 Echo print of loading conditions.

B.4.3 Printout of incremental results which includes forces and displacements at each joint, until all load increments have been completed, or there is an assumed stability or material failure. If there is an assumed material failure, the results just before failure and the location of the failure will be printed. The program will signal the appropriate level of appropriateness of the design only when the AASHTO load factor option of load application is used.

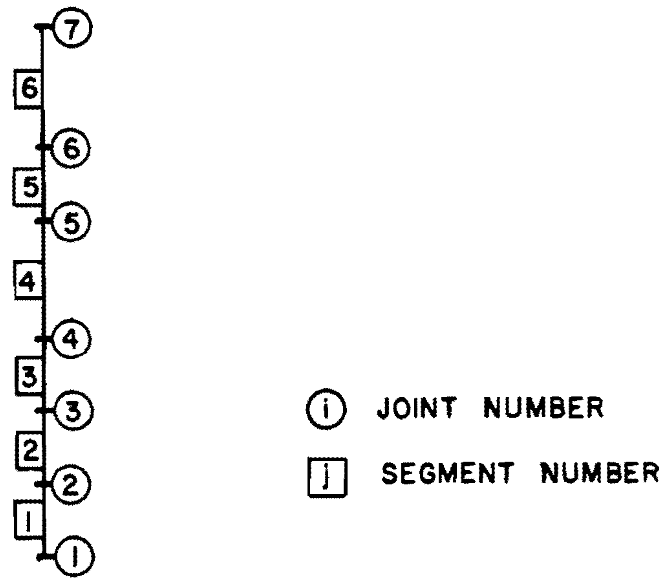


Fig. B.1 Numbering of segments and joints

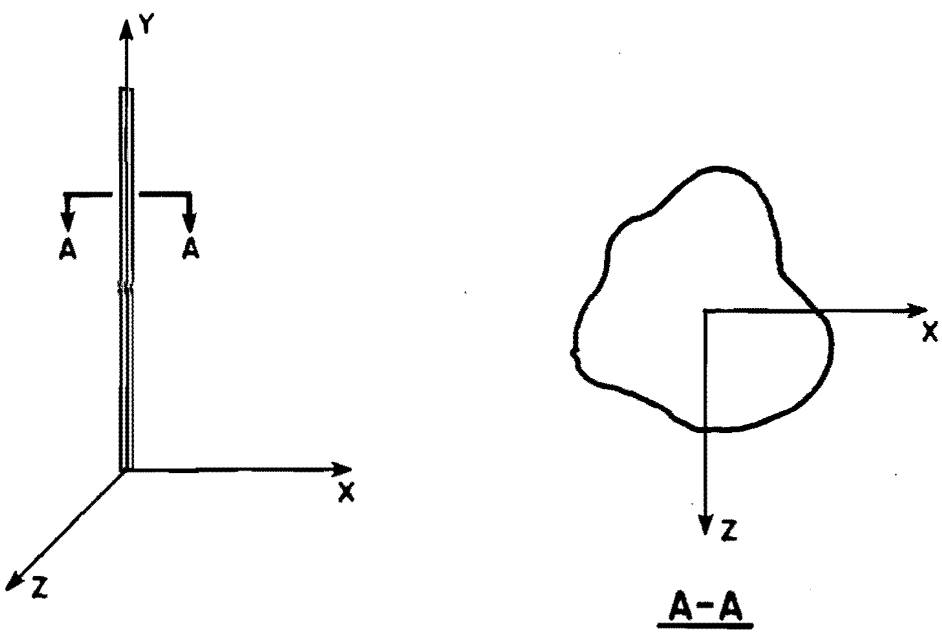
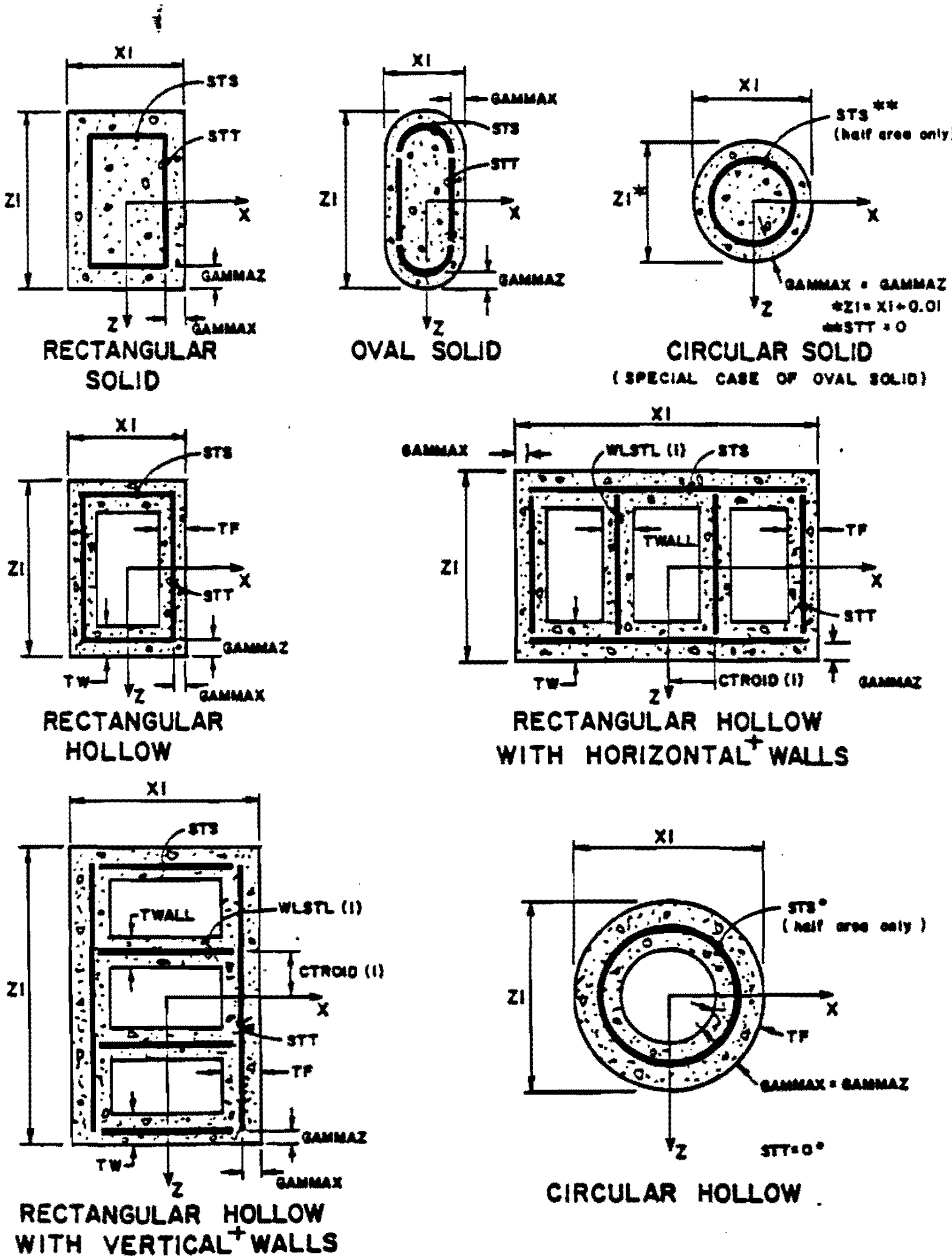


Fig. B.2 Location of axes for section properties



+ Interior walls are not oriented (vertical and horizontal) as seen in this figure because of the global coordinate system used in PIER.

Fig. B.3 Section characteristics

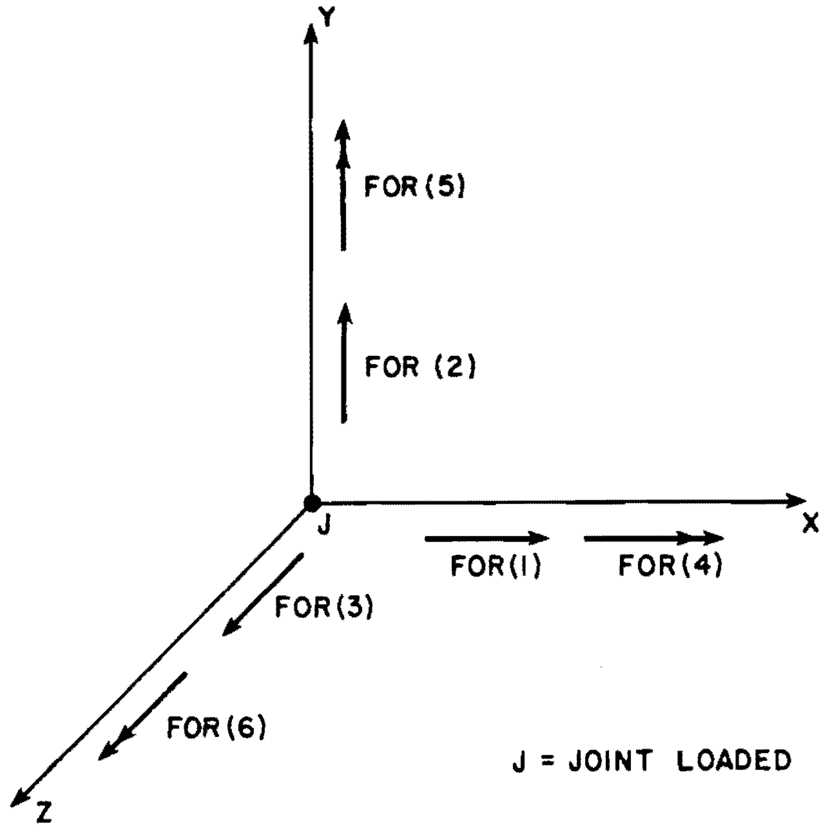


Fig. B.4 Positive convention of forces

TABLE B.1 Creep Factor, BETA

TIME (days)	BETA*
7	0.88
30	1.46
60	1.75
90	1.92
120	2.04
150	2.13
180	2.21
365 (1 year)	2.50
548	2.68
730 (2 y)	2.80
1095	3.91
1460 (4 y)	4.04
1825 (5 y)	4.11
2190 (6 y)	4.17
2555 (7 y)	4.21
2920 (8 y)	4.24
3285 (9 y)	4.27
3650 (10 y)	4.30
5475 (15 y)	4.39
7300 (20 y)	4.45
9125 (25 y)	4.49
10950 (30 y)	4.53

* Reference: "Analysis of Reinforced Concrete Columns Under Sustained Load," by V. Chovichien, M. J. Gutzwiller, and R. H. Lee, <u>ACI Journal</u>, October 1973, pp. 692-699.

Table B.2 Load Groups and Types of Load

ARRAY BFAC (Beta Coefficient)

Loading Group: Program	Loading Group: AASHTO	1 D*	2 D*	3 L+I	4 CF	5 E*	6 E*	7 B	8 SF	9 W	10 WL	11 LF	12 R+S+T	13 EQ	14 ICE	γ Values
1	I	β_d=1.0	β_d=0.75	1.67	1.0	1.3	0.5	1.0	1.0	0	0	0	0	0	0	1.3
2	IA	↓	↓	2.20	0	0	0	0	0	0	0	0	0	0	0	1.3
3	II	↓	↓	0	0	1.3	0.5	1.0	1.0	1.0	0	0	0	0	0	1.3
4	III	↓	↓	1	1	↓	↓	1.0	1.0	0.3	1.0	1.0	0	0	0	1.3
5	IV	↓	↓	1	1	↓	↓	1.0	1.0	0	0	0	1.0	0	0	1.3
6	V	↓	↓	0	0	↓	↓	1.0	1.0	1.0	0	0	1.0	0	0	1.25
7	VI	↓	↓	1	1	↓	↓	1.0	1.0	0.3	1.0	1.0	1.0	0	0	1.25
8	VII	↓	↓	0	0	↓	↓	1.0	1.0	0	0	0	0	1.0	0	1.3
9	VIII	↓	↓	1	1	↓	↓	1.0	1.0	0	0	0	0	0	1.0	1.3
10	IX	↓	↓	0	0	1.3	0.5	1.0	1.0	1.0	0	0	0	0	1.0	1.20

D = Dead Load
L = Live Load
I = Live Load Impact
E = Earth Pressure
B = Buoyancy

W = Wind Load on Structure
WL = Wind Load on Live Load (100 lbs. per lin. ft.) (1458 N/m)
LF = Longitudinal Force from Live Load

CF = Centrifugal Force
R = Rib Shortening
S = Shrinkage
T = Temperature
EQ = Earthquake
SF = Stream Flow Pressure
ICE = Ice Pressure

* See AASHTO manual Section 1.2.22; only 1 value of dead load and 1 value of earth pressure may be used per problem.

B.5 Listing of PIER

```
C        DATA SET T2241?1    AT LEVEL 006 AS OF 01/20/84
      PROGRAM PIER
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      CHARACTER*4,ASHTO(13),IHOLL,IHOLL1,IN1,ITITLE(20),ITYPE(2),
     *            ISOLID,IOVAL,ICHOLL,INC1,IV1,ISOLI1,IOVAL1,ICHOL1,
     *            GREE(27),ICR(3),INO,IN,IV,ICODE
      DIMENSION STGB(6,6),STAA(6,6),STAB(6,6),STBA(6,6),AMAT(3,3),
     1FOR(6),CU(66),FORA(6,10),FORB(6,10),DFORA(6,10),DFORB(6,10),EPS(10
     2,10),FIY(10,10),FIZ(10,10),EZC(200),EZS(200),CONFZ(200,10),
     3CONFX(200,10),CFA(200,10),STLZ(200,10),STLX(200,10),STLA(200,10),
     4SMAT(3,3,100),STK(720),PFOR(66),PDIS(66),AR(11),AKPX(11),AKPY(11),
     5AKPZ(11),AKX(11),AKY(11),AKZ(11),RCT(6,6),SA(6,6,11),SB(6,6,10),
     6SC(6,6,10),DUA(6),DUB(6),SKAA(6,6,10),SKAB(6,6,10),SKBA(6,6,10),
     7SKBB(6,6,10),U(66),AMP(10),CAL(10),
     8GAMMA(10),ZFAC(10,14)
      COMMON /ONE/ ZI,TP,XI,TA
      COMMON /TWO/ STRDIS(5),YWALL(5),NW,NWX,NWY
      COMMON /THREE/ ICR,INO,IN,IV
      COMMON /FOUR/ STT(3),STS(3),GAMMAZ(3),GAMMAX(3),WLSTL(5),NRING
      COMMON /FIVE/ ISOLID,IHOLL,IOVAL,ICHOLL
      COMMON /L1/ CYL,AK,EC,END,EU
      COMMON /L2/ FY,EY,RATIO(10)
      COMMON /STP/ DA11,DA12,DA13,DA21,DA22,DA23,DA31,DA32,DA33
      COMMON /SUP/ IPRNT
      COMMON /SAFE/ KCTYPE,KSTTPE,KPY,KNX,NX,NY,KV,RM,NI
      COMMON/ZSING/SUMF(66)
      COMMON/F1/ DISC,NLOAD,NINC,NSEG,NSECT
      COMMON /D1/ ASHTO
      COMMON /D2/ GAMMA,ZFAC,IPAS
      COMMON/F2/ SIGS,JLOAD,JINC,JSEG,JSECT
      DATA GREE/'ASSH','TO R','EQUI','DAI ','   G','AMMA',' (60*',
     1          'D*3L','*(L*','D)*A','C+CP','*95*','E*SE','*E*&',
     2          'S-S*','+SW*','A*SW','L*WL','*BL*','LP*B','R*(R',
     3          '*S-T','D*ES','WEQ ','*SIC','E*IC','E) '/
      DATA INC1/'NO  '/,IN1/'HCAI'/,IV1/'VERT'/,ISOLI1/'SOLI'/,
     *     IHOLL1/'HOLL'/,IOVAL1/'OVAL'/,ICHOL1/'CIRN'/
      DATA IONE/1/,IZERO/0/
      JT=1
      CALL TICTOC (JT)
      INO=INC1
      IN=IN1
      IV=IV1
      ISOLID=ISOLI1
      IHOLL=IHOLL1
      IOVAL=IOVAL1
      ICHOLL=ICHOL1
      KCTYPE=2
      KSTTPE=0
      READ (*,150,END=10000)(ITITLE(I),I=1,20)
 150  FORMAT(2X,20A4)
10000 PRINT 150,(ITITLE(I),I=1,20)
 1504 FORMAT('1',4X,20A4)
      READ (*,100,END=10001)AL,NSEG,NSECT,I,
 100  FORMAT (62,F10.0,3I10)
10001 IF(NSECT.EQ.0) NSECT=1
      PRINT 200,AL,NSEG,NSECT
 200  FORMAT ('0',6X,'LENGTH OF MEMBER   =',F10.2,/,
     1            3X,'NUMBER OF  SEGMENTS=',I5,/,
     2            3X,'NUMBER OF DIFFERENT TYPE OF SECTIONS=',I5)
      AN=NSEG
      IF(I.EQ.1) READ (*,101,END=10002)(CAL(I),I=1,NSEG)
10002 IF(I.EQ.1) GO TO 105
      DO 110 I=1,NSEG
 110  CAL(I)=AL/AN
 105  NIM=NSEG-1
      NJ=NSEG+1
      READ (*,101,END=10003)NCJ,NJSP
 101  FORMAT (2I10)
10003 PRINT 201,NCJ,NJSP
 201  FORMAT(///,5X,'NUMBER OF JOINTS WITH CONSTRAINED',/,
     1            3X,' OR SPECIFIED DISPLACEMENTS        =',I5,/,
     2            3X,'NUMBER OF JOINTS WITH SPRINGS      =',I5)
      DO 1 I=1,NJ
      AKX(I)=0.
      AKY(I)=0.
      AKZ(I)=0.
      AKPX(I)=0.
      AKPY(I)=0.
      AKPZ(I)=0.
 1    JR(I)=0
      IF (NCJ.EQ.0) GO TO 4
      PRINT 202
 202  FORMAT(///,1X,'JOINT NO.',4X,'RELEASE CODE')
      DO 2 I=1,NCJ
      READ (*,101,END= 2    )J,JR(J)
 2    PRINT 203,J,JR(J)
 203  FORMAT(4X,I5,7X,I10)
 4    IF(NJSP.EQ.0) GO TO 5
      PRINT 204
 204  FORMAT(///,16X,'**********TRANSLATIONAL SPRINGS***********',
     15X,'************ROTATIONAL SPRINGS************',/,
     23X,'JOINT NO.',6X,'X-DIR',10X,'Y-DIR',10X,'Z-DIR',7X,
     3'AROUND X-AXIS',2X,'AROUND Y-AXIS',2X,'AROUND Z-AXIS')
      DO 3 I=1,NJSP
      READ (*,102,END= 3    )J,AKX(J),AKY(J),AKZ(J),AKPX(J),AKPY(J),AKPZ(
     *J)
 3    PRINT 205,J,AKX(J),AKY(J),AKZ(J),AKPX(J),AKPY(J),AKPZ(J)
 205  FORMAT(4X,I5,6X,3(1PE12.5,3X),7X,3(1PE12.5,3X))
 102  FORMAT(8X,I10,7F10.0)
 5    READ (*,103,END=10004)CYL,FY,AK,EC,EY,EU,BETA
 103  FORMAT (2X,8F10.0)
10004 END=(1.0+BETA)*2.*AK*2.0*CYL/EC
      IF(EU.LE.0.0) EU=1.9*END
      IF(EU.LE.END) PRINT 6100
      IF(EU.LE.END) STOP 8200
 6100 FORMAT( '0',3X,'ULTIMATE STRAIN IS NOT GREATER THAN STRAIN',
     1                ' AT MAXIMUM STRESS')
      PRINT 206,CYL,FY,AK,EC,EY,EU,BETA
 206  FORMAT(///,3X,'CONCRETE STRENGTH           =',F10.2,/,
     1            3X,'STEEL YIELD POINT           =',F10.2,/,
     2            3X,'STRENGTH REDUCTION FACTOR   =',F10.2,/,
     3            3X,'CONCRETE MODULUS            =',1PE12.4,/,
     4            3X,'STEEL MODULUS               =',1PE12.4,/,
     5            3X,'CONCRETE ULTIMATE STRAIN    =',1PE12.4,/,
     6            3X,'CRUSH FACTOR                =',F10.2)
      READ (*,301,END=10005)ITYPE,ICON,ISTAT,IPRNT
 301  FORMAT(2X,2A4,2X,3I10)
10005 PRINT 207,ITYPE
 207  FORMAT(///,3X,'THE PIER SECTION IS',2X,2A4,//)
      IF(ICON.EQ.0) PRINT 3333
      IF(ICON.EQ.1) PRINT 3334
 3333 FORMAT('0',2X,'CONCRETE FIBERS WILL BE MODELLED AS',
     1' UNCONFINED CONCRETE')
 3334 FORMAT('0',2X,'CONCRETE FIBERS WILL BE MODELLED AS',
     1' CONFINED CONCRETE')
      IF(ISTAT.EQ.IZERO) PRINT 31_
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      IF(ISTAT.EQ.IONE) PRINT 213
 212  FORMAT('0',2X,'STEEL FIBERS ARE GENERATED BY THE PROGRAM')
 213  FORMAT('0',2X,'STEEL FIBERS ARE READ IN BY THE USER')
      IF(IPRNT.EQ.IZERO) PRINT 1311
 1311 FORMAT('0',2X,'PRINT OUT OF FIBER PROPERTIES',
     1' WILL BE SUPPRESSED')
      DO 6 JAY=1,NSECT
      PRINT 209,JAY
 209  FORMAT('0',2X,'SECTION TYPE',I2)
      IF(ITYPE(1).EQ.ISOLID) GO TO 151
      IF(ITYPE(1).EQ.IHOLL) GO TO 152
      IF(ITYPE(1).EQ.IOVAL) GO TO 151
      IF(ITYPE(1).EQ.ICHOLL) GO TO 156
      PRINT 187
 187  FORMAT ('0',2X,'MISTAKE MADE IN INPUT OF SECTION TYPE')
      STOP 3000
 151  READ (*,103,END=10006)ZI,XI
10006 PRINT 183,ZI,XI
 183  FORMAT('0',2X,'Z-DIMENSION=',F10.2,/,
     1              3X,'X-DIMENSION=',F10.2)
      GO TO 402
 156  READ (*,103,END=10007)ZI,TP
10007 PRINT 164,ZI,TP
 164  FORMAT('0',2X,'DIAMETER OF SECTION=',F10.2,/,
     1              3X,'WALL THICKNESS     =',F10.2)
      GO TO 402
 152  READ (*,103,END=10008)ZI,TP,XI,TW
10008 PRINT 208,ZI,TP,XI,TW
 208  FORMAT('0',2X,'Z-DIMENSION=',F10.2,/,
     1              3X,'FLANGE THICKNESS=',F10.2,/,
     2              3X,'X-DIMENSION=',F10.2,/,
     3              3X,'WEB THICKNESSS  =',F10.2)
      READ (*,301,END=10009)ICODE
10009 IF(ICODE.EQ.INO) GO TO 402
      READ (*,303,END=10010)IOR,NW
 303  FORMAT(A1,3A4,2X,I10)
10010 READ (*,103,END=10011)(CTROID(J),TWALL(J),J=1,NW)
10011 PRINT 211,NW,IOR
 211  FORMAT(///,3X,'NUMBER OF INNER WALLS=',I5,/,
     1              3X,'WALLS ARE ORIENTED',2X,3A4)
      PRINT 640
 640  FORMAT(//,3X,'NO.',5X,'Z OR X CENTROID',4X,'THICKNESS')
      DO 405 J=1,NW
 405  PRINT 650,J,CTROID(J),TWALL(J)
 650  FORMAT(1X,I5,5X,F10.2,6X,F10.2)
 402  CONTINUE
      IF(ITYPE(1).EQ.ISOLID)CALL SOLSUR(CONFZ(1,JAY),CONFX(1,JAY),
     1CPA(1,JAY),NFIB)
      IF(ITYPE(1).EQ.IHOLL)CALL OSCRIB(CONFZ(1,JAY),CONFX(1,JAY),
     1CPA(1,JAY),ICODE,NFIB)
      IF(ITYPE(1).EQ.IOVAL)CALL OSCRIB(CONFZ(1,JAY),CONFX(1,JAY),
     1CPA(1,JAY),NFIB)
      IF(ITYPE(1).EQ.ICHOLL) CALL CIFSCR(CONFZ(1,JAY),CONFX(1,JAY),
     1CPA(1,JAY),NFIB)
      ZIZ=0.0
      XIX=0.0
      DO 1001 KAY=1,NFIB
      ZIZ=ZIZ+CONFX(KAY,JAY)**2*CPA(KAY,JAY)
 1001 XIX=XIX+CONFZ(KAY,JAY)**2*CPA(KAY,JAY)
      PRINT 1002,ZIZ,XIX
 1002 FORMAT('0',2X,'MOMENT OF INERTIA ABOUT Z-AXIS=',1PE15.3,/,
     1              3X,'MOMENT OF INERTIA ABOUT X-AXIS=',1PE15.3,/)
      IF(ISTAT.EQ.IZERO) GO TO 307
      IF(ISTAT.EQ.IONE) GO TO 700
 307  READ (*,102,END=10012)NRING,(STT(I),STS(I),I=1,NRING)
10012 PRINT 705,NRING
 705  FORMAT(///,7X,'NUMBER OF STEEL RINGS=',I5,/,3X,
     1'NO.',5X,'AREA OF END STEEL',4X,'AREA OF SIDE STEEL')
      PRINT 706,(I,STT(I),STS(I),I=1,NRING)
 706  FORMAT(1X,I3,7X,F10.2,14X,F10.2)
      READ (*,102,END=10013)(GAMMAZ(J),GAMMAX(J),J=1,NRING)
10013 PRINT 710
 710  FORMAT(//,3X,'NO.',5X,'Z COVER',5X,'X COVER')
      PRINT 715,(K,GAMMAZ(K),GAMMAX(K),K=1,NRING)
 715  FORMAT(1X,I3,5X,F10.2,2X,F10.2)
      IF(ITYPE(1).EQ.IHOLL.AND.ICODE.NE.INO) GO TO 255
      GO TO 502
 255  READ (*,102,END=10014)(WLSTL(J),J=1,NW)
10014 PRINT 725
 725  FORMAT(///,3X,'NO.',5X,'AREA OF STEEL IN WALL')
      PRINT 730,(L,WLSTL(L),L=1,NW)
 730  FORMAT(1X,I3,12X,F10.2)
 502  CONTINUE
      IF(ITYPE(1).EQ.ISOLID)CALL SSCRIB(STLZ(1,JAY),STLX(1,JAY),
     1STLA(1,JAY),ICODE,NSTOT,ITYPE(1))
      IF(ITYPE(1).EQ.IHOLL) CALL SSCRIB(STLZ(1,JAY),STLX(1,JAY),
     1STLA(1,JAY),ICODE,NSTOT,ITYPE(1))
      IF(ITYPE(1).EQ.IOVAL) CALL OSSTL(STLZ(1,JAY),STLX(1,JAY),
     1STLA(1,JAY),NSTOT)
      IF(ITYPE(1).EQ.ICHOLL) CALL CIFSTL(STLZ(1,JAY),STLX(1,JAY),
     1STLA(1,JAY),NSTOT)
      GO TO 6
 302  CALL RECE(STLZ(1,JAY),STLX(1,JAY),STLA(1,JAY),NSTOT)
 6    CONTINUE
      IF(NSECT-2) 7,11,17
 7    DO 8 I=2,NSEG
      DO 9 J=1,NFIB
      CONFZ(J,I)=CONFZ(J,1)
      CONFX(J,I)=CONFX(J,1)
 9    CPA(J,I)=CPA(J,1)
      DO 10 J=1,NSTOT
      STLZ(J,I)=STLZ(J,1)
      STLX(J,I)=STLX(J,1)
 10   STLA(J,I)=STLA(J,1)
 8    CONTINUE
      GO TO 17
 11   DO 12 J=1,NFIB
      CONFZ(J,NSEG)=CONFZ(J,2)
      CONFX(J,NSEG)=CONFX(J,2)
 12   CPA(J,NSEG)=CPA(J,2)
      DO 13 J=1,NSTOT
      STLZ(J,NSEG)=STLZ(J,2)
      STLX(J,NSEG)=STLX(J,2)
 13   STLA(J,NSEG)=STLA(J,2)
      DO 14 I=2,N1
      AI=I-1
      AN=N1
      DX=AI/AN
      DY=1.-DX
      DO 15 J=1,NFIB
      CONFZ(J,I)=DY*CONFZ(J,1)+DX*CONFZ(J,NSEG)
      CONFX(J,I)=DY*CONFX(J,1)+DX*CONFX(J,NSEG)
 15   CPA(J,I)=DY*CPA(J,1)+DX*CPA(J,NSEG)
      DO 16 J=1,NSTOT
      STLZ(J,I)=DY*STLZ(J,1)+DX*STLZ(J,NSEG)
      STLX(J,I)=DY*STLX(J,1)+DX*STLX(J,NSEG)
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 15   STLA(J,I)=DY*STLA(J,1)+DX*STLA(J,NSEG)
 16   CONTINUE
 17   NOF=6*NJ
      DO 799 I=1,NSEG
      AST=0.0
      AG=0.0
      DO 601 J=1,NFIB
 601  AG=AG+CFA(J,I)
      DO 602 J=1,NSTOT
 602  AST=AST+STLA(J,I)
      RATIO(I)=AST/AG
 799  CONTINUE
      LGP=0
      READ (5,101,END=10015)NLFDG
10015 IF(NLFDG.GT.0) ILF=1
      IF(NLFDG.LE.0) ILF=0
 5080 DO 18 I=1,NOF
      U(I)=0.
 18   DU(I)=0.
      DO 42 I=1,NFIB
 42   EZC(I)=0.
      DO 43 I=1,NSTOT
 43   EZS(I)=0.
      DO 41 I=1,NSEG
      CALL CMSTP(COMPZ(1,I),COMPX(1,I),CFA(1,I),STLZ(1,I),
     1STLX(1,I),STLA(1,I),EZC,EZS,NFIB,NSTOT,I,ICON)
      DO 44 J=1,13
      JJ=13*(I-1)+J
      BMAT(1,1,JJ)=DA11
      BMAT(1,2,JJ)=DA12
      BMAT(1,3,JJ)=-DA13
      BMAT(2,1,JJ)=DA21
      BMAT(2,2,JJ)=DA22
      BMAT(2,3,JJ)=-DA23
      BMAT(3,1,JJ)=-DA31
      BMAT(3,2,JJ)=-DA32
      BMAT(3,3,JJ)=DA33
 44   CALL MSINV(BMAT(1,1,JJ),3,3)
 41   CONTINUE
      DO 45 I=1,NSEG
      AXP(I)=0.0
      DO 45 J=1,13
      EPS(J,I)=0.
      FIY(J,I)=0.
 45   FIZ(J,I)=0.
      DO 19 J=1,NSEG
      DO 20 I=1,6
      FORA(I,J)=0.
      DFORA(I,J)=0.
      DFORB(I,J)=0.
 20   FORB(I,J)=0.
      DO 19 I=1,13
      EPS(I,J)=0.
      FIY(I,J)=0.
 19   FIZ(I,J)=0.
      IF(NLFDG.GT.0)CALL   CDSIN(NLFDG,ILF,NOF,NLOAD,JPRNT,LGP)
      IF(NLFDG.LE.0) READ (5,101,END=10016)NLOAD,JPRNT
10016 IF(JPRNT.EQ.0)  JPRNT=1
      IF(NLFDG.GT.0) PRINT 5320,ASWTC(LGP),(GPEC(I),I=1,27),GAMMA(LGP),
     1 GFAC(LGP,IPP6),(EFAC(LGP,I),I=1,5),
     2 (EFAC(LGP,I),I=7,16)
 5320 FORMAT('1',20X,'**********************',/,21X,'*  LOAD GROUP',
     1        4X,'  *',/,21X,'**********************',///,5X,27X,//,
     2        5X,'GAMMA VALUE =',F5.2,//,5X,'EXT. VALUES:',2X,'ED =',
     3 F5.2,  /,22X,'EL =',F5.2,/,22X,'LC =',F5.2,/,22X,'BE =',F5.2,/,
     4        22X,'BS =',F5.2,/,22X,'SS =',F5.2,/,22X,'TM =',F5.2,/,
     5        22X,'BML=',F5.2,/,22X,'IC =',F5.2,/,20X,'FR =',F5.2,/,
     6        22X,'SEC=',F5.2,/,19X,'EICE=',F5.2)
      PRINT 634,NLOAD
 634  FORMAT(///,5X,'NUMBER OF LOAD CASES=',I3)
      BIGC=1000.
      BIGS=-1000.
      DO 1000 JAY=1,NLOAD
      KPRNT=1
      PRINT 636,JAY
 636  FORMAT('1',15(/),20X,'*** LOAD CASE ',I5,' ***')
      DO 30 I=1,NOF
      PFOR(I)=0.
 30   POIS(I)=0.
      IF(NLFDG.GT.0) GO TO 5000
      READ (5,101,END=10017)NLJ,NLINC
10017 PRINT 637,NLJ,NLINC
 637  FORMAT(///,3X,'NUMBER OF LOADED JOINTS  =',I5,/,
     1              3X,'NUMBER OF LOAD INCREMENTS=',I5)
      PRINT 638
 638  FORMAT(///,4X,'JOINT NO.',4X,'FORCE-X',6X,'FORCE-Y',6X,'FORCE-Z',
     16X,'MOMENT-X',6X,'MOMENT-Y',6X,'MOMENT-Z')
      DO 31 I=1,NLJ
      READ (5,102,END=10018)J,(FOR(L),L=1,6)
10018 PRINT 639,J,(FOR(L),L=1,6)
 639  FORMAT(4X,I3,6X,3(1X,1PE10.3),3(2X,1PE10.3))
      JJ=6*J-6
      JMASK=1000000
      K=JR(J)
      DO 31 L=1,6
      JJ=JJ+1
      JMASK=JMASK/10
      K=K-JMASK
      IF(K) 32,33,33
 32   PFOR(JJ)=FOR(L)
      K=K+JMASK
      GO TO 31
 33   POIS(JJ)=FOR(L)
 31   CONTINUE
      GO TO 5050
 5000 DO 5010 IJ=1,NOF
      PFOR(IJ)=SUMP(IJ)
      IF(JAY.EQ.2) PFOR(IJ)=5.0*SUMP(IJ)/12.0
 5010 CONTINUE
      NLINC=50-10*JAY
      IF(JAY.EQ.1) NLINC=50
      PRINT 6339
      DO 5015 I=1,NJ
      L2=6*I
      L1=L2-5
 5015 PRINT 639,I,(PFOR(IJ),IJ=L1,L2)
      PRINT 6341,NLINC
 6339 FORMAT('0',4X,'THE FACTORED INCREMENTAL FORCES ',
     1        'FOR THIS LOAD CASE ARE:',///,4X,'JOINT NO.',
     2        4X,'FORCE-X',6X,'FORCE-Y',6X,'FORCE-Z',6X,
     3        'MOMENT-X',6X,'MOMENT-Y',6X,'MOMENT-Z',/)
 6341 FORMAT('0',///,4X,'NUMBER OF LOAD INCREMENTS =',I3)
 5050 DO 999 IL=1,NLINC
      DO 60 ISEG=1,NSEG
      DO 60 I=1,6
      DO 60 J=1,6
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46    ST3B(I,J)=0.
      DO 50 ISECT=1,10
      JJ=(ISEG-1)*10+ISECT
      FOR(1)=DFCRS(1,ISEG)
      AI=ISECT-1
      AI=AI/9.
      DI=DAL(ISEG)/9.
      DIS=DAL(ISEG)*(1.-AI)
      FOR(2)=DFCRS(5,ISEG)-DIS*DFCRS(3,ISEG)
      FOR(3)=DFCRS(6,ISEG)+DIS*DFCRS(2,ISEG)
      CALL PSMULT(BMAT(1,1,JJ),FOR,3,3,1,1)
      EPS(ISECT,ISEG)=EPS(ISECT,ISEG)+FOR(1)
      FIY(ISECT,ISEG)=FIY(ISECT,ISEG)+FOR(2)
      FIZ(ISECT,ISEG)=FIZ(ISECT,ISEG)+FOR(3)
      DO 51 I=1,NFIB
      EZC(I)=EPS(ISECT,ISEG)*CONFX(I,ISEG)+FIY(ISECT,ISEG)
     1-CONFZ(I,ISEG)*FIZ(ISECT,ISEG)
      IF(EZC(I).LT.-EU) GO TO 6100
      IF(EZC(I).LT.BIGC)  MLOAD=JAY
      IF(EZC(I).LT.BIGC) MINC=IL
      IF(EZC(I).LT.BIGC) MSEG=ISEG
      IF(EZC(I).LT.BIGC) MSECT=ISECT
      IF(EZC(I).LT.BIGC) BIGC=EZC(I)
51    CONTINUE
      DO 52 I=1,NSTOT
      EZS(I)=EPS(ISECT,ISEG)*STLX(I,ISEG)+FIY(ISECT,ISEG)
     1-STLZ(I,ISEG)*FIZ(ISECT,ISEG)
      IF(EZS(I).GT.0.01) GO TO 6200
      IF(EZS(I).GT.BIGS) JLOAD=JAY
      IF(EZS(I).GT.BIGS) JINC=IL
      IF(EZS(I).GT.BIGS) JSEG=ISEG
      IF(EZS(I).GT.BIGS) JSECT=ISECT
      IF(EZS(I).GT.BIGS) BIGS=EZS(I)
52    CONTINUE
      CALL CMSTF(CONFX(1,ISEG),CONFX(1,ISEG),CFA(1,ISEG),
     1STLZ(1,ISEG),STLX(1,ISEG),STLA(1,ISEG),EZC,EZS,NFIB,
     2NSTOT,ISEG,ICON)
      BMAT(1,1,JJ)=DA11
      BMAT(1,2,JJ)=DA12
      BMAT(1,3,JJ)=-DA13
      BMAT(2,1,JJ)=DA21
      BMAT(2,2,JJ)=DA22
      BMAT(2,3,JJ)=-DA23
      BMAT(3,1,JJ)=-DA31
      BMAT(3,2,JJ)=-DA32
      BMAT(3,3,JJ)=DA33
      CALL PSINV(BMAT(1,1,JJ),3,3)
      DO 53 I=1,3
      DO 54 J=1,3
54    AMAT(I,J)=0.
53    AMAT(I,I)=1.
      CALL PSMULT(BMAT(1,1,JJ),AMAT,3,3,3,3)
      C=1.
      IF(ISECT.EQ.1.OR.ISECT.EQ.10) C=0.5
      STEE(2,2)=STEE(2,2)+C*DI*AMAT(1,1)
      STEE(2,3)=STEE(2,3)+C*DI*DIS*AMAT(1,2)
      ST33(1,2)=ST33(1,2)+C*DI*DIS*AMAT(1,2)
      STEE(2,5)=STEE(2,5)+C*DI*AMAT(1,3)
      ST33(2,6)=STEE(2,6)+C*DI*AMAT(1,3)
      ST33(2,3)=ST33(2,3)+C*DI*DIS*DIS*AMAT(2,2)
      ST33(1,3)=ST33(1,3)-C*DI*DIS*DIS*AMAT(3,2)
      ST33(3,6)=ST33(3,6)+C*DI*DIS*AMAT(2,3)
      ST33(3,4)=ST33(3,4)+C*DI*DIS*AMAT(2,3)
      ST33(1,1)=ST33(1,1)+C*DI*DIS*DIS*AMAT(3,3)
      ST33(1,2)=ST33(1,2)-C*DI*DIS*AMAT(2,3)
      ST33(1,4)=ST33(1,4)-C*DI*DIS*AMAT(3,3)
      ST33(2,2)=ST33(2,2)+C*DI*AMAT(2,2)
      ST33(4,5)=ST33(4,5)+C*DI*AMAT(3,3)
      ST33(4,4)=ST33(4,4)+C*DI*AMAT(3,3)
50    CONTINUE
      DO 39 I=1,6
      DO 39 J=1,6
39    ST33(J,I)=ST33(I,J)
      ST33(5,5)=1.
      CALL PSINV(ST33,6,6)
      DO 37 I=1,6
      DO 38 J=1,6
      STAA(I,J)=ST33(I,J)
38    ST33(I,J)=0.
37    ST33(I,I)=1.
      CALL PSMULT(STAA,ST33,6,6,6,6)
      TAAT=((STEE(4,4)*STEE(6,6))/ST33(2,2))*.25
      STE6(5,5)=0.4*(STEE(4,4)+STEE(6,6))
      IF(ITYPE(1).EQ.IMCLL.OR.ITYPE(1).EQ.IZ-ILL) GO TO 370
      DIM=ZI
      IF(DIM.GE.XI) DIM=XI
      ST66(5,5)=ST66(5,5)+.4*XI*ZI*DIM/((XI+ZI)*(XI*XI+ZI*ZI))
370   CONTINUE
      DO 36 I=1,6
      DO 35 J=1,3
35    STBA(I,J)=-ST33(I,J)
      STBA(I,3)=-ST33(I,3)
      STBA(I,6)=-ST33(I,6)+DAL(ISEG)*ST33(I,1)
36    STBA(I,4)=-ST33(I,4)-DAL(ISEG)*ST33(I,5)
      DO 57 J=1,6
      DO 56 I=1,3
      STAA(I,J)=-ST33(I,J)
56    STAB(I,J)=-ST33(I,J)
      STAA(5,J)=-STBA(5,J)
      STAB(5,J)=-ST33(5,J)
      STAA(6,J)=-ST33(6,J)+DAL(ISEG)*ST33(1,J)
      STAB(6,J)=-ST33(6,J)+DAL(ISEG)*ST33(1,J)
      STAA(4,J)=-STBA(4,J)-DAL(ISEG)*ST33(I,J)
57    STAB(4,J)=-ST33(4,J)-DAL(ISEG)*ST33(I,J)
      I1=ISEG
      I2=ISEG+1
      J1=3*I1-5
      J2=J1+6
      C1=(U(J2)-U(J1))/DAL(ISEG)
      J1=J1+2
      J2=J2+2
      C2=(U(J2)-U(J1))/DAL(ISEG)
      DO 58 I=1,6
      DO 59 J=1,6
59    RCT(I,J)=0.
58    RCT(I,I)=1.
      RCT(1,2)=-C1
      RCT(2,1)=C1
      RCT(2,3)=C2
      RCT(3,2)=-C2
      RCT(4,5)=-C1
      RCT(5,4)=C1
      RCT(5,6)=C2
      RCT(6,5)=-C2
      CALL MROT(STBB,RCT)
      CALL MROT(STAA,RCT)
```

```
      CALL MRCT(STAB,RCT)
      CALL MRCT(STBA,RCT)
      DAX=FORB(2,ISEG)/DAL(ISEG)
      DMX=(FORB(4,ISEG)-FORA(4,ISEG))/(2.*DAL(ISEG))
      DMZ=(FORB(6,ISEG)-FORA(6,ISEG))/(2.*DAL(ISEG))
      STBB(1,5)=STBB(1,5)+DMX
      STBB(3,5)=STBB(3,5)-DMZ
      STBB(5,1)=STBB(1,5)
      STBB(5,3)=STBB(3,5)
      STAA(1,5)=STAA(1,5)+DMX
      STAA(3,5)=STAA(3,5)-DMZ
      STAA(5,1)=STAA(1,5)
      STAA(5,3)=STAA(3,5)
      STAB(1,5)=STAB(1,5)-DMX
      STAB(3,5)=STAB(3,5)+DMZ
      STAB(5,1)=STAB(5,1)-DMX
      STAB(5,3)=STAB(5,3)+DMZ
      STBA(1,5)=STAB(5,1)
      STBA(3,5)=STAB(5,3)
      STBA(5,1)=STAB(1,5)
      STBA(5,3)=STAB(3,5)
      STBB(1,1)=STBB(1,1)+DAX
      STBB(3,3)=STBB(3,3)+DAX
      STAA(1,1)=STAA(1,1)+DAX
      STAA(3,3)=STAA(3,3)+DAX
      STAB(1,1)=STAB(1,1)-DAX
      STAB(3,3)=STAB(3,3)-DAX
      STBA(1,1)=STBA(1,1)-DAX
      STBA(3,3)=STBA(3,3)-DAX
      STBB(5,5)=STBB(5,5)+DAX*TRAT
      STAA(5,5)=STAA(5,5)+DAX*TRAT
      STAB(5,5)=STAB(5,5)-DAX*TRAT
      STBA(5,5)=STBA(5,5)-DAX*TRAT
      DO 35 I=1,6
      DO 35 J=1,6
      SAA(I,J,ISEG)=STAA(I,J)
      SAB(I,J,ISEG)=STAB(I,J)
      SBA(I,J,ISEG)=STBA(I,J)
      SBB(I,J,ISEG)=STBB(I,J)
      IF(ISEG-1) 60,60,61
60    SA(I,J,ISEG)=STAA(I,J)
      GO TO 62
61    SA(I,J,ISEG)=SA(I,J,ISEG)+STAA(I,J)
62    II=ISEG+1
      SA(I,J,II)=STBB(I,J)
      SB(I,J,ISEG)=STAB(I,J)
35    SC(I,J,ISEG)=STBA(I,J)
40    CONTINUE
C     NOW WE MUST MODIFY FOR SUPPORT CONDITIONS
      DO 53 I=1,NJ
      SA(1,1,I)=SA(1,1,I)+AKX(I)
      SA(2,2,I)=SA(2,2,I)+AKY(I)
      SA(3,3,I)=SA(3,3,I)+AKZ(I)
      SA(4,4,I)=SA(4,4,I)+AKPX(I)
      SA(5,5,I)=SA(5,5,I)+AKPY(I)
53    SA(6,6,I)=SA(6,6,I)+AKPZ(I)
      DO 73 I=1,NJ6
73    DU(I)=PCA(I)
      DO 64 I=1,NJ
      IA=I-1
      II=6*IA
      JMASK=100000
      K=JB(I)
      DO 65 L=1,6
      II=II+1
      JMASK=JMASK/10
      K=K-JMASK
      IF(K) 665,66,66
665   K=K+JMASK
      GO TO 63
66    I1=6*I-12
      IF(I.EQ.1) I1=6*IA
      IF(I.EQ.1) GO TO 67
      DO 68 M=1,6
      I1=I1+1
      DU(I1)=DU(I1)-SB(M,L,IA)*PDIS(II)
68    SB(M,L,IA)=0.
67    DO 69 M=1,6
      I1=I1+1
      DU(I1)=DU(I1)-SA(M,L,I)*PDIS(II)
69    SA(M,L,I)=0.
      IF(I.EQ.NJ) GO TO 72
      DO 71 M=1,6
      I1=I1+1
      DU(I1)=DU(I1)-SC(M,L,I)*PDIS(II)
71    SC(M,L,I)=0.
72    IF(I.EQ.1) GO TO 74
      DO 73 M=1,6
73    SC(L,M,IA)=0.
74    DO 75 M=1,6
75    SA(L,M,I)=0.
      IF(I.EQ.NJ) GO TO 77
      DO 76 M=1,6
76    SB(L,M,I)=0.
77    SA(L,L,I)=1.
      DU(II)=PDIS(II)
63    CONTINUE
64    CONTINUE
      N1=NJ-1
      SPV=1.
      DO 80 I=1,N1
      IA=I+1
      CALL PSINV(SA(1,1,I),6,6)
      DO 83 MAN=1,6
      IF(SA(MAN,MAN,I).LT.0.) SPV=-SPV
83    CONTINUE
      CALL PSMULT(SA(1,1,I),SB(1,1,I),6,6,6,6)
      CALL MATMUL(SC(1,1,I),SB(1,1,I),SA(1,1,IA),6,6,6,6)
      II=6*I-5
      I6=II+6
      CALL PSMULT(SA(1,1,I),DU(II),6,6,6,1)
80    CALL MATMUL(SC(1,1,I),DU(II),DU(I6),6,6,6,1)
      IA=6*NJ-5
      CALL PSINV(SA(1,1,NJ),6,6)
      DO 84 MAN=1,6
      IF(SA(MAN,MAN,NJ).LT.0.) SPV=-SPV
84    CONTINUE
      CALL PSMULT(SA(1,1,NJ),DU(IA),6,6,6,1)
      DO 81 II=1,N1
      I=NJ-II
      I5=6*I-5
      I6=I5+6
81    CALL MATMUL(SB(1,1,I),DU(I6),DU(I5),6,6,6,1)
      IF(SPV.LT.0.) GO TO [illegible]
      DO 82 ISEG=1,NSEG
      J1=6*ISEG-5
```

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      J2=J1+6
      DO 36 I=1,6
      SUM=0.
      I1=J1-1
      I2=J2-1
      DO 69 J=1,6
      I1=I1+1
      I2=I2+1
69    SUM=SUM+SKAA(I,J,ISEG)*DU(I1)+SKAB(I,J,ISEG)*DU(I2)
66    FORA(I,ISEG)=FORA(I,ISEG)+SUM
      DO 90 I=1,6
      SUM=0.
      I1=J1-1
      I2=J2-1
      DO 91 J=1,6
      I1=I1+1
      I2=I2+1
91    SUM=SUM+SKBA(I,J,ISEG)*DU(I1)+SKBB(I,J,ISEG)*DU(I2)
      FOR(I)=SUM
90    FORB(I,ISEG)=FORB(I,ISEG)+SUM
      J1=6*ISEG-5
      J2=J1+6
      C1=(U(J2)-U(J1))/DAL(ISEG)
      J1=J1+2
      J2=J2+2
      C2=(U(J2)-U(J1))/DAL(ISEG)
      J2=1
      DUB(1)=FOR(J2)-C1*FOR(J2+1)
      DUB(2)=C1*FOR(J2)+FOR(J2+1)+C2*FOR(J2+2)
      DUB(3)=-C2*FOR(J2+1)+FOR(J2+2)
      DUB(4)=FOR(J2+3)-C1*FOR(J2+4)
      DUB(5)=C1*FOR(J2+3)+FOR(J2+4)+C2*FOR(J2+5)
      DUB(6)=-C2*FOR(J2+4)+FOR(J2+5)
      DFORB(1,ISEG)=DUB(2)
      DFORB(3,ISEG)=DUB(1)
      DFORB(2,ISEG)=DUB(3)
      DFORB(5,ISEG)=DUB(6)
      DFORB(6,ISEG)=DUB(4)
      AXF(ISEG)=AXF(ISEG)+DUB(2)
32    CONTINUE
      DO 92 I=1,ND
92    U(I)=U(I)+DU(I)
      IF(KPRNT.NE.JPRNT.AND.IL.NE.1.AND.IL.NE.NLINC) GO TO 972
      L=1
      LL=6
      PRINT 7005,IL
7005  FORMAT('1',2X,'LOAD INCREMENT',I5,/,
     1                3X,'*******************')
      DO 952 K=1,NJ
      PRINT 772
772   FORMAT(//,3X,'JOINT NO.',4X,'DISPL.-X',4X,'DISPL.-Y',4X,
     1'DISPL.-Z',2X,'ROTATION-X',2X,'ROTATION-Y',2X,'ROTATION-Z')
      PRINT 773,K,(U(I),I=L,LL)
773   FORMAT(5X,I2,2X,6(2X,1PE10.3))
      L=L+6
      LL=LL+6
952   CONTINUE
      PRINT 7001
7001  FORMAT(/////)
      DO 963 M=1,NSEG
      PRINT 752,M
752   FORMAT(//,4X,'SEG. NO.',5X,'FORCE-X',5X,'FORCE-Y',5X,'FORCE-Z',
     14X,'MOMENT-X',4X,'MOMENT-Y',4X,'MOMENT-Z',/,6X,I5)
      PRINT 733,(-FORA(I,M),I=1,6)
733   FORMAT(7X,'END-A',6(2X,1PE10.3),/)
      PRINT 734,(FORB(I,M),I=1,6)
734   FORMAT(7X,'END-B',6(2X,1PE10.3))
963   CONTINUE
972   IF(KPRNT.EQ.JPRNT) KPRNT=0
      KPRNT=KPRNT+1
999   CONTINUE
1000  CONTINUE
      IF(NLFIG.GT.0) PRINT 6942,LGP
      PRINT 6940,SIGC,MLOAD,MINC,MSEG,MSECT
6940  FORMAT('1',2X,'AT THE END OF ALL LOADING',/,
     1            3X,'**************************',///,
     2            3X,'MAX. COMPRESSIVE CONCRETE STRAIN=',1PE10.3,/,
     3            7X,'OCCURED IN LOAD CASE            =',I5,/,
     C            3X,'OCCURED IN INCREMENT            =',I5,/,
     4            3X,'OCCURED IN SEGMENT              =',I5,/,
     5            3X,'OCCURED IN SECTION              =',I5,//)
      PRINT 6941,SIGS,JLOAG,JINC,JSEG,JSECT
6941  FORMAT('0',2X,'MAX. TENSILE STEEL STRAIN=',1PE10.3,/,
     1            3X,'OCCURED IN LOAD CASE     =',I5,/,
     C            3X,'OCCURED IN INCREMENT     =',I5,/,
     2            3X,'OCCURED IN SEGMENT       =',I5,/,
     3            3X,'OCCURED IN  SECTION      =',I5)
6942  FORMAT('1',10X,'****','CHECK FOR FAILURE HAS SEEN',
     1       '****',/,10X,'****','CARRIED UP TO 132 PERCENT',
     2       '****',/,10X,'****','OF THE REQUIRED ULTIMATE ',
     3       '****',/,10X,'****','LOAD                     ',
     4       '****',/,10X,'****',25X,'****',/,10X,'****',
     5       'THE PIER IS OVERDESIGNED','****',/,10X,'****',
     6       'FOR LOAD GROUP',I5,4X,'****')
      GO  TO 9900
6050  IFAIL=0
      CALL  FAIL(LGP,JAY,IL,IFAIL,ISECT,ISEG)
      GO TO 9972
6100  IFAIL=1
      CALL  FAIL(LGP,JAY,IL,IFAIL,ISECT,ISEG)
      GO TO 9972
6200  IFAIL=-1
      CALL  FAIL(LGP,JAY,IL,IFAIL,ISECT,ISEG)
9972  L=1
      LL=6
      PRINT 6401
6401  FORMAT('1',2X,'THE FINAL DISPLACEMENTS AND FORCES',
     1' BEFORE FAILURE')
      DO 4301 IK=1,NJ
      PRINT 772
      PRINT 773,IK,(U(I),I=L,LL)
      L=L+6
      LL=LL+6
4301  CONTINUE
      PRINT 7001
      DO 4303 KM=1,NSEG
      PRINT 752,KM
      PRINT 733,(FORA(I,KM),I=1,6)
      PRINT 734,(FORB(I,KM),I=1,6)
4303  CONTINUE
9900  ILF=ILF+1
      IF(ILF.LE.NLFIG) GO TO 5000
      STOP
      END
C*P4SV1P1*
      SUBROUTINE MPOT(A,B)
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      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      DIMENSION A(6,6),B(6,6),C(6)
      DO 1 I=1,6
      DO 2 J=1,6
      SUM=0.
      DO 3 K=1,6
    3 SUM=SUM+A(I,K)*B(K,J)
    2 C(J)=SUM
      DO 4 J=1,6
    4 A(I,J)=C(J)
    1 CONTINUE
      DO 5 J=1,6
      DO 6 I=1,6
      SUM=0.
      DO 7 K=1,6
    7 SUM=SUM+A(K,J)*B(K,I)
    6 C(I)=SUM
      DO 8 I=1,6
    8 A(I,J)=C(I)
    5 CONTINUE
      RETURN
C**  THIS PROGRAM VALID ON FTN4 AND FTN5 **
      END
C*FASV1PO*
      SUBROUTINE MATMUL(A,B,C,NDIM,N,NDIP,M)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      DIMENSION A(NDIM,NDIM),B(NDIM,NDIM),C(NDIM,NDIM)
      DO 10 I=1,N
      DO 10 J=1,M
      SUM=0.
      DO 11 K=1,N
   11 SUM=SUM+A(I,K)*B(K,J)
   10 C(I,J)=C(I,J)-SUM
      RETURN
C**  THIS PROGRAM VALID ON FTN4 AND FTN5 **
      END
C*FASV1PO*
      SUBROUTINE PSINV(A,NDIM,N)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      DIMENSION A(NDIM,NDIM)
      N1=N-1
      DO 10 I=1,N1
      C=1./A(I,I)
      I1=I+1
      DO 11 J=I1,N
   11 A(I,J)=A(I,J)*C
      DO 12 K=I1,N
      DO 12 J=I1,N
   12 A(K,J)=A(K,J)-A(K,I)*A(I,J)
   10 CONTINUE
      RETURN
C**  THIS PROGRAM VALID ON FTN4 AND FTN5 **
      END
C*FASV1PO*
      SUBROUTINE PSMULT(A,B,NDIM,N,NDIM,M)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      DIMENSION A(NDIM,NDIM),B(NDIM,NDIM)
      N1=N-1
      DO 10 I=1,N1
      C=1./A(I,I)
      DO 11 J=1,M
   11 B(I,J)=B(I,J)*C
      I1=I+1
      DO 12 K=I1,N
      DO 12 J=1,M
   12 B(K,J)=B(K,J)-A(K,I)*B(I,J)
   10 CONTINUE
      C=1./A(N,N)
      DO 13 J=1,M
   13 B(N,J)=B(N,J)*C
      DO 14 II=1,N1
      I=N-II
      I1=I+1
      DO 15 K=I1,N
      DO 15 J=1,M
   15 B(I,J)=B(I,J)-A(I,K)*B(K,J)
   14 CONTINUE
      RETURN
C**  THIS PROGRAM VALID ON FTN4 AND FTN5 **
      END
C*FASV1PO*
      SUBROUTINE DSCRIS(CONPX,CONPY,CPA,ICODE,NPIS)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      CHARACTER*4,IOR(3),IND,IN,IV,ICODE
      DIMENSION CONPX(200),CONPY(200),CPA(200)
      COMMON /ONE/ XI,TP,YI,TW
      COMMON /TWO/ CTHOIC(5),TWALL(5),NN,NWX,KWY
      COMMON /THRE/ IOR,IND,IN,IV
      COMMON /SUP/ IPRNT
      COMMON /SAME/ KCTYPE,KSTYPE,KPY,KWX,NX,NY,KV,KM,NI
      REAL XINERX,YINERY
      IXI=XI
      IYI=YI
      KPX=2
      KWY=2
      FACT=180.
      IF(ICODE.NE.IND) FACT=180.-NN*10.
      IF(KCTYPE.GT.0) GO TO 25
      XKPY=FACT/(6.*(1.+YI/XI))
      KPY=XKPY
      XKWX=YI/XI*XKPY
      KWX=XKWX
   25 XPINC=(XI-2.0*TW)/KPY
      YPINC=TP/KPX
      XWINC=TW/KWY
      YWINC=(YI-2.0*TP)/KWX
      NPIS=2*KPX*KPY+2*KWX*KWY+4*KPX*KWY
      XINERX=XI-2.0*TW
      YINERY=YI-2.0*TP
      NIX=XINERX/2.0
      NIY=YINERY/2.0
      STATX=-XI/2.0+XWINC/2.0
      STATY=YI/2.0-YPINC/2.0
      KC=1
      Y=STATY
      IT=0
   96 DO 10 I=1,KPX
      X=STATX
      DO 20 J=1,KWY-1
      CONPX(KC)=X
      CONPY(KC)=Y
      CPA(KC)=XWINC*YPINC
      X=X+XWINC
   20 KC=KC+1
      CONPX(KC)=X
      CONPY(KC)=Y
```

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      CPA(KC)=XWINC*YPINC
      KC=KC+1
      X=X+XPINC/2.0+XWINC/2.0
      CONFX(KC)=X
      CONFY(KC)=Y
      CPA(KC)=XPINC*YPINC
      DO 30 K=1,KPY-1
      X=X+XPINC
      KC=KC+1
      CPA(KC)=XPINC*YPINC
      CONFX(KC)=X
30    CONFY(KC)=Y
      X=X+XPINC/2.0+XWINC/2.0
      KC=KC+1
      CONFX(KC)=X
      CONFY(KC)=Y
      CPA(KC)=XWINC*YPINC
      KC=KC+1
      DO 40 JA=1,KWY-1
      X=X+XWINC
      CONFX(KC)=X
      CONFY(KC)=Y
      CPA(KC)=XWINC*YPINC
40    KC=KC+1
10    Y=Y-YPINC
      IF(IT.NE.0) GO TO 99
      Y=Y+YPINC/2.0-YWINC/2.0
      DO 50 L=1,NWX
      X=STRTX
      DO 60 M=1,KWY-1
      CONFX(KC)=X
      CONFY(KC)=Y
      CPA(KC)=XWINC*YWINC
      X=X+XWINC
60    KC=KC+1
      CONFX(KC)=X
      CONFY(KC)=Y
      CPA(KC)=XWINC*YWINC
      KC=KC+1
      X=X+XINERX+XWINC
      CONFX(KC)=X
      CONFY(KC)=Y
      CPA(KC)=XWINC*YWINC
      KC=KC+1
      DO 70 N=1,KWY-1
      X=X+XWINC
      CONFX(KC)=X
      CONFY(KC)=Y
      CPA(KC)=XWINC*YWINC
70    KC=KC+1
50    Y=Y-YWINC
      Y=Y+YWINC/2.0-YPINC/2.0
      IT=1
      GO TO 10
99    CONTINUE
      NPF=KPX*KPY+KPX*KWY*2
      NWF=KWX*KWY
      IF(IPRNT.EQ.0) GO TO 113
      PRINT 100
100   FORMAT (///,10X,'CONCRETE FIBER PROPERTIES',
     1//,3X,'NO.',5X,'X',12X,'Y',12X,'AREA')
      DO 110 MO=1,NFIS
      PRINT 102,MO,CONFX(MO),CONFY(MO),CPA(MO)
102   FORMAT (I5,3X,F10.2,2X,F10.2,3X,F10.2)
110   CONTINUE
113   CONTINUE
      IF(ICODE.EQ.IND) GO TO 199
      IF(IOR(1).EQ.IH) GO TO 140
      IF(IOR(1).EQ.IV) GO TO 130
130   NWX=2
      NWY=5
      WALLX=XINERX/NWY
      ANWX=NWX
      DO 150 L=1,NW
      WALLY=TWALL(L)/ANWX
      YSTWL=CTROID(L)+(ANWX-1.0)*.5*WALLY
      WY=YSTWL+WALLY
      DO 160 K=1,NWX
      WY=WY-WALLY
      WX=-XI/2.0+TW-WALLX/2.0
      DO 170 M=1,NWY
      WX=WX+WALLX
      CONFX(KC)=WX
      CONFY(KC)=WY
      CPA(KC)=WALLX*WALLY
170   KC=KC+1
160   CONTINUE
150   CONTINUE
      NWLFIS=NW*NWX*NWY
      INDEX=NFIS+1
      NFIS=NFIS+NWLFIS
      IF(IPRNT.EQ.0) GO TO 199
      DO 180 I=INDEX,NFIS
      PRINT 102,I,CONFX(I),CONFY(I),CPA(I)
180   CONTINUE
      GO TO 199
140   NWX=5
      NWY=2
      ANWY=NWY
      ANWX=NWX
      WALLY=YINERY/ANWX
      DO 210 L=1,NW
      WALLX=TWALL(L)/ANWY
      XSTWL=CTROID(L)-(ANWY-1.0)*.5*WALLX
      WX=XSTWL-WALLX
      DO 220 K=1,NWY
      WX=WX+WALLX
      WY=YI/2.0-TP+WALLY/2.0
      DO 230 M=1,NWX
      WY=WY-WALLY
      CONFX(KC)=WX
      CONFY(KC)=WY
      CPA(KC)=WALLX*WALLY
230   KC=KC+1
220   CONTINUE
210   CONTINUE
      NWLFIS=NW*NWX*NWY
      INDEX=NFIS+1
      NFIS=NFIS+NWLFIS
      IF(IPRNT.EQ.0) GO TO 199
      DO 250 I=INDEX,NFIS
      PRINT 102,I,CONFX(I),CONFY(I),CPA(I)
250   CONTINUE
199   CONTINUE
      KCTYPE=KCTYPE+1
      RETURN
```

```
      END
C*F45V1PC*
      SUBROUTINE SSCRIS(STLX,STLY,STLA,ICODE,NSTOT,ITYP)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      CHARACTER*6,IOR(3),INO,IH,IV,ISOLID,IHOLL,IOVAL,ICHOLL,
     . ITYP,ICODE
      DIMENSION STLX(200),STLY(200),STLA(200)
      COMMON /ONE/ XI,TF,YI,TW
      COMMON /TWO/ CTROID(3),TWALL(5),NW,NWX,NWY
      COMMON /THRE/ IOR,INO,IH,IV
      COMMON /FOUR/ STY(3),STS(3),GAMMAX(3),GAMMAY(3),WLSTL(5),NRING
      COMMON /FIVE/ ISOLID,IHOLL,IOVAL,ICHOLL
      COMMON /SUP/ IPRNT
      COMMON /SAME/ KCTYPE,KSTYPE,KPY,KWX,NX,NY,KV,KM,NZ
      REAL XINERX,YINERY
      NWX=10
      NWY=10
      KC=1
      IF(KSTYPE.GT.0) GO TO 25
      VAL=200./NRING
      IF(ITYP.EQ.IHOLL.AND.ICODE.NE.INO) VAL=(200.-NW*10.)/NRING
      IVAL=VAL
      XNX=VAL/(2.*(1.+YI/XI))
      NX=XNX
      NY=YI/XI*NX
   25 NSTOT=(2*(NX+NY))*NRING
      DO 10 I=1,NRING
      GX=XI-2.0*GAMMAX(I)
      GY=YI-2.0*GAMMAY(I)
      HT=STY(I)/GX
      HS=STS(I)/GY
      XINC=GX/NX
      YINC=GY/NY
      STRTX=-XI/2.0+GAMMAX(I)
      STRTY=YI/2.0-GAMMAY(I)
      X=STRTX-XINC/2.0
      Y=STRTY
      DO 20 J=1,NX
      X=X+XINC
      STLX(KC)=X
      STLY(KC)=Y
      STLA(KC)=XINC*HT
   20 KC=KC+1
      X=STRTX-XINC/2.0
      Y=-STRTY
      DO 30 K=1,NX
      X=X+XINC
      STLX(KC)=X
      STLY(KC)=Y
      STLA(KC)=XINC*HT
   30 KC=KC+1
      X=STRTX
      Y=STRTY+YINC/2.0
      DO 40 L=1,NY
      Y=Y-YINC
      STLX(KC)=X
      STLY(KC)=Y
      STLA(KC)=YINC*HS
   40 KC=KC+1
      X=-STRTX
      Y=STRTY+YINC/2.0
      DO 50 M=1,NY
      Y=Y-YINC
      STLX(KC)=X
      STLY(KC)=Y
      STLA(KC)=YINC*HS
   50 KC=KC+1
   10 CONTINUE
      IF(IPRNT.EQ.0) GO  TO 213
      PRINT 100
  100 FORMAT (///,10X,'STEEL FIBER PROPERTIES'
     1//,3X,'NO.',6X,'Z',13X,'X',10X,'AREA')
      PRINT 110,(K,STLX(K),STLY(K),STLA(K),K=1,NSTOT)
  110 FORMAT (I5,3X,F10.2,2X,F10.2,3X,F10.2)
  213 CONTINUE
      IF(ITYP.EQ.ISOLID) GO TO 199
      IF(ICODE.EQ.INO) GO TO 199
      IF(IOR(1).EQ.IH) GO TO 140
      IF(IOR(1).EQ.IV) GO TO 130
  130 XINERX=XI-2.0*TW
      WALLX=XINERX/NWY
      DO 150 L=1,NW
      HW=WLSTL(L)/XINERX
      YSTWL=CTROID(L)
      WX=-XI/2.0+TW-WALLX/2.0
      DO 160 K=1,NWY
      WX=WX+WALLX
      STLX(KC)=WX
      STLY(KC)=YSTWL
      STLA(KC)=HW*WALLX
  160 KC=KC+1
  150 CONTINUE
      INDEX=NSTOT+1
      NSTOT=NSTOT+NW*NWY
      IF(IPRNT.EQ.0) GO TO 199
      PRINT 110,(I,STLX(I),STLY(I),STLA(I),I=INDEX,NSTOT)
      GO TO 199
  140 YINERY=YI-2.0*TF
      WALLY=YINERY/NWX
      DO 170 K=1,NW
      HW=WLSTL(K)/YINERY
      XSTWL=CTROID(K)
      WY=YI/2.0-TF+WALLY/2.0
      DO 180 P=1,NWX
      WY=WY-WALLY
      STLX(KC)=XSTWL
      STLY(KC)=WY
      STLA(KC)=HW*WALLY
  180 KC=KC+1
  170 CONTINUE
      INDEX=NSTOT+1
      NSTOT=NSTOT+NW*NWX
      IF(IPRNT.EQ.0) GO TO 199
      PRINT 110,(I,STLX(I),STLY(I),STLA(I),I=INDEX,NSTOT)
  199 CONTINUE
      KSTYPE=KSTYPE+1
      RETURN
      END
C*F45V1PO*
      SUBROUTINE CNSTP(CONPX,CONPY,CPA,STLX,STLY,STLA,EZC,EZS,
     1NFIS,NSTOT,ISEG,ICON)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      DIMENSION CONPX(200),CONPY(200),CPA(200),STLX(200),STLY(200),
     1STLA(200),EZC(200),EZS(200)
      COMMON /STP/ DA11,DA12,DA13,DA21,DA22,DA23,DA31,DA32,DA33
      DA11=0.0
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      DA12=0.0
      DA13=0.0
      DA21=0.0
      DA22=0.0
      DA23=0.0
      DA31=0.0
      DA32=0.0
      DA33=0.0
      DO 10 I=1,NFIB
      ECON=EZC(I)
      IF(ICON.EQ.0) CALL FC(ECON,SC,ETC)
      IF(ICON.EQ.1) CALL FCON(ECON,SC,ETC)
      DA11=DA11+ETC*CPA(I)
      DA12=DA12+ETC*CONFY(I)*CPA(I)
      DA13=DA13+ETC*CONFX(I)*CPA(I)
      DA21=DA21+ETC*CONFY(I)*CPA(I)
      DA22=DA22+ETC*CONFY(I)**2*CPA(I)
      DA23=DA23+ETC*CONFX(I)*CONFY(I)*CPA(I)
      DA31=DA31+ETC*CONFX(I)*CPA(I)
      DA32=DA32+ETC*CONFX(I)*CONFY(I)*CPA(I)
      DA33=DA33+ETC*CONFX(I)**2*CPA(I)
 10   CONTINUE
      DO 20 J=1,NSTOT
      ESTL=EZS(J)
      CALL FS(ESTL,SS,ETS,ISEC)
      IF(ICON.EQ.0) CALL FC(ESTL,SSB,ESUB)
      IF(ICON.EQ.1) CALL FCON(ESTL,SSB,ESUB)
      EEFF=ETS-ESUB
      DA11=DA11+EEFF*STLA(J)
      DA12=DA12+EEFF*STLY(J)*STLA(J)
      DA13=DA13+EEFF*STLX(J)*STLA(J)
      DA21=DA21+EEFF*STLY(J)*STLA(J)
      DA22=DA22+EEFF*STLY(J)**2*STLA(J)
      DA23=DA23+EEFF*STLX(J)*STLY(J)*STLA(J)
      DA31=DA31+EEFF*STLX(J)*STLA(J)
      DA32=DA32+EEFF*STLX(J)*STLY(J)*STLA(J)
      DA33=DA33+EEFF*STLX(J)**2*STLA(J)
 20   CONTINUE
      RETURN
C**   THIS PROGRAM VALID ON FTN4 AND FTN5 **
      END
C*F45V1P0*
      SUBROUTINE SOLSCR(CONFI,CONFX,CPA,NFIS)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      DIMENSION CONFI(200),CONFX(200),CPA(200)
      COMMON /ONE/ ZI,TF,XI,TW
      COMMON /SUP/ IPRNT
      COMMON /SAME/ KCTYPE,KSTTPE,KFY,KMX,NX,NY,KV,KH,NZ
      IF(KCTYPE.GT.0) GO TO 25
      FACTOR=SQRT(200./(XI/ZI))
      KV=FACTOR
      KH=XI/ZI*KV
 25   NFIS=KV*KH
      IF(NFIB.GT.200) STOP 555
      HINC=ZI/KV
      VINC=XI/KH
      HSTRT=-ZI/2.0+HINC/2.0
      VSTRT=XI/2.0-VINC/2.0
      KC=1
      V=VSTRT
      DO 10 I=1,KH
      H=HSTRT
      DO 20 J=1,KV
      CONFI(KC)=H
      CONFX(KC)=V
      CPA(KC)=HINC*VINC
      H=H+HINC
 20   KC=KC+1
 10   V=V-VINC
      IF(IPRNT.EQ.0) GO  TO 113
      PRINT 100
 100  FORMAT(///,10X,'CONCRETE FIBER PROPERTIES',
     1//,3X,'NO.',6X,'Z',12X,'X',12X,'AREA')
      DO 110 M=1,NFIS
      PRINT 102,M,CONFI(M),CONFX(M),CPA(M)
 102  FORMAT(I5,3(3X,F10.2))
 110  CONTINUE
 113  CONTINUE
      KCTYPE=KCTYPE+1
      RETURN
      END
C*F45V1P0*
      SUBROUTINE OSCRIB(CONFI,CONFX,CPA,NFIS)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      DIMENSION CONFI(200),CONFX(200),CPA(200)
      COMMON /ONE/ ZI,TF,XI,TW
      COMMON /SUP/ IPRNT
      COMMON /SAME/ KCTYPE,KSTYPE,KFY,KMX,NX,NY,KV,KH,NI
      SI=ZI-XI
      IF(SI.LE.0.) GO TO 15
      UI=XI
      VI=ZI
      GO TO 33
  15  UI=ZI
      VI=XI
   35 SI=ABS(SI)
      IF(SI.LE.1.0) GO TO 50
      IF(KCTYPE.GT.0) GO TO 25
      FACTOR=SQRT(125./(UI/SI))
      KV=FACTOR
      KH=UI/SI*KV
 25   NFIS=KV*KH
      HINC=SI/KV
      VINC=UI/KH
      HSTRT=-SI/2.0+HINC/2.0
      VSTRT=UI/2.0-VINC/2.0
      KC=1
      V=VSTAT
      DO 10 I=1,KH
      H=HSTRT
      DO 20 J=1,KV
      CONFI(KC)=H
      CONFX(KC)=V
      CPA(KC)=HINC*VINC
      H=H+HINC
 20   KC=KC+1
 10   V=V-VINC
      RADIUS=UI/2.0
      PI=3.1415927
      H=1.0/6.0*RADIUS
      ANG=75.0*PI/180.
      C1=.5*30.*PI/180.*H
      ALPHA=15.*PI/180.
      CON=2.*SIN(ALPHA)/(3.*ALPHA)
      DO 30 I=1,3
      R1=RADIUS
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      R2=5./6.*RADIUS
      DO 40 J=1,6
      A=C1*(R1+R2)
      ONE=CON*R1*ALPHA*R1**2
      TWO=CON*R2*ALPHA*R2**2
      RAD=(ONE-TWO)/A
      Z=VI/2.0-RADIUS+RAD*COS(ANG)
      X=RAD*SIN(ANG)
      CONFZ(KC)=Z
      CONFX(KC)=X
      CFA(KC)=A
      KC=KC+1
      CONFZ(KC)=-Z
      CONFX(KC)=X
      CFA(KC)=A
      KC=KC+1
      CONFZ(KC)=-Z
      CONFX(KC)=-X
      CFA(KC)=A
      KC=KC+1
      CONFZ(KC)=Z
      CONFX(KC)=-X
      CFA(KC)=A
      KC=KC+1
      R1=R1-W
40    R2=R2-W
30    ANG=ANG-30.*PI/180.
      NFIB=NFIB+72
      GO TO 90
 50   KC=1
      RADIUS=DI/2.0
      PI=3.1415927
      W=1.0/6.0*RADIUS
      ANG=52.5*PI/180.
      C1=.5*15.*PI/180.*W
      ALPHA=7.5*PI/180.
      CON=2.*SIN(ALPHA)/(3.*ALPHA)
      DO 70 I=1,6
      R1=RADIUS
      R2=7./8.*RADIUS
      DO 60 J=1,5
      A=C1*(R1+R2)
      ONE=CON*R1*ALPHA*R1**2
      TWO=CON*R2*ALPHA*R2**2
      RAD=(ONE-TWO)/A
      Z=RAD*COS(ANG)
      X=RAD*SIN(ANG)
      CONFZ(KC)=Z
      CONFX(KC)=X
      CFA(KC)=A
      KC=KC+1
      CONFZ(KC)=-Z
      CONFX(KC)=X
      CFA(KC)=A
      KC=KC+1
      CONFZ(KC)=-Z
      CONFX(KC)=-X
      CFA(KC)=A
      KC=KC+1
      CONFZ(KC)=Z
      CONFX(KC)=-X
      CFA(KC)=A
      KC=KC+1
      R1=R1-W
60    R2=R2-W
70    ANG=ANG-15.*PI/180.
      NFIB=192
   90 IF(NFIB.GT.200) STOP 500
      IF(IPRNT.EQ.0) GO TO 113
      PRINT 100
100   FORMAT(///,10X,'CONCRETE FIBER PROPERTIES',
     1//,3X,'NO.',9X,'Z',12X,'X',12X,'AREA')
      DO 110 M=1,NFIB
      PRINT 102,M,CONFZ(M),CONFX(M),CFA(M)
102   FORMAT(I5,3(3X,F10.2))
110   CONTINUE
113   CONTINUE
      KCTYPE=KCTYPE+1
      RETURN
      END
C*FASVIPG*
      SUBROUTINE SSSTL(STLZ,STLX,STLA,NSTOT)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      DIMENSION STLZ(200),STLX(200),STLA(200)
      COMMON /ONE/ ZI,TF,XI,T,
      COMMON /FOUR/ STT(5),STS(5),GAMMAZ(3),GAMMAX(3),WLSTL(5),NRING
      COMMON /SUP/ IPRNT
      COMMON /SAME/ KCTYPE,KSTYPE,KFY,K,K,NX,NY,NV,KM,NZ
      KC=1
      IF(KSTYPE.GT.0) GO TO 25
      VAL=200.-24.*NRING
      PI=3.1415927
      ZZ=(VAL/NRING)/4.0
      NZ=ZZ
25    NSTOT=(2*NZ)*NRING+24*NRING
      DO 10 I=1,NRING
      GZ=ZI-XI
      RAD=XI/2.0-GAMMAZ(I)
      GCIR=PI*RAD
      WT=STT(I)/GZ
      WCIR=STS(I)/GCIR
      ZINC=GZ/NZ
      CIRINC=GCIR/12.0
      STATZ=-ZI/2.0+XI/2.0+ZINC/2.0
      STATX=XI/2.0-GAMMAX(I)
      Z=STATZ
      X=STATX
      DO 20 J=1,NZ
      STLZ(KC)=Z
      STLX(KC)=X
      STLA(KC)=ZINC*WT
      KC=KC+1
      STLZ(KC)=Z
      STLX(KC)=-X
      STLA(KC)=ZINC*WT
      Z=Z+ZINC
20    KC=KC+1
      ANG=82.5*PI/180.
      DO 30 K=1,6
      Z=ZI/2.0-XI/2.0+RAD*COS(ANG)
      X=RAD*SIN(ANG)
      A=CIRINC*WCIR
      STLZ(KC)=Z
      STLX(KC)=X
      STLA(KC)=A
      KC=KC+1
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      STLZ(KC)=-Z
      STLX(KC)=X
      STLA(KC)=A
      KC=KC+1
      STLZ(KC)=-Z
      STLX(KC)=-X
      STLA(KC)=A
      KC=KC+1
      STLZ(KC)=Z
      STLX(KC)=-X
      STLA(KC)=A
      KC=KC+1
 30   ANG=ANG-15.*PI/180.
 10   CONTINUE
      IF(IPRNT.EQ.0) GO TO 111
      PRINT 100
 100  FORMAT(///,10X,'STEEL FIBER PROPERTIES'
     1//,3X,'NO.',8X,'Z',13X,'X',10X,'AREA')
      PRINT 110,(K,STLZ(K),STLX(K),STLA(K),K=1,NSTOT)
 110  FORMAT(1X,I4,3X,F10.2,2X,F10.2,3X,F10.2)
 111  CONTINUE
      KSTYPE=KSTYPE+1
      RETURN
      END
C*P45V1PC*
      SUBROUTINE CIRSCR(CONFX,CONFY,CPA,NFIB)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      DIMENSION CONFX(200),CONFY(200),CPA(200)
      COMMON /ONE/ XI,TF,TI,XF
      KC=1
      PI=3.14156
      ANG=9.0*PI/180.0
      RADIUS=XI/2.0
      H=TF/5.0
      STANG=ANG/2.0
      R1=RADIUS
      R2=RADIUS-H
      DO 10 I=1,5
      AREA=0.5*ANG*H*(R1+R2)
      C1=2.0/3.0*SIN(STANG)/STANG
      C2=R1**3-R2**3
      C3=R1**2-R2**2
      RCEN=C1*C2/C3
      TANG=STANG
      DO 20 J=1,10
      X=RCEN*COS(TANG)
      Y=RCEN*SIN(TANG)
      CPA(KC)=AREA
      CONFX(KC)=X
      CONFY(KC)=Y
      KC=KC+1
      CPA(KC)=AREA
      CONFX(KC)=X
      CONFY(KC)=-Y
      KC=KC+1
      CPA(KC)=AREA
      CONFX(KC)=-X
      CONFY(KC)=Y
      KC=KC+1
      CPA(KC)=AREA
      CONFX(KC)=-X
      CONFY(KC)=-Y
      KC=KC+1
```

```
      TANG=TANG+ANG
 20   CONTINUE
      R1=R1-H
      R2=R2-H
 10   CONTINUE
      NFIB=KC-1
      IF(NFIB.GT.200) STOP 500
      PRINT 100
 100  FORMAT(///,10X,'CONCRETE FIBER PROPERTIES',
     1//,3X,'NO.',5X,'X',12X,'Y',10X,'AREA')
      DO 110 M=1,NFIB
      PRINT 102,M,CONFX(M),CONFY(M),CPA(M)
 102  FORMAT(I5,3(3X,F10.2))
 110  CONTINUE
      RETURN
      END
C*P45V1PD*
      SUBROUTINE CIRSTL(STLX,STLY,STLA,NSTOT)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      DIMENSION STLX(200),STLY(200),STLA(200)
      COMMON /ONE/ XI,TF,TI,TW
      COMMON /FOUR/ STT(3),STS(3),GAMMAX(3),GAMMAY(3),NLSTL(3),NRING
      PI=3.14156
      VAL=200.0/NRING
      NVAL=VAL
      KVAL=NVAL/4
      ANG=(PI/2.0)/KVAL
      RADIUS=XI/2.0
      STANG=ANG/2.0
      KC=1
      DO 10 I=1,NRING
      R=RADIUS-GAMMAX(I)
      ST=0.5*STS(I)
      AREA=ST/KVAL
      RCEN=R*SIN(STANG)/(STANG)
      TANG=STANG
      DO 20 J=1,KVAL
      X=RCEN*COS(TANG)
      Y=RCEN*SIN(TANG)
      STLA(KC)=AREA
      STLX(KC)=X
      STLY(KC)=Y
      KC=KC+1
      STLA(KC)=AREA
      STLX(KC)=X
      STLY(KC)=-Y
      KC=KC+1
      STLA(KC)=AREA
      STLX(KC)=-X
      STLY(KC)=Y
      KC=KC+1
      STLA(KC)=AREA
      STLX(KC)=-X
      STLY(KC)=-Y
      KC=KC+1
      TANG=TANG+ANG
 20   CONTINUE
 10   CONTINUE
      NSTOT=KC-1
      IF(NSTOT.GT.200) STOP 570
      PRINT 100
 100  FORMAT(///,10X,'STEEL FIBER PROPERTIES',
     1//,3X,'NO.',7X,'X',11X,'Y',10X,'AREA')
```

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      PRINT 110,(K,STLX(K),STLY(K),STLA(K),K=1,NSTOT)
 110  FORMAT(1X,I4,3X,F10.2,2X,F10.2,3X,F10.2)
      RETURN
      END
C*P45V1PO*
      SUBROUTINE READ(STLZ,STLX,STLA,NSTOT)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      DIMENSION STLZ(200),STLX(200),STLA(200)
      READ (*,100,END=10000)NSTOT
 100  FORMAT(5Z,I10)
10000 PRINT 150,NSTOT
 150  FORMAT(///,3X,'NUMBER OF READ IN STEEL FIBERS=',I5)
      PRINT 200
 200  FORMAT(///,10X,'STEEL FIBER PROPERTIES',
     1//,3X,'NO.',8X,'Z',13X,'X',13X,'AREA')
      READ (*,300,END=10001)(STLZ(I),STLX(I),STLA(I),I=1,NSTOT)
 300  FORMAT(8X,3(3F10.2))
10001 PRINT 110,(K,STLZ(K),STLX(K),STLA(K),K=1,NSTOT)
 110  FORMAT(1X,I4,3X,F10.2,2X,F10.2,3X,F10.2)
      RETURN
      END
C*P45V1PO*
      SUBROUTINE FAIL(LGP,JAY,IL,IFAIL,ISECT,ISEG)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      CHARACTER*4,ASHTO(10)
      COMMON /D1/ ASHTO
      COMMON/P1/ SIGC,MLOAD,MINC,MSEG,MSECT
      COMMON/P2/ SIGS,JLOAD,JINC,JSEG,JSECT
 2000 FORMAT('1',2X,'THE STIFFNESS MATRIX HAS BECOME NEGATIVE',
     * '. NO FURTHER CALCULATIONS POSSIBLE',/,3X,
     *       '*****************************************')
 2010 FORMAT('1',2X,'THE PIER HAS A CONCRETE COMPRESSION FAILURE',/,3X,
     *       '********************************************')
 2020 FORMAT('1',2X,'THE PIER HAS A STEEL TENSION FAILURE',/,3X,
     *       '************************************')
 2030 FORMAT('0',5X,'IN SECTION',I5,/,6X,'OF SEGMENT',I5,/)
 2040 FORMAT('0',5X,'IN LOAD INCREMENT',I5,/,6X,'OF LOAD CASE',I5)
 2050 FORMAT('0',///,10X,'JUST BEFORE FAILURE',/,
     * 10X,'*******************',//,
     1        3X,'MAX. COMPRESSIVE CONCRETE STRAIN=',1PE10.3,/,
     2        3X,'OCCURED IN LOAD CASE             =',I5,/,
     C             3X,'OCCURED IN INCREMENT          =',I5,/,
     4             3X,'OCCURED IN SEGMENT            =',I5,/,
     5             3X,'OCCURED IN SECTION            =',I5,/)
 2060 FORMAT('0',2X,'MAX. TENSILE STEEL STRAIN=',1PE10.3,/,
     1             3X,'OCCURED IN LOAD CASE     =',I5,/,
     C             3X,'OCCURED IN INCREMENT     =',I5,/,
     2             3X,'OCCURED IN SEGMENT       =',I5,/,
     3             3X,'OCCURED IN  SECTION      =',I5)
 2070 FORMAT ('0',2X,'AT ',F6.2,' PERCENT OF THE REQUIRED ULTIMATE',
     1        2X,' LOAD CORRESPONDING TO AASHTO LOAD GROUP',A4)
 2080 FORMAT('0',//,2X,'THE PIER HAS BEEN GROSSLY UNDERDESIGNED',/)
 2082 FORMAT('0',//,2X,'THE PIER HAS BEEN SIGNIFICANTLY UNDERDESIGN')
 2085 FORMAT ('0',//,2X,'THE PIER HAS BEEN UNDERDESIGNED',/)
 2090 FORMAT('0',//,2X,'THE PIER HAS BEEN ACCEPTABLY DESIGNED',/)
 2110 FORMAT('0',//,3X,'THE PIER HAS BEEN OVERDESIGNED',/)
 2112 FORMAT('0',//,2X,'THE PIER HAS BEEN SIGNIFICANTLY OVERDESIGN')
 2115 FORMAT('0',//,2X,'THE PIER HAS BEEN GROSSLY OVERDESINGED',/)
 2120 FORMAT('0',2X,'FOR LOAD GROUP',A4)
      PER=-1.0
      IF(LGP.LE.0) GO TO 50
      KJAY=JAY-2
      IF(KJAY) 20,30,40
   20 PER=IL*1./50.
      GO TO 50
   30 PER=50.+IL*.2/50.
      GO TO 50
   40 PER=100.+IL*.2/50.
   50 IF(IFAIL) 60,70,80
   60 PRINT 2020
      PRINT 2030,ISECT,ISEG
      IF(LGP.LE.0) PRINT 2040,IL,JAY
      IF(LGP.GT.0) PRINT 2070,PER,ASHTO(LGP)
      GO TO 100
   70 PRINT 2000
      IF(LGP.LE.0) PRINT 2040,IL,JAY
      IF(LGP.GT.0) PRINT 2070,PER,ASHTO(LGP)
      GO TO 100
   80 PRINT 2010
      PRINT 2030,ISECT,ISEG
      IF(LGP.LE.0) PRINT 2040,IL,JAY
      IF(LGP.GT.0) PRINT 2070,PER,ASHTO(LGP)
  100 PRINT 2050,SIGC,MLOAD,MINC,MSEG,MSECT
      PRINT 2060,SIGS,JLOAD,JINC,JSEG,JSECT
      IF(PER.LT.0) GO TO 110
      IF(PER.GE.0.AND.PER.LT.70.) PRINT 2080
      IF(PER.GE.70..AND.PER.LT.85.) PRINT 2082
      IF(PER.GE.85..AND.PER.LT.95.)PRINT 2085
      IF(PER.GE.95..AND.PER.LE.105.)PRINT 2090
      IF(PER.GT.105..AND.PER.LE.115.) PRINT 2110
      IF(PER.GT.115..AND.PER.LE.130) PRINT 2112
      IF(PER.GT.130.) PRINT 2115
      PRINT 2120,ASHTO(LGP)
  110 RETURN
      END
C*P45V1PO*
      SUBROUTINE GDSIN(NLPOG,ILP,NOP,NLOAD,JPRNT,LGP)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      CHARACTER*4,A(14,13),ASHTO(10)
      DIMENSION POR(6),PLOAD(66,14),PFAC(10,14),GAMMA(10),
     1          FORLG(66,14),LGI(13)
      COMMON /D1/ ASHTO
      COMMON/CSING/SUMF(66)
      COMMON /D2/ GAMMA,BFAC,IPRS
      DATA (A(1,J),J=1,13)/'DEAD',' LOA','D (G','ETA ','C EQ',
     1      'UAL ','1.0)',6*'    '/
      DATA (A(2,J),J=1,13)/'DEAD',' LOA','D (B','ETA ','O EQ',
     1      'UAL ','.75)',6*'    '/
      DATA (A(3,J),J=1,13)/'LIVE',' LOA','D AN','D OR',' LIV',
     1      'E LO','AD I','MPAC','T   ',4*'    '/
      DATA (A(4,J),J=1,13)/'CENT','RIFU','GAL ','FORC','E   ',
     1      8*'    '/
      DATA (A(5,J),J=1,13)/'EART','H PR','ESSU','RE  ',9*'    '/,
     1(A(6,J),J=1,13)/'EART','H PR','ESSU','RE(F','RAME','S)  ',
     2                 7*'    '/,(A(7,J),J=1,13)/'BUOY','ANCY',11*'    '/,
      DATA (A(8,J),J=1,13)/'STRE','AM F','LOW ','PRES','SURE',8*'    '/,
     1      (A(9,J),J=1,13)/'WIND',' LOA','D ON',' STR','UCTU','RE  ',
     2      7*'    '/
      DATA (A(10,J),J=1,13)/'WIND',' LOA','D ON',' LIV','E LO','AD  ',
     1      7*'    '/,(A(11,J),J=1,13)/'LONG','ITUD','INAL',' FOR',
     2      'CE F','ROM ','LIVE',' LOA','D   ',4*'    '/
      DATA (A(12,J),J=1,13)/'RIB ','S-OR','TEMP','NG A','ND-D','R SH',
     1      'RINK','AGE ','AND-','GE T','EMPE','RATU','RE  '/
      DATA (A(13,J),J=1,13)/'EART','HQUA','KE  ',10*'    '/,
     1      (A(14,J),J=1,13)/'ICE ','PRES','SURE',10*'    '/
      IF(ILP.GT.1) GO TO 50
```

```
      DO 10 K=1,NJP
      DO 10 J=1,16
   10 PLOAD(K,J)=0.0
      READ (*,1000,END=10000) JPRNT,NLTYP,(LGI(I),I=1,NLPDG)
10000 PRINT 2000,NLPDG
      DO 20 I=1,NLPDG
      I1=LGI(I)
   20 PRINT 2005,ASHTO(I1)
      PRINT 2010,NLTYP
      DO 40 I=1,NLTYP
      READ (*,1000,END=10001)LTI,NJL
10001 IPRA=1
      IF(LTI.EQ.2)IPRB=2
      PRINT 2020,I,NJL,(A(LTI,J),J=1,13)
      DO 30 J=1,NJL
      READ (*,1020,END=10002)IJ,(FOR(L),L=1,6)
10002 PRINT 2030,IJ,(FOR(L),L=1,6)
                JJ=6*IJ-6
      DO 30 L=1,6
      JJ=JJ+1
   30 PLOAD(JJ,LTI)=FOR(L)
   40 CONTINUE
   50 LGP=LGI(ILP)
      DO 70 K=1,NJP
      SUMF(K)=0.0
      DO 70 J=1,16
      FOPLD(K,J)=PLOAD(K,J)*EFAC(LGP,J)*GAMMA(LGP)
   70 SUMF(K)=SUMF(K)+FOPLD(K,J)
      DO 80 K=1,NJP
      SUMF(K)=0.8*SUMF(K)/53.
   80 CONTINUE
      NLOAD=3
 1000 FORMAT(8I,6I10)
 1020 FORMAT(5X,I10,6F10.0)
 2000 FORMAT('1',6X,'THE FOLLOWING',I3,' AASHTO LOAD GROUPS WILL BE',
     1       ' CONSIDERED :',//)
 2005 FORMAT('0',10X,/,'LOAD GROUP',A4)
 2010 FORMAT('0',4X,//,'THERE ARE',I3,' DIFFERENT LOAD TYPES:',///)
 2020 FORMAT('0',//,20X,'  **********  LOAD TYPE',I3,'    ******',
     *  '*****', //,6X,'*',I3,' JOINTS ARE LOADED WITH ',13A4,/////,
     *              20X,'*** INPUT UNFACTORED LOAD VALUES ***',//,
     2       4X,'JOINT NO.',4X,'FORCE-X',4X,'FORCE-Y',4X,'FORCE-Z',
     3       4X,'MOMENT-X',4X,'MOMENT-Y',4X,'MOMENT-Z',//)
 2030 FORMAT('0',4X,I5,4X,3(1X,1PE10.3),3(2X,1PE10.3))
      RETURN
      END
C*P45V1PG*
      SUBROUTINE FS(EP,SS,ETS,ISIG)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      COMMON /L2/ FY,EY,RATIO(10)
      EP=-EP
C     CALCULATION OF STEEL STRESS AND TANGENT STIFFNESS
      IF(EP.LT.-0.00010) GO TO 4
      SS=EY*EP
      ETS=EY
      IF(SS.GT.FY) SS=FY
      IF(SS.GE.FY) ETS=0.0
      GO TO 10
    4 X=ABS(EP/.0001)
      ADD=EY/50577./RATIO(ISIG)
      ADK=EY/2566.
      IF(RATIO(ISIG).LT..005) W=ADK+(ADD-ADK)*(RATIO(ISIG)/.005)
      SS=EP*EY-W*(1.0-1./X)
      ETS=EY+3.0D0.1*ADD/(EP**2)
      IF(SS.LT.-FY) SS=-FY
      IF(SS.LE.-FY) ETS=0.0
 10   EP=-EP
      SS=-SS
      RETURN
C**  THIS PROGRAM VALID ON FTN4 AND FTN5 **
      END
C*P45V1PG*
      SUBROUTINE FC(EP,STRESS,ETC)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      COMMON /L1/ CTL,PK,EC,ENO,EU
C     CONCRETE STRESS AND TANGENT STIFFNESS USING HOGNESTAD STRESS BLOCK
      EUHOG=1.9*ENO
      EP=-EP
      IF(EP) 2,3,4
    2 SC=0.
      ETAN=EC/10**10
      GO TO 1
 3    SC=0.0
      ETAN=EC
      GO TO 1
 4    SC=PK*CTL*(2.0*EP/ENO-(EP/ENO)**2)
      ETAN=PK*CTL*(2.0/ENO-2.0*(EP/ENO)*(1.0/ENO))
      IF(EP.GT.ENO) SC=PK*CTL*(1.0-.15*(EP-ENO)/(EUHOG-ENO))
      IF(EP.GT.ENO) ETAN=-.15*PK*CTL/(EUHOG-ENO)
 1    STRESS=-SC
      ETC=ETAN
      EP=-EP
      RETURN
C**  THIS PROGRAM VALID ON FTN4 AND FTN5 **
      END
C*P45V1PG*
      SUBROUTINE FCON(EP,STRESS,ETC)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      COMMON /L1/ CTL,PK,EC,ENO,EU
C      CONCRETE STRESS AND TANGENT STIFFNESS FOR CONFINED
C         CONCRETE USING FORD STRESS-STRAIN CURVE
      EP=-EP
      IF(EP) 2,3,4
    2 SC=0.
      ETAN=EC/10**10
      GO TO 1
 3    SC=0.0
      ETAN=EC
      GO TO 1
 4    SC=PK*CTL*(2.0*EP/ENO-(EP/ENO)**2)
      ETAN=PK*CTL*(2.0/ENO-2.0*(EP/ENO)*(1.0/ENO))
      IF(EP.GT.ENO) SC=PK*CTL*(1.0-20.0*(EP-ENO))
      IF(EP.GT.ENO) ETAN=-20.0*PK*CTL
 1    STRESS=-SC
      ETC=ETAN
      EP=-EP
      RETURN
C**  THIS PROGRAM VALID ON FTN4 AND FTN5 **
      END
      BLOCK DATA BLDAT
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      CHARACTER*4 ASHTO(10)
      DIMENSION GAMMA(10),EFAC(10,16)
      COMMON /L4/GAMMA,EFAC,IPRA
      COMMON /L11/ ASHTO
      DATA (EFAC(I,1),I=1,10)/1.,9*1./,(EFAC(I,2),I=1,10)/10*.75/
```

```
      DATA (BFAC(I,3),I=1,10)/1.67,2.2,0.,2*1.0,0.,1.,0.,1.,0./
      DATA (BFAC(I,4),I=1,10)/1.,2*0.,2*1.,0.,1.,0.,1.,0./
      DATA (BFAC(I,5),I=1,10)/1.3,0.,8*1.3/,(BFAC(I,6),I=1,10)/.5,0.,
     1     8*.5/
      DATA (BFAC(I,7),I=1,10)/1.,0.,8*1./,(BFAC(I,8),I=1,10)/1.,0.,8*1./
      DATA (BFAC(I,9),I=1,10)/2*0.,1.,.3,0.,1.,.3,2*0.,1./
      DATA (BFAC(I,10),I=1,10)/3*0.,1.,2*0.,1.,3*0./,
     1     (BFAC(I,11),I=1,10)/3*0.,1.,2*0.,1.,3*0./,
     2     (BFAC(I,12),I=1,10)/4*0.,3*1.,3*0./,
     3     (BFAC(I,13),I=1,10)/7*0.,1.,2*0./,
     4     (BFAC(I,14),I=1,10)/8*0.,2*1./
      DATA (GAMMA(I),I=1,10)/5*1.3,2*1.25,2*1.3,1.2/
      DATA ASNTG/'  I ',' IA ',' II ',' III',' IV ','  V ',' VI ',
     1           ' VII','VIII',' IX'/
      END
```

B.6 Example Problem

The following are the input data, explanatory remarks for the input data, and the data output for the example tapered pier shown in Ref. 44. The input will vary slightly for different types of piers.

B.6.1 Input Data for Example Problem

DATA FILE: SAM2LG PAGE 1 OF 1

CARD Nº	CARD TYPE	1	10	20	30	40	50	60	70
1	1	SAMPLE OF TAPERED PIER							
2	2	660.	10	2	0				
3	4	1	1						
4	5	1	11111						
5	6	11	100000.		500000.	20000.		40000.	
6	7	4000.	40000.	.85	360000.	29000000.	.0038		
7	8	SOLID				1			
8	9.a	174.	67.						
9	10.a.1	1	40.0	15.5					
10	10.a.2	4.	4.						
11	9.a	114.	36.						
12	10.a.1	1	16.	5.					
13	10.a.2	4.	4.						
14	11	0							
15	12.a.1	1	5						
16	12.a.2	1	50						
17	12.a.3	11		-200000.		1000000.		2000000.	

B.6.2 Explanatory Remarks for Data Input of Example

Card No.	Card Type	Remarks
1	1	Title
2	2	Member description (linearly tapered pier) all segments same length
3	4	Support conditions (bottom joint supported; top joint has springs)
4	5	Release code of joint 1 (all degrees of freedom fixed)
5	6	Spring stiffnesses of top joint 11
6	7	Material properties
7	8	Section card
8	9.a	Geometry of bottom section
9	10.a.1	Reinforcing steel description
10	10.a.2	Concrete cover
11	9.a	Geometry of top section
12	10.a.1	Reinforcing steel description
13	10.a.2	Concrete cover
14	11	Type of analysis
15	12.a.1	Number of load cases
16	12.a.2	Number of loaded joints and load increments
17	12.a.3	Load description at loaded joints

SAMPLE OF TAPERED PIER

LENGTH OF MEMBER = 600.00
NUMBER OF SEGMENTS= 10
NUMBER OF DIFFERENT TYPE OF SECTIONS= 2

NUMBER OF JOINTS WITH CONSTRAINED
OR SPECIFIED DISPLACEMENTS = 1
NUMBER OF JOINTS WITH SPRINGS = 1

JOINT NO.	RELEASE CODE
1	111111

	****TRANSLATIONAL SPRINGS****			****ROTATIONAL SPRINGS****		
JOINT NO.	X-DIR	Y-DIR	Z-DIR	AROUND X-AXIS	AROUND Y-AXIS	AROUND Z-AXIS
11	.10000E+06	0.	.50000E+06	.20000E+05	0.	.40000E+05

CONCRETE STRENGTH = 4000.00
STEEL YIELD POINT = 60000.00
STRENGTH REDUCTION FACTOR= .85
CONCRETE MODULUS = .3600E+07
STEEL MODULUS = .2900E+08
CONCRETE ULTIMATE STRAIN = .3000E-02
CREEP FACTOR = 0.00

THE PIER SECTION IS SOLID

CONCRETE FIBERS WILL BE MODELLED AS UNCONFINED CONCRETE

STEEL FIBERS ARE GENERATED BY THE PROGRAM

SECTION TYPE 1

Z-DIMENSION= 174.00
Y-DIMENSION= 67.00

CONCRETE FIBER PROPERTIES

NO.	Z	Y	AREA
1	-83.05	29.31	66.24
2	-75.14	29.31	66.24
3	-67.23	29.31	66.24
4	-59.32	29.31	66.24
5	-51.41	29.31	66.24
6	-43.50	29.31	66.24
7	-35.59	29.31	66.24
8	-27.68	29.31	66.24
9	-19.77	29.31	66.24
10	-11.86	29.31	66.24

11	-3.95	29.31	66.24
12	3.95	29.31	66.24
13	11.86	29.31	66.24
14	19.77	29.31	66.24
15	27.68	29.31	66.24
16	35.59	29.31	66.24
17	43.50	29.31	66.24
18	51.41	29.31	66.24
19	59.32	29.31	66.24
20	67.23	29.31	66.24
21	75.14	29.31	66.24
22	83.05	29.31	66.24
23	-83.05	20.94	66.24
24	-75.14	20.94	66.24
25	-67.23	20.94	66.24
26	-59.32	20.94	66.24
27	-51.41	20.94	66.24
28	-43.50	20.94	66.24
29	-35.59	20.94	66.24
30	-27.68	20.94	66.24
31	-19.77	20.94	66.24
32	-11.86	20.94	66.24
33	-3.95	20.94	66.24
34	3.95	20.94	66.24
35	11.86	20.94	66.24
36	19.77	20.94	66.24
37	27.68	20.94	66.24
38	35.59	20.94	66.24
39	43.50	20.94	66.24
40	51.41	20.94	66.24
41	59.32	20.94	66.24
42	67.23	20.94	66.24
43	75.14	20.94	66.24
44	83.05	20.94	66.24
45	-83.05	12.56	66.24
46	-75.14	12.56	66.24
47	-67.23	12.56	66.24
48	-59.32	12.56	66.24
49	-51.41	12.56	66.24
50	-43.50	12.56	66.24
51	-35.59	12.56	66.24
52	-27.68	12.56	66.24
53	-19.77	12.56	66.24
54	-11.86	12.56	66.24
55	-3.95	12.56	66.24
56	3.95	12.56	66.24
57	11.86	12.56	66.24
58	19.77	12.56	66.24
59	27.68	12.56	66.24
60	35.59	12.56	66.24
61	43.50	12.56	66.24
62	51.41	12.56	66.24
63	59.32	12.56	66.24
64	67.23	12.56	66.24
65	75.14	12.56	66.24
66	83.05	12.56	66.24
67	-83.05	4.19	66.24
68	-75.14	4.19	66.24
69	-67.23	4.19	66.24
70	-59.32	4.19	66.24
71	-51.41	4.19	66.24
72	-43.50	4.19	66.24
73	-35.59	4.19	66.24
74	-27.68	4.19	66.24
75	-19.77	4.19	66.24
76	-11.86	4.19	66.24
77	-3.95	4.19	66.24
78	3.95	4.19	66.24
79	11.86	4.19	66.24
80	19.77	4.19	66.24
81	27.68	4.19	66.24
82	35.59	4.19	66.24
83	43.50	4.19	66.24
84	51.41	4.19	66.24
85	59.32	4.19	66.24
86	67.23	4.19	66.24
87	75.14	4.19	66.24
88	83.05	4.19	66.24
89	-83.05	-4.19	66.24
90	-75.14	-4.19	66.24
91	-67.23	-4.19	66.24
92	-59.32	-4.19	66.24
93	-51.41	-4.19	66.24
94	-43.50	-4.19	66.24
95	-35.59	-4.19	66.24
96	-27.68	-4.19	66.24
97	-19.77	-4.19	66.24
98	-11.86	-4.19	66.24
99	-3.95	-4.19	66.24
100	3.95	-4.19	66.24
101	11.86	-4.19	66.24
102	19.77	-4.19	66.24
103	27.68	-4.19	66.24
104	35.59	-4.19	66.24
105	43.50	-4.19	66.24
106	51.41	-4.19	66.24
107	59.32	-4.19	66.24
108	67.23	-4.19	66.24
109	75.14	-4.19	66.24
110	83.05	-4.19	66.24
111	-83.05	-12.56	66.24
112	-75.14	-12.56	66.24
113	-67.23	-12.56	66.24
114	-59.32	-12.56	66.24
115	-51.41	-12.56	66.24
116	-43.50	-12.56	66.24
117	-35.59	-12.56	66.24
118	-27.68	-12.56	66.24
119	-19.77	-12.56	66.24
120	-11.86	-12.56	66.24
121	-3.95	-12.56	66.24
122	3.95	-12.56	66.24
123	11.86	-12.56	66.24
124	19.77	-12.56	66.24
125	27.68	-12.56	66.24
126	35.59	-12.56	66.24
127	43.50	-12.56	66.24
128	51.41	-12.56	66.24
129	59.32	-12.56	66.24
130	67.23	-12.56	66.24
131	75.14	-12.56	66.24
132	83.05	-12.56	66.24
133	-83.05	-20.94	66.24
134	-75.14	-20.94	66.24

135	-67.23	-20.94	66.24
136	-59.32	-20.94	66.24
137	-51.41	-20.94	66.24
138	-43.50	-20.94	66.24
139	-35.59	-20.94	66.24
140	-27.68	-20.94	66.24
141	-19.77	-20.94	66.24
142	-11.86	-20.94	66.24
143	-3.95	-20.94	66.24
144	3.95	-20.94	66.24
145	11.86	-20.94	66.24
146	19.77	-20.94	66.24
147	27.68	-20.94	66.24
148	35.59	-20.94	66.24
149	43.50	-20.94	66.24
150	51.41	-20.94	66.24
151	59.32	-20.94	66.24
152	67.23	-20.94	66.24
153	75.14	-20.94	66.24
154	83.05	-20.94	66.24
155	-83.05	-29.31	66.24
156	-75.14	-29.31	66.24
157	-67.23	-29.31	66.24
158	-59.32	-29.31	66.24
159	-51.41	-29.31	66.24
160	-43.50	-29.31	66.24
161	-35.59	-29.31	66.24
162	-27.68	-29.31	66.24
163	-19.77	-29.31	66.24
164	-11.86	-29.31	66.24
165	-3.95	-29.31	66.24
166	3.95	-29.31	66.24
167	11.86	-29.31	66.24
168	19.77	-29.31	66.24
169	27.68	-29.31	66.24
170	35.59	-29.31	66.24
171	43.50	-29.31	66.24
172	51.41	-29.31	66.24
173	59.32	-29.31	66.24
174	67.23	-29.31	66.24
175	75.14	-29.31	66.24
176	83.05	-29.31	66.24

MOMENT OF INERTIA ABOUT Z-AXIS= .429E+07
MOMENT OF INERTIA ABOUT X-AXIS= .294E+08

NUMBER OF STEEL RINGS= 1

NO.	AREA OF END STEEL	AREA OF SIDE STEEL
1	40.00	15.50

NO.	Z COVER	X COVER
1	4.00	4.00

STEEL FIBER PROPERTIES

NO.	Z	X	AREA
1	-81.85	29.50	.56
2	-79.54	29.50	.56
3	-77.24	29.50	.56
4	-74.93	29.50	.56
5	-72.63	29.50	.56
6	-70.32	29.50	.56
7	-68.01	29.50	.56
8	-65.71	29.50	.56
9	-63.40	29.50	.56
10	-61.10	29.50	.56
11	-58.79	29.50	.56
12	-56.49	29.50	.56
13	-54.18	29.50	.56
14	-51.87	29.50	.56
15	-49.57	29.50	.56
16	-47.26	29.50	.56
17	-44.96	29.50	.56
18	-42.65	29.50	.56
19	-40.35	29.50	.56
20	-38.04	29.50	.56
21	-35.74	29.50	.56
22	-33.43	29.50	.56
23	-31.12	29.50	.56
24	-28.82	29.50	.56
25	-26.51	29.50	.56
26	-24.21	29.50	.56
27	-21.90	29.50	.56
28	-19.60	29.50	.56
29	-17.29	29.50	.56
30	-14.99	29.50	.56
31	-12.68	29.50	.56
32	-10.37	29.50	.56
33	-8.07	29.50	.56
34	-5.76	29.50	.56
35	-3.46	29.50	.56
36	-1.15	29.50	.56
37	1.15	29.50	.56
38	3.46	29.50	.56
39	5.76	29.50	.56
40	8.07	29.50	.56
41	10.38	29.50	.56
42	12.68	29.50	.56
43	14.99	29.50	.56
44	17.29	29.50	.56
45	19.60	29.50	.56
46	21.90	29.50	.56
47	24.21	29.50	.56
48	26.51	29.50	.56
49	28.82	29.50	.56
50	31.13	29.50	.56
51	33.43	29.50	.56
52	35.74	29.50	.56
53	38.04	29.50	.56
54	40.35	29.50	.56
55	42.65	29.50	.56
56	44.96	29.50	.56
57	47.26	29.50	.56
58	49.57	29.50	.56
59	51.88	29.50	.56
60	54.18	29.50	.56
61	56.49	29.50	.56
62	58.79	29.50	.56

63	61.10	29.50	.56
64	63.40	29.50	.56
65	65.71	29.50	.56
66	68.01	29.50	.56
67	70.32	29.50	.56
68	72.63	29.50	.56
69	74.93	29.50	.56
70	77.24	29.50	.56
71	79.54	29.50	.56
72	81.85	29.50	.56
73	-81.85	-29.50	.56
74	-79.54	-29.50	.56
75	-77.24	-29.50	.56
76	-74.93	-29.50	.56
77	-72.63	-29.50	.56
78	-70.32	-29.50	.56
79	-68.01	-29.50	.56
80	-65.71	-29.50	.56
81	-63.40	-29.50	.56
82	-61.10	-29.50	.56
83	-58.79	-29.50	.56
84	-56.49	-29.50	.56
85	-54.18	-29.50	.56
86	-51.87	-29.50	.56
87	-49.57	-29.50	.56
88	-47.26	-29.50	.56
89	-44.96	-29.50	.56
90	-42.65	-29.50	.56
91	-40.35	-29.50	.56
92	-38.04	-29.50	.56
93	-35.74	-29.50	.56
94	-33.43	-29.50	.56
95	-31.12	-29.50	.56
96	-28.82	-29.50	.56
97	-26.51	-29.50	.56
98	-24.21	-29.50	.56
99	-21.90	-29.50	.56
100	-19.60	-29.50	.56
101	-17.29	-29.50	.56
102	-14.99	-29.50	.56
103	-12.68	-29.50	.56
104	-10.37	-29.50	.56
105	-8.07	-29.50	.56
106	-5.76	-29.50	.56
107	-3.46	-29.50	.56
108	-1.15	-29.50	.56
109	1.15	-29.50	.56
110	3.46	-29.50	.56
111	5.76	-29.50	.56
112	8.07	-29.50	.56
113	10.38	-29.50	.56
114	12.68	-29.50	.56
115	14.99	-29.50	.56
116	17.29	-29.50	.56
117	19.60	-29.50	.56
118	21.90	-29.50	.56
119	24.21	-29.50	.56
120	26.51	-29.50	.56
121	28.82	-29.50	.56
122	31.13	-29.50	.56
123	33.43	-29.50	.56
124	35.74	-29.50	.56

125	38.04	-29.50	.56
126	40.35	-29.50	.56
127	42.65	-29.50	.56
128	44.96	-29.50	.56
129	47.26	-29.50	.56
130	49.57	-29.50	.56
131	51.88	-29.50	.56
132	54.18	-29.50	.56
133	56.49	-29.50	.56
134	58.79	-29.50	.56
135	61.10	-29.50	.56
136	63.40	-29.50	.56
137	65.71	-29.50	.56
138	68.01	-29.50	.56
139	70.32	-29.50	.56
140	72.63	-29.50	.56
141	74.93	-29.50	.56
142	77.24	-29.50	.56
143	79.54	-29.50	.56
144	81.85	-29.50	.56
145	-83.00	28.41	.57
146	-83.00	26.22	.57
147	-83.00	24.04	.57
148	-83.00	21.85	.57
149	-83.00	19.67	.57
150	-83.00	17.48	.57
151	-83.00	15.30	.57
152	-83.00	13.11	.57
153	-83.00	10.93	.57
154	-83.00	8.74	.57
155	-83.00	6.56	.57
156	-83.00	4.37	.57
157	-83.00	2.19	.57
158	-83.00	-.00	.57
159	-83.00	-2.19	.57
160	-83.00	-4.37	.57
161	-83.00	-6.56	.57
162	-83.00	-8.74	.57
163	-83.00	-10.93	.57
164	-83.00	-13.11	.57
165	-83.00	-15.30	.57
166	-83.00	-17.48	.57
167	-83.00	-19.67	.57
168	-83.00	-21.85	.57
169	-83.00	-24.04	.57
170	-83.00	-26.22	.57
171	-83.00	-28.41	.57
172	83.00	28.41	.57
173	83.00	26.22	.57
174	83.00	24.04	.57
175	83.00	21.85	.57
176	83.00	19.67	.57
177	83.00	17.48	.57
178	83.00	15.30	.57
179	83.00	13.11	.57
180	83.00	10.93	.57
181	83.00	8.74	.57
182	83.00	6.56	.57
183	83.00	4.37	.57
184	83.00	2.19	.57
185	83.00	-.00	.57
186	83.00	-2.19	.57

187	83.00	-4.37	.57
188	83.00	-6.56	.57
189	83.00	-8.74	.57
190	83.00	-10.93	.57
191	83.00	-13.11	.57
192	83.00	-15.30	.57
193	83.00	-17.48	.57
194	83.00	-19.67	.57
195	83.00	-21.85	.57
196	83.00	-24.04	.57
197	83.00	-26.22	.57
198	83.00	-28.41	.57

SECTION TYPE 2

Z-DIMENSION= 114.00
X-DIMENSION= 36.00

CONCRETE FIBER PROPERTIES

NO.	Z	X	AREA
1	-54.41	15.75	23.32
2	-49.23	15.75	23.32
3	-44.05	15.75	23.32
4	-38.86	15.75	23.32
5	-33.68	15.75	23.32
6	-28.50	15.75	23.32
7	-23.32	15.75	23.32
8	-18.14	15.75	23.32
9	-12.95	15.75	23.32
10	-7.77	15.75	23.32
11	-2.59	15.75	23.32
12	2.59	15.75	23.32
13	7.77	15.75	23.32
14	12.95	15.75	23.32
15	18.14	15.75	23.32
16	23.32	15.75	23.32
17	28.50	15.75	23.32
18	33.68	15.75	23.32
19	38.86	15.75	23.32
20	44.05	15.75	23.32
21	49.23	15.75	23.32
22	54.41	15.75	23.32
23	-54.41	11.25	23.32
24	-49.23	11.25	23.32
25	-44.05	11.25	23.32
26	-38.86	11.25	23.32
27	-33.68	11.25	23.32
28	-28.50	11.25	23.32
29	-23.32	11.25	23.32
30	-18.14	11.25	23.32
31	-12.95	11.25	23.32
32	-7.77	11.25	23.32
33	-2.59	11.25	23.32
34	2.59	11.25	23.32
35	7.77	11.25	23.32
36	12.95	11.25	23.32
37	18.14	11.25	23.32
38	23.32	11.25	23.32
39	28.50	11.25	23.32
40	33.68	11.25	23.32
41	38.86	11.25	23.32
42	44.05	11.25	23.32
43	49.23	11.25	23.32
44	54.41	11.25	23.32
45	-54.41	6.75	23.32
46	-49.23	6.75	23.32
47	-44.05	6.75	23.32
48	-38.86	6.75	23.32
49	-33.68	6.75	23.32
50	-28.50	6.75	23.32
51	-23.32	6.75	23.32
52	-18.14	6.75	23.32
53	-12.95	6.75	23.32
54	-7.77	6.75	23.32
55	-2.59	6.75	23.32
56	2.59	6.75	23.32
57	7.77	6.75	23.32
58	12.95	6.75	23.32
59	18.14	6.75	23.32
60	23.32	6.75	23.32
61	28.50	6.75	23.32
62	33.68	6.75	23.32
63	38.86	6.75	23.32
64	44.05	6.75	23.32
65	49.23	6.75	23.32
66	54.41	6.75	23.32
67	-54.41	2.25	23.32
68	-49.23	2.25	23.32
69	-44.05	2.25	23.32
70	-38.86	2.25	23.32
71	-33.68	2.25	23.32
72	-28.50	2.25	23.32
73	-23.32	2.25	23.32
74	-18.14	2.25	23.32
75	-12.95	2.25	23.32
76	-7.77	2.25	23.32
77	-2.59	2.25	23.32
78	2.59	2.25	23.32
79	7.77	2.25	23.32
80	12.95	2.25	23.32
81	18.14	2.25	23.32
82	23.32	2.25	23.32
83	28.50	2.25	23.32
84	33.68	2.25	23.32
85	38.86	2.25	23.32
86	44.05	2.25	23.32
87	49.23	2.25	23.32
88	54.41	2.25	23.32
89	-54.41	-2.25	23.32
90	-49.23	-2.25	23.32
91	-44.05	-2.25	23.32
92	-38.86	-2.25	23.32
93	-33.68	-2.25	23.32
94	-28.50	-2.25	23.32
95	-23.32	-2.25	23.32
96	-18.14	-2.25	23.32
97	-12.95	-2.25	23.32
98	-7.77	-2.25	23.32
99	-2.59	-2.25	23.32
100	2.59	-2.25	23.32
101	7.77	-2.25	23.32

102	12.95	-2.25	23.32
103	18.14	-2.25	23.32
104	23.32	-2.25	23.32
105	28.50	-2.25	23.32
106	33.68	-2.25	23.32
107	38.86	-2.25	23.32
108	44.05	-2.25	23.32
109	49.23	-2.25	23.32
110	54.41	-2.25	23.32
111	-54.41	-6.75	23.32
112	-49.23	-6.75	23.32
113	-44.05	-6.75	23.32
114	-38.86	-6.75	23.32
115	-33.68	-6.75	23.32
116	-28.50	-6.75	23.32
117	-23.32	-6.75	23.32
118	-18.14	-6.75	23.32
119	-12.95	-6.75	23.32
120	-7.77	-6.75	23.32
121	-2.59	-6.75	23.32
122	2.59	-6.75	23.32
123	7.77	-6.75	23.32
124	12.95	-6.75	23.32
125	18.14	-6.75	23.32
126	23.32	-6.75	23.32
127	28.50	-6.75	23.32
128	33.68	-6.75	23.32
129	38.86	-6.75	23.32
130	44.05	-6.75	23.32
131	49.23	-6.75	23.32
132	54.41	-6.75	23.32
133	-54.41	-11.25	23.32
134	-49.23	-11.25	23.32
135	-44.05	-11.25	23.32
136	-38.86	-11.25	23.32
137	-33.68	-11.25	23.32
138	-28.50	-11.25	23.32
139	-23.32	-11.25	23.32
140	-18.14	-11.25	23.32
141	-12.95	-11.25	23.32
142	-7.77	-11.25	23.32
143	-2.59	-11.25	23.32
144	2.59	-11.25	23.32
145	7.77	-11.25	23.32
146	12.95	-11.25	23.32
147	18.14	-11.25	23.32
148	23.32	-11.25	23.32
149	28.50	-11.25	23.32
150	33.68	-11.25	23.32
151	38.86	-11.25	23.32
152	44.05	-11.25	23.32
153	49.23	-11.25	23.32
154	54.41	-11.25	23.32
155	-54.41	-15.75	23.32
156	-49.23	-15.75	23.32
157	-44.05	-15.75	23.32
158	-38.86	-15.75	23.32
159	-33.68	-15.75	23.32
160	-28.50	-15.75	23.32
161	-23.32	-15.75	23.32
162	-18.14	-15.75	23.32
163	-12.95	-15.75	23.32
164	-7.77	-15.75	23.32
165	-2.59	-15.75	23.32
166	2.59	-15.75	23.32
167	7.77	-15.75	23.32
168	12.95	-15.75	23.32
169	18.14	-15.75	23.32
170	23.32	-15.75	23.32
171	28.50	-15.75	23.32
172	33.68	-15.75	23.32
173	38.86	-15.75	23.32
174	44.05	-15.75	23.32
175	49.23	-15.75	23.32
176	54.41	-15.75	23.32

MOMENT OF INERTIA ABOUT Z-AXIS= .436E+06
MOMENT OF INERTIA ABOUT X-AXIS= .444E+07

NUMBER OF STEEL RINGS= 1

NO.	AREA OF END STEEL	AREA OF SIDE STEEL
1	10.00	5.00

NO.	Z COVER	X COVER
1	4.00	4.00

STEEL FIBER PROPERTIES

NO.	Z	X	AREA
1	-52.26	14.00	.22
2	-50.79	14.00	.22
3	-49.32	14.00	.22
4	-47.85	14.00	.22
5	-46.38	14.00	.22
6	-44.92	14.00	.22
7	-43.43	14.00	.22
8	-41.96	14.00	.22
9	-40.49	14.00	.22
10	-39.01	14.00	.22
11	-37.54	14.00	.22
12	-36.07	14.00	.22
13	-34.60	14.00	.22
14	-33.13	14.00	.22
15	-31.65	14.00	.22
16	-30.18	14.00	.22
17	-28.71	14.00	.22
18	-27.24	14.00	.22
19	-25.76	14.00	.22
20	-24.29	14.00	.22
21	-22.82	14.00	.22
22	-21.35	14.00	.22
23	-19.88	14.00	.22
24	-18.40	14.00	.22
25	-16.93	14.00	.22
26	-15.46	14.00	.22
27	-13.99	14.00	.22
28	-12.51	14.00	.22
29	-11.04	14.00	.22

30	-9.57	14.00	.22
31	-8.10	14.00	.22
32	-6.63	14.00	.22
33	-5.15	14.00	.22
34	-3.68	14.00	.22
35	-2.21	14.00	.22
36	-.74	14.00	.22
37	.74	14.00	.22
38	2.21	14.00	.22
39	3.68	14.00	.22
40	5.15	14.00	.22
41	6.62	14.00	.22
42	8.10	14.00	.22
43	9.57	14.00	.22
44	11.04	14.00	.22
45	12.51	14.00	.22
46	13.99	14.00	.22
47	15.46	14.00	.22
48	16.93	14.00	.22
49	18.40	14.00	.22
50	19.87	14.00	.22
51	21.35	14.00	.22
52	22.82	14.00	.22
53	24.29	14.00	.22
54	25.76	14.00	.22
55	27.24	14.00	.22
56	28.71	14.00	.22
57	30.18	14.00	.22
58	31.65	14.00	.22
59	33.12	14.00	.22
60	34.60	14.00	.22
61	36.07	14.00	.22
62	37.54	14.00	.22
63	39.01	14.00	.22
64	40.49	14.00	.22
65	41.96	14.00	.22
66	43.43	14.00	.22
67	44.90	14.00	.22
68	46.37	14.00	.22
69	47.85	14.00	.22
70	49.32	14.00	.22
71	50.79	14.00	.22
72	52.26	14.00	.22
73	-52.26	-14.00	.22
74	-50.79	-14.00	.22
75	-49.32	-14.00	.22
76	-47.85	-14.00	.22
77	-46.38	-14.00	.22
78	-44.90	-14.00	.22
79	-43.43	-14.00	.22
80	-41.96	-14.00	.22
81	-40.49	-14.00	.22
82	-39.01	-14.00	.22
83	-37.54	-14.00	.22
84	-36.07	-14.00	.22
85	-34.60	-14.00	.22
86	-33.13	-14.00	.22
87	-31.65	-14.00	.22
88	-30.18	-14.00	.22
89	-28.71	-14.00	.22
90	-27.24	-14.00	.22
91	-25.76	-14.00	.22
92	-24.29	-14.00	.22
93	-22.82	-14.00	.22
94	-21.35	-14.00	.22
95	-19.88	-14.00	.22
96	-18.40	-14.00	.22
97	-16.93	-14.00	.22
98	-15.46	-14.00	.22
99	-13.99	-14.00	.22
100	-12.51	-14.00	.22
101	-11.04	-14.00	.22
102	-9.57	-14.00	.22
103	-8.10	-14.00	.22
104	-6.63	-14.00	.22
105	-5.15	-14.00	.22
106	-3.68	-14.00	.22
107	-2.21	-14.00	.22
108	-.74	-14.00	.22
109	.74	-14.00	.22
110	2.21	-14.00	.22
111	3.68	-14.00	.22
112	5.15	-14.00	.22
113	6.62	-14.00	.22
114	8.10	-14.00	.22
115	9.57	-14.00	.22
116	11.04	-14.00	.22
117	12.51	-14.00	.22
118	13.99	-14.00	.22
119	15.46	-14.00	.22
120	16.93	-14.00	.22
121	18.40	-14.00	.22
122	19.87	-14.00	.22
123	21.35	-14.00	.22
124	22.82	-14.00	.22
125	24.29	-14.00	.22
126	25.76	-14.00	.22
127	27.24	-14.00	.22
128	28.71	-14.00	.22
129	30.18	-14.00	.22
130	31.65	-14.00	.22
131	33.12	-14.00	.22
132	34.60	-14.00	.22
133	36.07	-14.00	.22
134	37.54	-14.00	.22
135	39.01	-14.00	.22
136	40.49	-14.00	.22
137	41.96	-14.00	.22
138	43.43	-14.00	.22
139	44.90	-14.00	.22
140	46.37	-14.00	.22
141	47.85	-14.00	.22
142	49.32	-14.00	.22
143	50.79	-14.00	.22
144	52.26	-14.00	.22
145	-53.00	13.48	.19
146	-53.00	12.44	.19
147	-53.00	11.41	.19
148	-53.00	10.37	.19
149	-53.00	9.33	.19
150	-53.00	8.30	.19
151	-53.00	7.26	.19
152	-53.00	6.22	.19
153	-53.00	5.19	.19

154	-53.00	4.15	.10
155	-53.00	3.11	.10
156	-53.00	2.07	.10
157	-53.00	1.04	.10
158	-53.00	.00	.10
159	-53.00	-1.04	.10
160	-53.00	-2.07	.10
161	-53.00	-3.11	.10
162	-53.00	-4.15	.10
163	-53.00	-5.19	.10
164	-53.00	-6.22	.10
165	-53.00	-7.26	.10
166	-53.00	-8.30	.10
167	-53.00	-9.33	.10
168	-53.00	-10.37	.10
169	-53.00	-11.41	.10
170	-53.00	-12.44	.10
171	-53.00	-13.48	.10
172	53.00	13.48	.10
173	53.00	12.44	.10
174	53.00	11.41	.10
175	53.00	10.37	.10
176	53.00	9.33	.10
177	53.00	8.30	.10
178	53.00	7.26	.10
179	53.00	6.22	.10
180	53.00	5.19	.10
181	53.00	4.15	.10
182	53.00	3.11	.10
183	53.00	2.07	.10
184	53.00	1.04	.10
185	53.00	.00	.10
186	53.00	-1.04	.10
187	53.00	-2.07	.10
188	53.00	-3.11	.10
189	53.00	-4.15	.10
190	53.00	-5.19	.10
191	53.00	-6.22	.10
192	53.00	-7.26	.10
193	53.00	-8.30	.10
194	53.00	-9.33	.10
195	53.00	-10.37	.10
196	53.00	-11.41	.10
197	53.00	-12.44	.10
198	53.00	-13.48	.10

NUMBER OF LOAD CASES= 1

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
2						
END-A	-.281E+04	.208E+06	.107E+04	-.365E+06	0.	-.320E+06
END-B	.281E+04	-.208E+06	-.107E+04	.435E+06	0.	.514E+06
3						
END-A	-.281E+04	.208E+06	.107E+04	-.435E+06	0.	-.514E+06
END-B	.281E+04	-.208E+06	-.107E+04	.500E+06	0.	.700E+06
4						
END-A	-.281E+04	.208E+06	.107E+04	-.500E+06	0.	-.700E+06
END-B	.281E+04	-.208E+06	-.107E+04	.576E+06	0.	.886E+06
5						
END-A	-.281E+04	.208E+06	.107E+04	-.576E+06	0.	-.886E+06
END-B	.281E+04	-.208E+06	-.107E+04	.647E+06	0.	.107E+07
6						
END-A	-.281E+04	.208E+06	.107E+04	-.647E+06	0.	-.107E+07
END-B	.281E+04	-.208E+06	-.107E+04	.718E+06	0.	.126E+07
7						
END-A	-.281E+04	.208E+06	.107E+04	-.718E+06	0.	-.126E+07
END-B	.281E+04	-.208E+06	-.107E+04	.780E+06	0.	.144E+07
8						
END-A	-.281E+04	.208E+06	.107E+04	-.780E+06	0.	-.144E+07
END-B	.281E+04	-.208E+06	-.107E+04	.859E+06	0.	.163E+07
9						
END-A	-.281E+04	.208E+06	.107E+04	-.859E+06	0.	-.163E+07
END-B	.281E+04	-.208E+06	-.107E+04	.929E+06	0.	.181E+07
10						
END-A	-.281E+04	.208E+06	.107E+04	-.929E+06	0.	-.181E+07
END-B	.281E+04	-.208E+06	-.107E+04	.100E+07	0.	.200E+07

LOAD INCREMENT 5
****

JOINT NO.	DISPL.-X	DISPL.-Y	DISPL.-Z	ROTATION-X	ROTATION-Y	ROTATION-Z
1	0.	0.	0.	0.	0.	0.
2	-.112E-03	-.148E-02	.284E-04	.897E-06	-.292E-08	.402E-05
3	-.656E-03	-.368E-02	.129E-03	.219E-05	-.631E-08	.132E-04
4	-.204E-02	-.481E-02	.331E-03	.309E-05	-.102E-07	.297E-04
5	-.484E-02	-.609E-02	.679E-03	.640E-05	-.147E-07	.564E-04
6	-.900E-02	-.878E-02	.121E-02	.989E-05	-.196E-07	.901E-04
7	-.184E-01	-.111E-01	.202E-02	.146E-04	-.245E-07	.162E-03
8	-.322E-01	-.137E-01	.319E-02	.218E-04	-.207E-07	.258E-03
9	-.540E-01	-.167E-01	.487E-02	.302E-04	-.290E-07	.406E-03
10	-.885E-01	-.202E-01	.732E-02	.444E-04	-.200E-07	.645E-03
11	-.145E+00	-.241E-01	.110E-01	.678E-04	.250E-07	.186E-02

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
1						
END-A	-.145E+05	.100E+07	.550E+04	-.130E+07	.254E+03	-.577E+06
END-B	.145E+05	-.100E+07	-.550E+04	.170E+07	-.254E+03	.153E+07

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z

LOAD INCREMENT 1
****

JOINT NO.	DISPL.-X	DISPL.-Y	DISPL.-Z	ROTATION-X	ROTATION-Y	ROTATION-Z
1	0.	0.	0.	0.	0.	0.
2	-.254E-04	-.295E-03	.597E-05	.188E-06	0.	.891E-06
3	-.142E-03	-.612E-03	.269E-04	.454E-06	0.	.280E-05
4	-.432E-03	-.956E-03	.686E-04	.821E-06	0.	.614E-05
5	-.101E-02	-.133E-02	.139E-03	.132E-05	0.	.115E-04
6	-.203E-02	-.175E-02	.249E-03	.201E-05	0.	.198E-04
7	-.374E-02	-.221E-02	.411E-03	.294E-05	0.	.323E-04
8	-.648E-02	-.272E-02	.647E-03	.422E-05	0.	.513E-04
9	-.108E-01	-.332E-02	.983E-03	.599E-05	0.	.802E-04
10	-.175E-01	-.401E-02	.146E-02	.849E-05	0.	.125E-03
11	-.281E-01	-.484E-02	.214E-02	.121E-04	0.	.197E-03

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
1						
END-A	-.281E+04	.200E+06	.107E+04	-.294E+06	0.	-.143E+06
END-B	.281E+04	-.200E+06	-.107E+04	.365E+06	0.	.328E+06

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z

2						
END-A	-.145E+05	.100E+07	.550E+04	-.174E+07	.254E+03	-.153E+07
END-B	.145E+05	-.100E+07	-.550E+04	.210E+07	-.253E+03	.248E+07
SEG. NO. 3	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.145E+05	.100E+07	.550E+04	-.210E+07	.253E+03	-.248E+07
END-B	.145E+05	-.100E+07	-.550E+04	.247E+07	-.251E+03	.344E+07
SEG. NO. 4	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.145E+05	.100E+07	.550E+04	-.247E+07	.251E+03	-.344E+07
END-B	.145E+05	-.100E+07	-.550E+04	.283E+07	-.247E+03	.439E+07
SEG. NO. 5	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.145E+05	.100E+07	.550E+04	-.283E+07	.247E+03	-.439E+07
END-B	.145E+05	-.100E+07	-.550E+04	.319E+07	-.239E+03	.534E+07
SEG. NO. 6	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.145E+05	.100E+07	.550E+04	-.319E+07	.239E+03	-.534E+07
END-B	.145E+05	-.100E+07	-.550E+04	.355E+07	-.225E+03	.629E+07
SEG. NO. 7	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.145E+05	.100E+07	.550E+04	-.355E+07	.225E+03	-.629E+07
END-B	.145E+05	-.100E+07	-.550E+04	.392E+07	-.201E+03	.723E+07
SEG. NO. 8	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.145E+05	.100E+07	.550E+04	-.392E+07	.201E+03	-.723E+07
END-B	.145E+05	-.100E+07	-.550E+04	.428E+07	-.163E+03	.816E+07
SEG. NO. 9	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.145E+05	.100E+07	.550E+04	-.428E+07	.163E+03	-.816E+07
END-B	.145E+05	-.100E+07	-.550E+04	.464E+07	-.101E+03	.909E+07
SEG. NO. 10	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.145E+05	.100E+07	.550E+04	-.464E+07	.101E+03	-.909E+07
END-B	.145E+05	-.100E+07	-.550E+04	.500E+07	-.171E-09	.100E+00

LOAD INCREMENT 10
****

JOINT NO.	DISPL.-X	DISPL.-Y	DISPL.-Z	ROTATION-X	ROTATION-Y	ROTATION-Z
1	0.	0.	0.	0.	0.	0.
2	-.205E-03	-.298E-02	.554E-04	.175E-05	-.138E-07	.758E-05
3	-.125E-02	-.619E-02	.252E-03	.438E-05	-.298E-07	.256E-04
4	-.396E-02	-.967E-02	.652E-03	.790E-05	-.483E-07	.585E-04
5	-.953E-02	-.135E-01	.133E-02	.129E-04	-.694E-07	.113E-03
6	-.197E-01	-.177E-01	.241E-02	.197E-04	-.926E-07	.197E-03
7	-.368E-01	-.224E-01	.401E-02	.292E-04	-.116E-06	.327E-03
8	-.648E-01	-.277E-01	.636E-02	.423E-04	-.136E-06	.525E-03
9	-.109E+00	-.338E-01	.976E-02	.612E-04	-.140E-06	.830E-03
10	-.180E+00	-.408E-01	.148E-01	.912E-04	-.883E-07	.133E-02
11	-.296E+00	-.488E-01	.224E-01	.141E-03	.155E-06	.222E-02

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
1						
END-A	-.296E+05	.200E+07	.112E+05	-.265E+07	.119E+04	-.969E+06
END-B	.296E+05	-.200E+07	-.112E+05	.339E+07	-.119E+04	.293E+07

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z

2						
END-A	-.296E+05	.200E+07	.112E+05	-.339E+07	.119E+04	-.293E+07
END-B	.296E+05	-.200E+07	-.112E+05	.412E+07	-.118E+04	.488E+07
SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
3						
END-A	-.296E+05	.200E+07	.112E+05	-.412E+07	.118E+04	-.488E+07
END-B	.296E+05	-.200E+07	-.112E+05	.486E+07	-.117E+04	.683E+07
SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
4						
END-A	-.296E+05	.200E+07	.112E+05	-.486E+07	.117E+04	-.683E+07
END-B	.296E+05	-.200E+07	-.112E+05	.560E+07	-.116E+04	.878E+07
SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
5						
END-A	-.296E+05	.200E+07	.112E+05	-.560E+07	.116E+04	-.878E+07
END-B	.296E+05	-.200E+07	-.112E+05	.634E+07	-.112E+04	.107E+08
SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
6						
END-A	-.296E+05	.200E+07	.112E+05	-.634E+07	.112E+04	-.107E+08
END-B	.296E+05	-.200E+07	-.112E+05	.708E+07	-.105E+04	.126E+08
SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
7						
END-A	-.296E+05	.200E+07	.112E+05	-.708E+07	.105E+04	-.126E+08
END-B	.296E+05	-.200E+07	-.112E+05	.781E+07	-.904E+03	.145E+08
SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
8						
END-A	-.296E+05	.200E+07	.112E+05	-.781E+07	.904E+03	-.145E+08
END-B	.296E+05	-.200E+07	-.112E+05	.854E+07	-.765E+03	.164E+08
SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
9						
END-A	-.296E+05	.200E+07	.112E+05	-.854E+07	.765E+03	-.164E+08
END-B	.296E+05	-.200E+07	-.112E+05	.927E+07	-.477E+03	.183E+08
SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
10						
END-A	-.296E+05	.200E+07	.112E+05	-.927E+07	.477E+03	-.183E+08
END-B	.296E+05	-.200E+07	-.112E+05	.100E+08	-.837E-10	.200E+08

LOAD INCREMENT 15
****

JOINT NO.	DISPL.-X	DISPL.-Y	DISPL.-Z	ROTATION-X	ROTATION-Y	ROTATION-Z
1	0.	0.	0.	0.	0.	0.
2	-.279E-03	-.849E-02	.813E-04	.250E-05	-.337E-07	.105E-04
3	-.177E-02	-.934E-02	.372E-03	.636E-05	-.720E-07	.371E-04
4	-.576E-02	-.146E-01	.964E-03	.117E-04	-.118E-06	.866E-04
5	-.141E-01	-.204E-01	.198E-02	.192E-04	-.170E-06	.169E-03
6	-.293E-01	-.267E-01	.359E-02	.206E-04	-.227E-06	.290E-03
7	-.554E-01	-.330E-01	.601E-02	.440E-04	-.205E-06	.497E-03
8	-.988E-01	-.419E-01	.956E-02	.639E-04	-.333E-06	.842E-03
9	-.166E+00	-.512E-01	.147E-01	.932E-04	-.340E-06	.127E-02
10	-.275E+00	-.620E-01	.224E-01	.148E-03	-.198E-06	.205E-02
11	-.456E+00	-.745E-01	.342E-01	.210E-03	.451E-06	.345E-02

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
1						
END-A	-.456E+05	.380E+07	.171E+05	-.382E+07	.207E+04	-.118E+07
END-B	.456E+05	-.380E+07	-.171E+05	.495E+07	-.207E+04	.419E+07

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z

2						
END-A	-.456E+05	.308E+07	.171E+05	-.495E+07	.287E+04	-.419E+07
END-B	.456E+05	-.308E+07	-.171E+05	.607E+07	-.286E+04	.719E+07

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
3						
END-A	-.456E+05	.308E+07	.171E+05	-.607E+07	.286E+04	-.719E+07
END-B	.456E+05	-.308E+07	-.171E+05	.720E+07	-.284E+04	.102E+08

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
4						
END-A	-.456E+05	.308E+07	.171E+05	-.720E+07	.284E+04	-.102E+08
END-B	.456E+05	-.308E+07	-.171E+05	.833E+07	-.280E+04	.132E+08

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
5						
END-A	-.456E+05	.308E+07	.171E+05	-.833E+07	.280E+04	-.132E+08
END-B	.456E+05	-.308E+07	-.171E+05	.945E+07	-.271E+04	.161E+08

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
6						
END-A	-.456E+05	.308E+07	.171E+05	-.945E+07	.271E+04	-.161E+08
END-B	.456E+05	-.308E+07	-.171E+05	.106E+08	-.255E+04	.191E+08

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
7						
END-A	-.456E+05	.308E+07	.171E+05	-.106E+08	.255E+04	-.191E+08
END-B	.456E+05	-.308E+07	-.171E+05	.117E+08	-.229E+04	.220E+08

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
8						
END-A	-.456E+05	.308E+07	.171E+05	-.117E+08	.229E+04	-.220E+08
END-B	.456E+05	-.308E+07	-.171E+05	.128E+08	-.186E+04	.248E+08

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
9						
END-A	-.456E+05	.308E+07	.171E+05	-.128E+08	.186E+04	-.248E+08
END-B	.456E+05	-.308E+07	-.171E+05	.139E+08	-.116E+04	.275E+08

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
12						
END-A	-.456E+05	.308E+07	.171E+05	-.139E+08	.116E+04	-.275E+08
END-B	.456E+05	-.308E+07	-.171E+05	.150E+08	-.837E-10	.308E+08

LOAD INCREMENT 20
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JOINT NO.	DISPL.-X	DISPL.-Y	DISPL.-Z	ROTATION-X	ROTATION-Y	ROTATION-Z
1	0.	0.	0.	0.	0.	0.
2	-.327E-03	-.682E-02	.105E-03	.335E-05	-.640E-07	.127E-04
3	-.220E-02	-.125E-01	.485E-03	.834E-05	-.139E-06	.475E-04
4	-.738E-02	-.196E-01	.126E-02	.155E-04	-.225E-06	.114E-03
5	-.194E-01	-.273E-01	.261E-02	.255E-04	-.324E-06	.224E-03
6	-.385E-01	-.359E-01	.474E-02	.345E-04	-.434E-06	.408E-03
7	-.739E-01	-.455E-01	.707E-02	.560E-04	-.546E-06	.672E-03
8	-.132E+00	-.564E-01	.127E-01	.859E-04	-.636E-06	.109E-02
9	-.225E+00	-.690E-01	.197E-01	.126E-03	-.646E-06	.174E-02
10	-.375E+00	-.838E-01	.302E-01	.192E-03	-.343E-06	.283E-02
11	-.624E+00	-.101E+00	.464E-01	.302E-03	.104E-05	.479E-02

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
1						
END-A	-.624E+05	.400E+07	.232E+05	-.487E+07	.541E+04	-.116E+07
END-B	.624E+05	-.400E+07	-.232E+05	.648E+07	-.541E+04	.528E+07

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z

2						
END-A	-.624E+05	.400E+07	.232E+05	-.660E+07	.501E+04	-.520E+07
END-B	.624E+05	-.400E+07	-.232E+05	.703E+07	-.540E+04	.939E+07

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
3						
END-A	-.624E+05	.400E+07	.232E+05	-.703E+07	.540E+04	-.939E+07
END-B	.624E+05	-.400E+07	-.232E+05	.946E+07	-.536E+04	.135E+08

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
4						
END-A	-.624E+05	.400E+07	.232E+05	-.946E+07	.536E+04	-.135E+08
END-B	.624E+05	-.400E+07	-.232E+05	.110E+08	-.520E+04	.176E+08

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
5						
END-A	-.624E+05	.400E+07	.232E+05	-.110E+08	.520E+04	-.176E+08
END-B	.624E+05	-.400E+07	-.232E+05	.125E+08	-.512E+04	.216E+08

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
6						
END-A	-.624E+05	.400E+07	.232E+05	-.125E+08	.512E+04	-.216E+08
END-B	.624E+05	-.400E+07	-.232E+05	.140E+08	-.402E+04	.256E+08

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
7						
END-A	-.624E+05	.400E+07	.232E+05	-.140E+08	.402E+04	-.256E+08
END-B	.624E+05	-.400E+07	-.232E+05	.155E+08	-.433E+04	.295E+08

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
8						
END-A	-.624E+05	.400E+07	.232E+05	-.155E+08	.433E+04	-.295E+08
END-B	.624E+05	-.400E+07	-.232E+05	.170E+08	-.352E+04	.333E+08

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
9						
END-A	-.624E+05	.400E+07	.232E+05	-.170E+08	.352E+04	-.333E+08
END-B	.624E+05	-.400E+07	-.232E+05	.185E+08	-.221E+04	.368E+08

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
10						
END-A	-.624E+05	.400E+07	.232E+05	-.185E+08	.221E+04	-.368E+08
END-B	.624E+05	-.400E+07	-.232E+05	.200E+08	-.224E-00	.400E+08

LOAD INCREMENT 25
****

JOINT NO.	DISPL.-X	DISPL.-Y	DISPL.-Z	ROTATION-X	ROTATION-Y	ROTATION-Z
1	0.	0.	0.	0.	0.	0.
2	-.341E-03	-.757E-02	.127E-03	.406E-05	-.107E-06	.139E-04
3	-.251E-02	-.158E-01	.589E-03	.102E-04	-.232E-06	.561E-04
4	-.876E-02	-.246E-01	.154E-02	.100E-04	-.377E-06	.138E-03
5	-.223E-01	-.344E-01	.320E-02	.316E-04	-.543E-06	.278E-03
6	-.478E-01	-.452E-01	.566E-02	.402E-04	-.728E-06	.542E-03
7	-.921E-01	-.573E-01	.949E-02	.737E-04	-.916E-06	.851E-03
8	-.166E+00	-.712E-01	.159E-01	.108E-03	-.107E-05	.139E-02
9	-.284E+00	-.873E-01	.247E-01	.160E-03	-.100E-05	.223E-02
10	-.478E+00	-.106E+00	.381E-01	.247E-03	-.523E-06	.366E-02
11	-.803E+00	-.129E+00	.591E-01	.347E-03	.214E-05	.626E-02

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
1						
END-A	-.803E+05	.500E+07	.296E+09	-.577E+07	.893E+04	-.847E+06
END-B	.803E+05	-.500E+07	-.296E+09	.772E+07	-.893E+04	.615E+07

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z

SEG. NO.		FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
2	END-A	-.803E+05	.580E+07	.296E+05	-.772E+07	.893E+04	-.615E+07
	END-B	.803E+05	-.580E+07	-.296E+05	.967E+07	-.891E+04	.114E+08
3	END-A	-.803E+05	.580E+07	.296E+05	-.967E+07	.891E+04	-.114E+08
	END-B	.803E+05	-.580E+07	-.296E+05	.116E+08	-.886E+04	.167E+08
4	END-A	-.803E+05	.580E+07	.296E+05	-.116E+08	.886E+04	-.167E+08
	END-B	.803E+05	-.580E+07	-.296E+05	.136E+08	-.872E+04	.219E+08
5	END-A	-.803E+05	.580E+07	.296E+05	-.136E+08	.872E+04	-.219E+08
	END-B	.803E+05	-.580E+07	-.296E+05	.155E+08	-.846E+04	.271E+08
6	END-A	-.803E+05	.580E+07	.296E+05	-.155E+08	.846E+04	-.271E+08
	END-B	.803E+05	-.580E+07	-.296E+05	.174E+08	-.790E+04	.322E+08
7	END-A	-.803E+05	.580E+07	.296E+05	-.174E+08	.790E+04	-.322E+08
	END-B	.803E+05	-.580E+07	-.296E+05	.194E+08	-.717E+04	.372E+08
8	END-A	-.803E+05	.580E+07	.296E+05	-.194E+08	.717E+04	-.372E+08
	END-B	.803E+05	-.580E+07	-.296E+05	.213E+08	-.584E+04	.419E+08
9	END-A	-.803E+05	.580E+07	.296E+05	-.213E+08	.584E+04	-.419E+08
	END-B	.803E+05	-.580E+07	-.296E+05	.231E+08	-.367E+04	.463E+08
10	END-A	-.803E+05	.580E+07	.296E+05	-.231E+08	.367E+04	-.463E+08
	END-B	.803E+05	-.580E+07	-.296E+05	.250E+08	-.189E-00	.500E+08

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LOAD INCREMENT   30
******************

JOINT NO.    DISPL.-X    DISPL.-Y    DISPL.-Z   ROTATION-X  ROTATION-Y  ROTATION-Z
      1      0.          0.          0.          0.          0.          0.

JOINT NO.    DISPL.-X    DISPL.-Y    DISPL.-Z   ROTATION-X  ROTATION-Y  ROTATION-Z
      2      -.387E-03   -.914E-02   .146E-03    .467E-05   -.160E-06    .138E-04

JOINT NO.    DISPL.-X    DISPL.-Y    DISPL.-Z   ROTATION-X  ROTATION-Y  ROTATION-Z
      3      -.264E-02   -.190E-01   .602E-03    .119E-04   -.357E-06    .623E-04

JOINT NO.    DISPL.-X    DISPL.-Y    DISPL.-Z   ROTATION-X  ROTATION-Y  ROTATION-Z
      4      -.977E-02   -.298E-01   .180E-02    .224E-04   -.582E-06    .160E-03

JOINT NO.    DISPL.-X    DISPL.-Y    DISPL.-Z   ROTATION-X  ROTATION-Y  ROTATION-Z
      5      -.257E-01   -.416E-01   .376E-02    .374E-04   -.840E-06    .330E-03

JOINT NO.    DISPL.-X    DISPL.-Y    DISPL.-Z   ROTATION-X  ROTATION-Y  ROTATION-Z
      6      -.561E-01   -.547E-01   .691E-02    .586E-04   -.113E-05    .603E-03

JOINT NO.    DISPL.-X    DISPL.-Y    DISPL.-Z   ROTATION-X  ROTATION-Y  ROTATION-Z
      7      -.118E+00   -.694E-01   .117E-01    .864E-04   -.142E-05    .103E-02

JOINT NO.    DISPL.-X    DISPL.-Y    DISPL.-Z   ROTATION-X  ROTATION-Y  ROTATION-Z
      8      -.199E+00   -.863E-01   .189E-01    .131E-03   -.166E-05    .170E-02

JOINT NO.    DISPL.-X    DISPL.-Y    DISPL.-Z   ROTATION-X  ROTATION-Y  ROTATION-Z
      9      -.345E+00   -.106E+00   .296E-01    .166E-03   -.167E-05    .275E-02

JOINT NO.    DISPL.-X    DISPL.-Y    DISPL.-Z   ROTATION-X  ROTATION-Y  ROTATION-Z
     10      -.585E+00   -.130E+00   .461E-01    .304E-03   -.731E-06    .456E-02

JOINT NO.    DISPL.-X    DISPL.-Y    DISPL.-Z   ROTATION-X  ROTATION-Y  ROTATION-Z
     11      -.994E+00   -.159E+00   .726E-01    .505E-03    .416E-05    .794E-02



SEG. NO.       FORCE-X     FORCE-Y     FORCE-Z    MOMENT-X    MOMENT-Y    MOMENT-Z
    1
   END-A     -.994E+05    .600E+07    .363E+05   -.647E+07    .136E+05   -.107E+06

   END-B      .994E+05   -.600E+07   -.363E+05    .807E+07   -.136E+05    .671E+07

SEG. NO.       FORCE-X     FORCE-Y     FORCE-Z    MOMENT-X    MOMENT-Y    MOMENT-Z
```

2						
END-A	-.994E+05	.608E+07	.363E+05	-.887E+07	.136E+05	-.671E+07
END-B	.994E+05	-.608E+07	-.363E+05	.113E+08	-.136E+05	.133E+08
SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
3						
END-A	-.994E+05	.608E+07	.363E+05	-.113E+08	.136E+05	-.133E+08
END-B	.994E+05	-.608E+07	-.363E+05	.136E+08	-.135E+05	.198E+08
SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
4						
END-A	-.994E+05	.608E+07	.363E+05	-.136E+08	.135E+05	-.198E+08
END-B	.994E+05	-.608E+07	-.363E+05	.160E+08	-.133E+05	.262E+08
SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
5						
END-A	-.994E+05	.608E+07	.363E+05	-.160E+08	.133E+05	-.262E+08
END-B	.994E+05	-.608E+07	-.363E+05	.184E+08	-.129E+05	.326E+08
SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
6						
END-A	-.994E+05	.608E+07	.363E+05	-.184E+08	.129E+05	-.326E+08
END-B	.994E+05	-.608E+07	-.363E+05	.208E+08	-.122E+05	.389E+08
SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
7						
END-A	-.994E+05	.608E+07	.363E+05	-.208E+08	.122E+05	-.389E+08
END-B	.994E+05	-.608E+07	-.363E+05	.231E+08	-.110E+05	.449E+08
SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
8						
END-A	-.994E+05	.608E+07	.363E+05	-.231E+08	.110E+05	-.449E+08
END-B	.994E+05	-.608E+07	-.363E+05	.255E+08	-.895E+04	.506E+08
SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
9						
END-A	-.994E+05	.608E+07	.363E+05	-.255E+08	.895E+04	-.506E+08
END-B	.994E+05	-.608E+07	-.363E+05	.278E+08	-.503E+04	.550E+08
SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
10						
END-A	-.994E+05	.608E+07	.363E+05	-.278E+08	.503E+04	-.550E+08
END-B	.994E+05	-.608E+07	-.363E+05	.304E+08	-.530E-08	.600E+08

LOAD INCREMENT 35
****

JOINT NO.	DISPL.-X	DISPL.-Y	DISPL.-Z	ROTATION-X	ROTATION-Y	ROTATION-Z
1	0.	0.	0.	0.	0.	0.
2	-.191E-03	-.107E-01	.158E-03	.509E-05	-.243E-06	.113E-04
3	-.245E-02	-.223E-01	.740E-03	.131E-04	-.527E-06	.640E-04
4	-.101E-01	-.350E-01	.200E-02	.252E-04	-.861E-06	.176E-03
5	-.279E-01	-.489E-01	.921E-02	.825E-04	-.125E-05	.374E-03
6	-.628E-01	-.643E-01	.782E-02	.673E-04	-.160E-05	.697E-03
7	-.125E+00	-.817E-01	.134E-01	.102E-03	-.213E-05	.121E-02
8	-.231E+00	-.102E+00	.218E-01	.153E-03	-.254E-05	.201E-02
9	-.405E+00	-.125E+00	.348E-01	.232E-03	-.252E-05	.329E-02
10	-.694E+00	-.154E+00	.508E-01	.366E-03	-.102E-05	.553E-02
11	-.120E+01	-.191E+00	.872E-01	.655E-03	.851E-05	.104E-01

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
1						
END-A	-.120E+06	.700E+07	.436E+05	-.682E+07	.198E+05	.120E+07
END-B	.120E+06	-.700E+07	-.436E+05	.970E+07	-.198E+05	.674E+07

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z

2						
END-A	-.120E+06	.700E+07	.436E+05	-.970E+07	.198E+05	-.674E+07
END-B	.120E+06	-.700E+07	-.436E+05	.126E+08	-.198E+05	.147E+08
SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
3						
END-A	-.120E+06	.700E+07	.436E+05	-.126E+08	.198E+05	-.147E+08
END-B	.120E+06	-.700E+07	-.436E+05	.154E+08	-.197E+05	.225E+08
SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
4						
END-A	-.120E+06	.700E+07	.436E+05	-.154E+08	.197E+05	-.225E+08
END-B	.120E+06	-.700E+07	-.436E+05	.183E+08	-.194E+05	.304E+08
SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
5						
END-A	-.120E+06	.700E+07	.436E+05	-.183E+08	.194E+05	-.304E+08
END-B	.120E+06	-.700E+07	-.436E+05	.212E+08	-.189E+05	.381E+08
SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
6						
END-A	-.120E+06	.700E+07	.436E+05	-.212E+08	.189E+05	-.381E+08
END-B	.120E+06	-.700E+07	-.436E+05	.240E+08	-.178E+05	.456E+08
SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
7						
END-A	-.120E+06	.700E+07	.436E+05	-.240E+08	.178E+05	-.456E+08
END-B	.120E+06	-.700E+07	-.436E+05	.268E+08	-.161E+05	.528E+08
SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
8						
END-A	-.120E+06	.700E+07	.436E+05	-.268E+08	.161E+05	-.528E+08
END-B	.120E+06	-.700E+07	-.436E+05	.296E+08	-.131E+05	.596E+08
SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
9						
END-A	-.120E+06	.700E+07	.436E+05	-.296E+08	.131E+05	-.596E+08
END-B	.120E+06	-.700E+07	-.436E+05	.323E+08	-.829E+04	.655E+08
SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
10						
END-A	-.120E+06	.700E+07	.436E+05	-.323E+08	.829E+04	-.655E+08
END-B	.120E+06	-.700E+07	-.436E+05	.350E+08	-.724E-08	.700E+08

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THE PIER HAD A CONCRETE COMPRESSION FAILURE
*******************************************

   IN SECTION    10
   OF SEGMENT    10


   IN LOAD INCREMENT    60
   OF LOAD CASE     1



       JUST BEFORE FAILURE
       *******************

MAX. COMPRESSIVE CONCRETE STRAIN= -.370E-02
OCCURED IN LOAD CASE            =    1
OCCURED IN INCREMENT            =   60
OCCURED IN SEGMENT              =   10
OCCURED IN SECTION              =   10

MAX. TENSILE STEEL STRAIN=  .153E-02
OCCURED IN LOAD CASE     =    1
OCCURED IN INCREMENT     =   60
OCCURED IN SEGMENT       =   10
OCCURED IN  SECTION      =    9
```

THE FINAL DISPLACEMENTS AND FORCES BEFORE FAILURE

JOINT NO.	DISPL.-X	DISPL.-Y	DISPL.-Z	ROTATION-X	ROTATION-Y	ROTATION-Z
1	0.	0.	0.	0.	0.	0.
2	.126E-03	-.120E-01	.150E-03	.491E-05	-.336E-06	.269E-05
3	-.142E-02	-.258E-01	.732E-03	.131E-04	-.733E-06	.528E-04
4	-.838E-02	-.392E-01	.280E-02	.258E-04	-.128E-05	.169E-03
5	-.262E-01	-.548E-01	.430E-02	.445E-04	-.175E-05	.383E-03
6	-.628E-01	-.721E-01	.811E-02	.716E-04	-.237E-05	.742E-03
7	-.130E+00	-.917E-01	.141E-01	.111E-03	-.348E-05	.132E-02
8	-.246E+00	-.110E+00	.232E-01	.167E-03	-.363E-05	.223E-02
9	-.441E+00	-.141E+00	.371E-01	.258E-03	-.370E-05	.371E-02
10	-.771E+00	-.174E+00	.592E-01	.416E-03	-.179E-05	.635E-02
11	-.148E+01	-.226E+00	.102E+00	.924E-03	.228E-04	.135E-01

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
1						
END-A	-.188E+06	.788E+07	.589E+05	-.621E+07	.272E+05	.398E+07
END-B	.188E+06	-.788E+07	-.589E+05	.957E+07	-.272E+05	.528E+07

SEG. NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
2						

END-A	-.140E+06	.780E+07	.509E+05	-.957E+07	.272E+05	-.528E+07
END-B	.140E+06	-.780E+07	-.509E+05	.129E+08	-.272E+05	.145E+08
SEG. NO. 3	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.140E+06	.780E+07	.509E+05	-.129E+08	.272E+05	-.145E+08
END-B	.140E+06	-.780E+07	-.509E+05	.163E+08	-.270E+05	.238E+08
SEG. NO. 4	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.140E+06	.780E+07	.509E+05	-.163E+08	.270E+05	-.238E+08
END-B	.140E+06	-.780E+07	-.509E+05	.196E+08	-.267E+05	.329E+08
SEG. NO. 5	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.140E+06	.780E+07	.509E+05	-.196E+08	.267E+05	-.329E+08
END-B	.140E+06	-.780E+07	-.509E+05	.229E+08	-.259E+05	.419E+08
SEG. NO. 6	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.140E+06	.780E+07	.509E+05	-.229E+08	.259E+05	-.419E+08
END-B	.142E+06	-.780E+07	-.509E+05	.262E+08	-.245E+05	.506E+08
SEG. NO. 7	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.140E+06	.780E+07	.509E+05	-.262E+08	.245E+05	-.506E+08
END-B	.140E+06	-.780E+07	-.509E+05	.295E+08	-.221E+05	.598E+08
SEG. NO. 8	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.140E+06	.780E+07	.509E+05	-.295E+08	.221E+05	-.598E+08
END-B	.140E+06	-.780E+07	-.509E+05	.328E+08	-.182E+05	.690E+08
SEG. NO. 9	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.140E+06	.780E+07	.509E+05	-.328E+08	.182E+05	-.690E+08
END-B	.140E+06	-.780E+07	-.509E+05	.360E+08	-.115E+05	.735E+08
SEG. NO. 10	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.140E+06	.780E+07	.509E+05	-.360E+08	.115E+05	-.735E+08
END-B	.140E+06	-.780E+07	-.509E+05	.390E+08	-.631E-00	.780E+08

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19 NOV 82 UNIVERSITY OF TEXAS 170/750B UT2D-242

17,20,33 NEW JOB - SUBMIT CC=F2 ID=57 PR=300 TM=500 DJ
22,20.10 -    520ACM     0.000CP      0MS       0MT
22,20.12 +   200ARCM     0.002CP     19MS       0MT
22,20.12 READPF C934 PIER77B SAM2LG
22,20.23  COPIED FILE PIER77B.
22,20.24  COPIED FILE SAM2LG.
22,20.25 +   240PMCM     0.167CP    660MS       0MT
22,20.25 PIER77B SAM2LG
22,20.25 +   700ABCM     0.193CP    762MS       0MT
22,20.26 +   744RACM     0.227CP    838MS       0MT
22,20.26 + 1000PPCM     0.243CP    874MS       0MT
22,20.27 + 1040PPCM     0.258CP    917MS       0MT
22,20.27 + 1100ADCM     0.281CP    925MS       0MT
22,20.29 + 1140PPCM     0.349CP   1156MS       0MT
22,20.30 + 1200APCM     0.378CP   1220MS       0MT
22,20.32 + 1240PPCM     0.491CP   1413MS       0MT
22,20.33    CM LWA+1 =1015558, LOADER USED 117600B
22,20.33 - 1016PPCM     0.521CP   1429MS       0MT
22,20.33 0 1016PUCM     0.522CP   1429MS       0MT
22,20.55 + 1036PPCM     0.523CP   1429MS       0MT
22,20.55 + 1063PPCM     0.525CP   1445MS       0MT
22,20.55 - 1062PPCM     0.526CP   1463MS       0MT
22,22.56 - 1042PPCM     0.529CP   1463MS       0MT
22,20.56 + 1075PPCM     0.531CP   1480MS       0MT
22,20.56 + 1116PUCM     0.533CP   1488MS       0MT
22,20.56 - 1115PPCM     0.535CP   1540MS       0MT
22,20.56 - 1076PPCM     0.537CP   1506MS       0MT
22,20.57 + 1126PPCM     0.541CP   1530MS       0MT
22,39.57 - 1042PPCM   154.820CP   1689MS       0MT
22,39.57      STOP
22,39.57      112600 MAXIMUM EXECUTION FL.
22,39.57     247.708 CP SECONDS EXECUTION TIME.
22,39.57 -    470PCM  154.821CP   1689MS       0MT
22,39.57  FILES ENTERED IN SYSTEM QUEUES:
22,39.57   OUTPUT
22,39.58  JOB COMPLETED.
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A P P E N D I X C

PROGRAM FPIER

C.1 General

Identification. The computer program discussed within this appendix, entitled FPIER, is written in FORTRAN 77 computer language.

Application. The primary objective of the program is the structural analysis of geometrically symmetrical multiple bay and/or multiple story reinforced concrete pier bents subjected to static loads.

Coordinate System and Sign Convention. The positive axes for the piers are shown in Fig. C.5. Forces and displacements are positive if in direction shown in Fig. C.7. Moments and rotations are positive if given by the right hand rule as in Fig. C.7.

C.2 Definition of Main Variables

AG	= gross area of section
AKFX(12)	= vector of translational spring stiffnesses in the x-direction at joints
AKFY(12)	= vector of translational spring stiffnesses in the y-direction at joints
AKFZ(12)	= vector of translational spring stiffnesses in the z-direction
AKX(12)	= vector of rotational spring stiffnesses about the x-axis at joints
AKY(12)	= vector of rotational spring stiffnesses about the z-axis at joints
AMAT(3,3)	= A-matrix for a segment
AST	= area of reinforcing steel
BETA	= creep factor
BMAT(3,3,150)	= B-matrix of all sections in a member
BW(2)	= bay spans
CFA(4000)	= matrix containing the area of all concrete fibers in a member
CONFX(4000)	= matrix containing x-centroid of all concrete fibers in a member

CONFZ(4000)	= matrix containing z-centroid of all concrete fibers in a member
CTROID(5)	= distance from centroid of section to interior walls
CYL	= compressive cylinder strength of concrete
DA_{11}, DA_{12}, . . ., DA_{13}	= stiffness parameters of section
DFORA(6,60), DFORB(6,60)	= auxiliary matrices containing member end forces
DU(72)	= matrix containing the incremental deformations of each joint
DUA(6), DUB(6)	= auxiliary vector containing segment forces
EC	= initial modulus of concrete
ENO	= strain corresponding to maximum stress on concrete compression stress-strain curve
EPS(150)	= contains axial strain for a section
EU	= ultimate strength of concrete
EY	= steel modulus
EZC(200)	= array containing total strain of a concrete fiber of a given section
EZS(200)	= array containing total strain of a steel fiber of a given section
FIY(150)	= contains rotational strain about the y-axis of a section
FIZ(150)	= contains rotational strain about the z-axis of a section
FOR(6)	= auxiliary array to read forces or displacements
FORA(6,60)	= contains member end-A forces
FORB(6,60)	= contains member end-B forces
FY	= yield stress of steel
GAMMAX(3)	= x-distance from end edge of section to centroid of top or bottom steel
GAMMAX(3)	= z-distance from side edge of section to centroid of side steel

H(3)	= column heights per level
HH(10,9)	= lengths of segments per member
ICODE	= code to indicate if hollow section has interior walls
ICON	= code to define if concrete is unconfined or confined
ICON(9)	= code to define if concrete is unconfined or confined per member
INSECT(9)	= number of sections per member
INSEG(9)	= number of segments per member
IOR	= orientation of interior walls; either horizontal or vertical
IPRNT	= print option of section fibers
ISTAT	= code to indicate if program will generate steel fibers or if the user will read in individual reinforcement properties
ITITLE(20)	= title of problem
ITYPE	= type of section (solid, hollow, oval or circular hollow)
JPRNT	= code to indicate which incremental results will be printed
JR(12)	= vector of joint release codes of nodal points
LGI	= identification of load factor groups
LTI	= load type
NCJ	= number of constrained joints
NDF	= number of degrees of freedom
NFIB	= number of concrete fibers in a section
NJ	= number of joints
NJL	= number of loaded joints
NJSP	= number of joints with springs
NLFDG	= number of load factor design groups
NLINC	= number of load increments
NLOAD	= number of load cases
NLTYP	= number of different load types
NRING	= number of rings of steel
NSECT	= number of sections
NSEG	= number of segments

NSTOT	= number of steel fibers in a section
NW	= number of interior walls in hollow section
PDIS(72)	= vector of applied displacement increments
PFOR(72)	= vector of applied force increments
RATIO(10)	= ratio of reinforcement area to section area
RK	= reduction factor applied to f'_c
ROT(6,6)	= rotation matrix
SA(6,6,60), SB(6,6,60), SC(6,6,60)	= auxiliary matrices containing stiffnesses of members
SKAA(6,6,60), SKAB(6,6,60), SKBA(6,6,60), SKBB(6,6,60)	= flexibility or stiffness of matrices of segments
STBB(6,6), STAA(6,6) STAB(6,6), STBA (6,6)	= flexibility of matrices or stiffnesses matrices of a segment at a given load increment
STK(72,72)	= total stiffness of matrix of a member
STLA(2000)	= matrix containing the area of all steel fibers
STLX(2000)	= matrix containing the x-centroid of all steel fibers in a member
STLZ(2000)	= matrix containing the z-centroid of all steel fibers in a member
STS(3)	= area of steel of one side of section
STT(3)	= area of top or bottom steel of section
TF	= thickness of flange on hollow section
TRAT	= torsional constant
TW	= thickness of web on hollow section
TWALL(5)	= thickness of interior walls
U(72)	= vector of joint deformations

WLSTL(5)	=	area of steel interior walls
XI	=	depth of section (dimension along x-axis)
XIX	=	moment of inertia about the x-axis
ZI	=	width of section (dimension along z-axis)
ZIZ	=	moment of inertia about the z-axis

C.3 Guide to Input Data

The program data are input by means of data cards. No units are implied. The user must use consistent units. See Figs. C.1 through C.7 for sign conventions and section descriptions. F-fields are for real numbers and data can be placed anywhere within the field. A decimal point must be used. I-fields are for integers and are right justified. A-fields are for alphanumeric designations and are left justified.

1. Title Card (20A4)

   Column 1-80--ITITLE(20), title of the problem. Any FORTRAN characters are acceptable.

2. Pier Bent Geometry (3I10) (See Fig. C.1)

   Column 1-10--NCOL, number of column lines (maximum of 3)

   Column 11-20--NLVL, number of levels (maximum of 3)

3. Pier Dimensions (8F10.0) (from bottom to top, see Fig. C.1)

   Column 1-10--H(1), height of piers at level one

   Colume 11-20--H(2), height of piers at level two

4. Bay Spans (8F10.0) (from left to right, see Fig. C.1)

   Column 1-10--BW(1), span of first bay

   Colume 11-20--BW(2), span of second bay

5. Support Conditions (3I10)

   Column 1-10--NCJ, total number of constrained joints (supports)

   Column 11-20--NJSP, total number of joints with springs

   (Constrained joints and joints with springs are limited to joints at member ends)

6. Release Code of Joints (3I10)

   Only if NCJ ≠ 0; one card for each constrained joint

   Column 1-10--JR(J), joint release code of joint J;
   1 for a degree of freedom which is fixed;
   0 for a degree of freedom which is not fixed;
   (Column 15--x-translation; Column 16--y-translation; Column 17--z-translation; Column 18--rotation about x-axis; Column 19--rotation about y-axis; Column 20--rotation about z-axis)

7. Spring Information (I10,7F10.0)

Only if NJSP ≠ 0; one card for each joint with springs.

Column 1-10--J, number of the joint which has springs (see Fig. C.2 for joint numbering scheme)

Column 11-20--AKX(J), stiffness of x-translational spring

Column 21-30--AKY(J), stiffness of y-translational spring

Column 31-40--AKZ(J), stiffness of z-translational spring

Column 41-50--AKFX(J), stiffness of x-rotational spring

Column 51-60--AKFY(J), stiffness of y-rotational spring

Column 61-70--AKFZ(J), stiffness of z-rotational spring

8. Material Properties (8F10.0)

Column 1-10--CYL, compressive cylinder strength of concrete

Column 11-20--FY, yield stress of reinforcing steel

Column 21-30--RK, strength reduction factor for concrete (i.e., $f''_c = 0.85f'_c$; see Sec. 4.5.3)

Column 31-40--EC, initial modulus of concrete

Column 41-50--EY, modulus of reinforcing steel

Column 51-60--EU, ultimate strain of concrete; if left blank, program assumes EU = 1.9 (1.0 + BETA) RK (2.0 CYL/EC) (See Sec. 4.5.3)

Column 61-70--BETA, creep factor (see Table C.1)

Characteristics of Members. The program assumes geometric symmetry of the pier bent. The mirror image symmetrical members which are shown dashed in Fig. C.3 are not read in. Cards 9 through 14 must be included for each solid member in Fig. C.3. Columns are read first from bottom to top, and left to right. Beams are read afterwards from left to right, and bottom to top. (See Fig. C.3.)

9. Segments and Sections (3I10)

Note: The maximum number of segments in the entire structure is 150 (10 segments for each of 15 members). However, the maximum number of sections which are the segment subdivisions in the entire structure is also 150. Therefore, for frames with large numbers of members the columns should be allocated more of the segments and the beams should be allocated fewer segments to preserve accuracy. The number of segments should not be less than 5 for columns and 3 for beams.

Column 1-10--INSEG(I), number of segments of member IN (maximum of 10 in any member)

Column 11-20--INSECT(IN), number of different type of cross sections required to define the geometry of member IN (maximum of 10 in any member)

10. Segment Lengths (15F5.0)

Only if INSEG(IN) > 1

Column 1- 5--HH(1,IN), length of first segment of member IN

Column 6-10--HH(2,IN), length of second segment of member IN

Column 11-15--HH(3,IN), length of third segment of member IN

Continue up to number of segments of member IN (maximum is 10)

11. Section Card (2A4,2X,3I10)

Column 1- 8--ITYPE, type of section. Type either SOLID, HOLLOW, OVAL, or CIRHOLLO (circular hollow section), starting in column 1 for type of section desired

Column 11-20--ICON, status of concrete (if ICON = 0, unconfined concrete; if ICON = 1, confined concrete). To use confined concrete option, user must ensure enough lateral ties are provided to meet the equivalent of ACI Building Code 318-83 Appendix A Sec. A.4.4 for compression members.

Column 12-30--ISTAT, status of reinforcing steel (if ISTAT = 0, steel fibers generated by the program; if ISTAT = 1, steel fibers read in by the user)

Column 31-40--IPRNT, print parameter (if IPRNT = 0, print out of fiber properties will be suppressed)

Description of Sections. Cards 12 through 14 should be repeated INSECT(IN) times (J = 1 to INSECT(IN))

12. Segments with Section J (3I10) (See Fig. C.4)

Column 1-10--I1, first segment with section J

Column 11-20--I2, last segment with section J

13. Geometry of Section J (8F10.0) (see Fig. C.5 and C.6)

(a) If ITYPE = SOLID (rectangular solid)

Column 1-10--ZI, width of section, dimension along the z-axis

Column 11-20--XI, depth of section, dimension along the x-axis

(b) If ITYPE = HOLLOW (rectangular hollow with or without interior walls)

Column 1-10--ZI, width of section (exterior dimension along the z-axis)

Column 11-20--TF, thickness of exterior wall which is parallel to z-axis

Column 21-30--XI, depth of section (exterior dimension along the x-axis)

Column 31-40--TW, thickness of exterior wall which is parallel to x-axis

13.b.1 Status of Interior Walls of Hollow Section (A4)

Column 1- 4--ICODE (Are there any interior walls? Type YES or NO beginning in column 1.)

If ICODE = YES

13.b.2 Description of Interior Walls (3A4, 8X, I10)

Column 1-12--IOR, orientation of walls (type VERTICAL if the walls are parallel to the x-axis; type HORIZONTAL if the walls are parallel to the z-axis; begin in column 1)

Column 21-30--NW, number of interior walls (maximum 5)

If ICODE = YES

13.b.3 Geometry of Interior Walls (8F10.0)

Column 1-80--Starting with CTROID(I), alternate CTROID(I), distance from centroid of section to centroid of interior wall, with TWALL(I), thickness of wall, in fields of F10.0 for I = 1 to NW

(One data card can contain pairs of CTROID(I) and TWALL(I) for four interior walls; therefore, in case of a fifth interior wall a new card should be added.)

(c) If ITYPE = OVAL (also used for solid circular*)

Column 1-10--ZI, width of section (dimension along z-axis)

Column 11-20--XI, depth of section (dimension along x-axis)

*The solid circular section is a special case of the oval section with XI = the diameter and ZI = XI + 0.01.

(d) If ITYPE = CIRHOLLO (circular hollow)

Column 1-10--ZI, exterior diameter of section

Column 11-20--TF, thickness of wall section (thickness must be less than 1/2 the diameter)

14. Reinforcing Steel Description (see Fig. C.6)

(a) If ISTAT = 0

14.a.1 Description of Steel Rings (I10,7F10.0)

Column 1-10--NRING, number of rings of steel (at least one ring, maximum of 3 rings). The rings are symmetric.

Column 11-70--Starting with STT(I), alternate STT(I) area of end steel in ring (parallel to the z-axis) with STS(I), area of side steel in ring (parallel to the x-axis) in fields of F10.0 for I=1 to NRING (for solid circular and hollow circular sections, STT=0 and STS is half the total area of steel)

14.a.2 Concrete Cover (8F10.0)

Column 1-60--Starting with GAMMAZ(I), alternate GAMMAZ(I), distance (parallel to z-axis) from side edge of section to center of side steel with GAMMAX(I), distance from end edge of section to center of end steel, in fields of F10.0 for I = 1 to NRING

14.a.3 Steel in Interior Walls (8F10.0)

Only if ITYPE = HOLLOW and ICODE = YES

Column 1-50--WLSTL(I), area of steel in each interior wall in fields of F10.0 for I = 1 to NW

(b) If ISTAT = 1

14.b.1 Steel Fibers (I10)

Column 1-10--NSTOT, number of reinforcing bars to be read (maximum 20)

14.b.2 Description of Reinforcing Bars (2(3F10.2))

Column 1-60--Starting with STLZ(I), alternate STLZ(I), z-distance from centroid of section to reinforcing bar, with STLX(I), x-distance from centroid of section to reinforcing bar, and STLA(I), area of reinforcing bar, in fields of F10.0 for I = 1 to NSTOT (2 bar descriptions per card)

15. Type of Analysis (I10)

Column 1-10--NLFDG, number of load factor design groups

If NLFDG > 0, a number equal to NLFDG of AASHTO load factor groups will be considered in the analysis (number of load factor groups is limited to 10)

If NLFDG $\leq$ 0, the analysis will be carried out without factoring the loads or using factored load groups

16. Loading System

(a) If NLFDG $\leq$ 0

16.a.1 Load Cases (I10)

Column 1-10--NLOAD, number of load cases for the problem (load cases represent the number of different load groups. All pier behavior is cumulative from one load case to the next.)

Cards 16.a.2 to 16.a.6 are repeated for the number of load cases, NLOAD

16.a.2 <u>Load Description (3I10)</u>

Column 1-10--NLINC, number of load increments. The number of load increments for the total number of load cases should not be less than 30 to obtain an accurate solution.

Column 11-20--NML, number of total members loaded

Column 21-30--KPRNT, print parameter (results from every KPRNTth increment will be printed)

If NML > 0, cards 16.a.3 and 16.a.4 are repeated NML times.

16.a.3 <u>Membership Loading (3I10)</u>

Column 1-10--I, number of the member loaded

Column 11-20--NMJL, total number of joints of member I loaded

Card 16.a.4 is repeated NMJL times.

16.a.4 <u>Loads (I10,7F10.0) (see Fig. B.7 for sign convention)</u>

One card for loads at each loaded joint.

If a joint was specified as a support and has a 1 specified in the release code for a degree of freedom, and if the user specifies a value at the support for that degree of freedom, he is specifying an incremental displacement or rotation, not a load. For factored load design all loads should be factored loads and should also be divided by proper strength reduction factors of AASHTO Sec. 1.5.30(b)(2).

Column 1-10--J, number of the member joint loaded

Column 11-20--FOR(1), incremental force in the x-direction

Column 21-30--FOR(2), incremental force in the y-direction

Column 31-40--FOR(3), incremental force in the z-direction

Column 41-50--FOR(4), incremental moment about the x-axis

Column 51-60--FOR(5), incremental moment about the y-axis

Column 61-70--FOR(6), incremental moment about the z-axis

16.a.5 <u>Frame Loading (I10)</u>

Column 1-10--NFJL, number of total frame joints loaded

16.a.6 <u>Loads (I10,7F10.0)</u>

Only if NFJL > 0. One card for loads at each joint. If a joint was specified as a support, and has a 1 specified in the release code for a degree of freedom, and if the user specifies a value at the support for that degree of freedom, he is specifying an incremental displacement or rotation, not a load. For factored load design, see note on 12.a.3.

Column 1-10--J, number of the frame joint loaded

Column 11-20--FOR(1), incremental force in the x-direction

Column 21-30--FOR(2), incremental force in the y-direction

Column 31-40--FOR(3), incremental force in the z-direction

Column 41-50--FOR(4), incremental moment about the x-axis

Column 51-60--FOR(5), incremental moment about the y-axis

Column 61-70--FOR(6), incremental moment about the z-axis

(b) If NLFDG > 0

16.b.1 <u>Load Description (8I10)</u>

Column 1-10--JPRNT, print parameter (results from every JPRNTth incremental analysis will be printed)

Column 11-20--NLTYP, number of different load types to be input (D, L, ICE, etc.) (maximum of 14)

Column 21-30--LGI(1), identification of the AASHTO load factor design group combination to be considered first (see Table C.2)

Column 31-40--LFI(2), identification of the AASHTO load factor design group combination to be considered second.

Continue in fields of I10 until NLFDG load factor design group combinations have been completed. If NLFDG > 6 begin a new card with LGI(7).

Cards 16.b.2 to 16.b.6 are repeated NLTYP times.

16.b.2 Type of Load (8I10)

Column 1-10--LTI, load type (see Table C.2) (for example, load type ICE is input as 14)

Column 11-20--NML, number of total members loaded with load type LTI (for example, all members subjected to ice loads)

If NML > 0, cards 16.b.3 and 16.b.4 are repeated NML times.

16.b.3 Member Loading (8I10)

Column 1-10--IM, number of the member loaded

Column 11-20--NMJL, number of total joints of member IM loaded

Card 16.b.4 is repeated NMJL times.

16.b.4 Loads (I10,7F10.0) (Fig. C.7)

One card for loads at each loaded joint. In this option (AASHTO Load Group) specified displacements or rotation are not directly permitted. To achieve specified displacements the user must input an equivalent static force system that produces the desired displacements.

(All loads should be divided by proper strength reduction factor ϕ of AASHTO Sec. 1.5.30(B)(2).)

Column 1-10--J, number of the member joint

Column 11-20--FOR(1), total unfactored force in the x-direction corresponding to load type LTI

Column 21-30--FOR(2), total unfactored force in the y-direction corresponding to load type LTI

Column 31-40--FOR(3), total unfactored force in the z-direction corresponding to load type LTI

Column 41-50--FOR(4), total unfactored moment about the x-axis, corresponding to load type LTI

Column 51-60--FOR(5), total unfactored moment about the y-axis, corresponding to load type LTI

Column 61-70--FOR(6), total unfactored moment about the z-axis, corresponding to load type LTI

16.b.5 Frame Loading (8I10)

Column 1-10--NFJL, number of total frame joints loaded with load type LTI

16.b.6 Loads (I10,7F10.0) (Fig. C.7)

Only if NFJL > 0. One card for loads at each joint. In this option (AASHTO Load Group) specified displacements or rotations are not directly permitted. To achieve specified displacements the user must input an equivalent static force system that produces the desired displacements. (Apply proper Ø factor; see 16.b.4).

Column 1-10--J, number of the frame joint loaded

Column 11-20--FOR(1), total unfactored force in the x-direction, corresponding to load type LTI

Column 21-30--FOR(2), total unfactored force in the y-direction, corresponding to load type LTI

Column 31-40--FOR(3), total unfactored force in the z-direction, corresponding to load type LTI

Column 41-50--FOR(4), total unfactored moment about the x-axis, corresponding to load type LTI

Column 51-60--FOR(5), total unfactored moment about the y-axis, corresponding to load type LTI

Column 61-70--FOR(6), total unfactored moment about the z-axis, corresponding to load type LTI

C.4 Computer Output

FPIER computer the load-deflection analysis of multiple pier bents. The output consists of:

C.4.1 Title and echo print of all section and member properties.

C.4.2 Echo print of loading conditions.

C.4.3 Printout of incremental results which includes forces and displacements at each joint, until all load increments have been completed, or there is an assumed stability or material failure. If there is an assumed material failure, the results just before failure and the location of the failure will be printed. The program will signal the appropriate level of appropriateness of the design only when the AASHTO load factor option of load application is used.

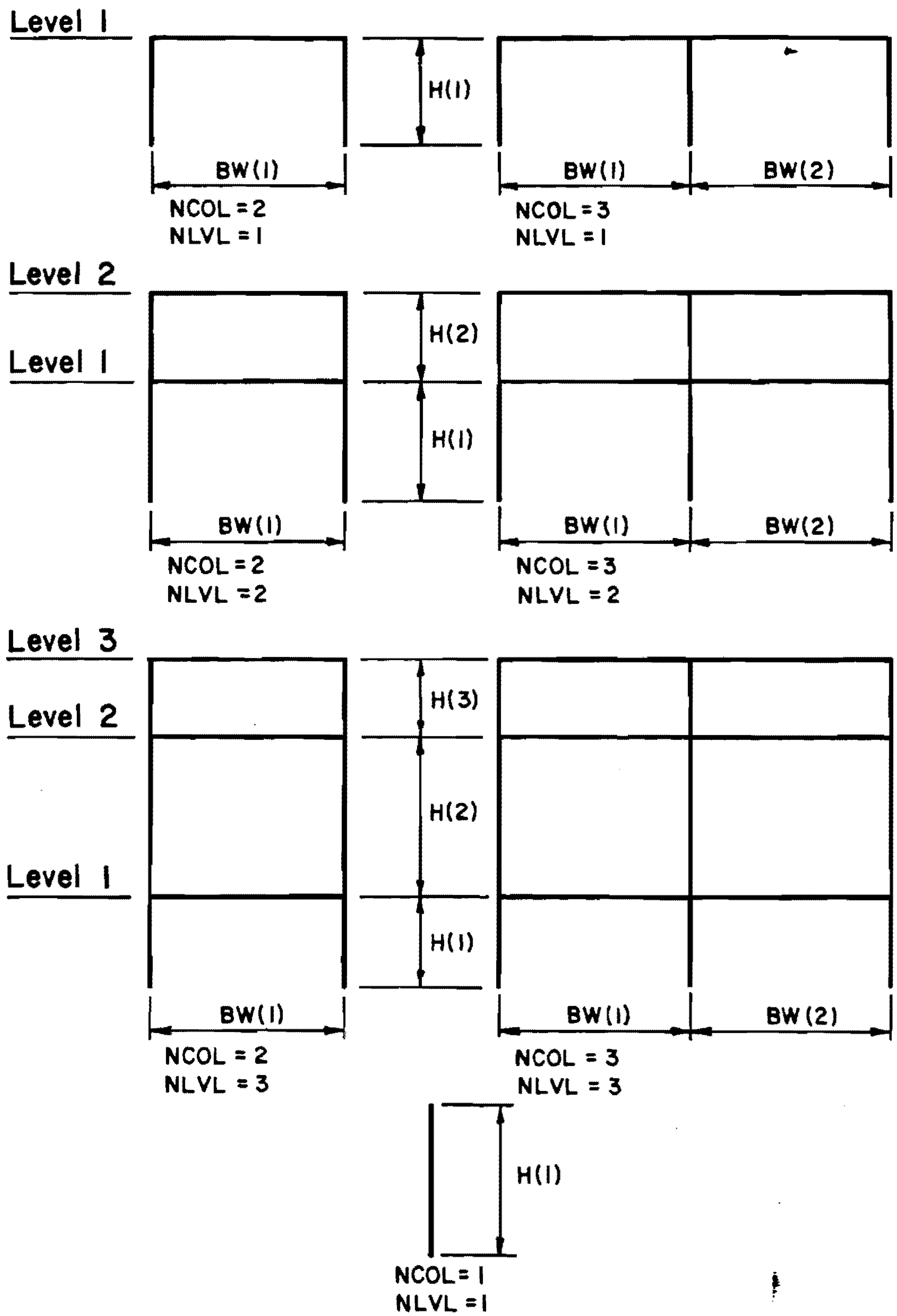


Fig. C.1 Pier bent geometry

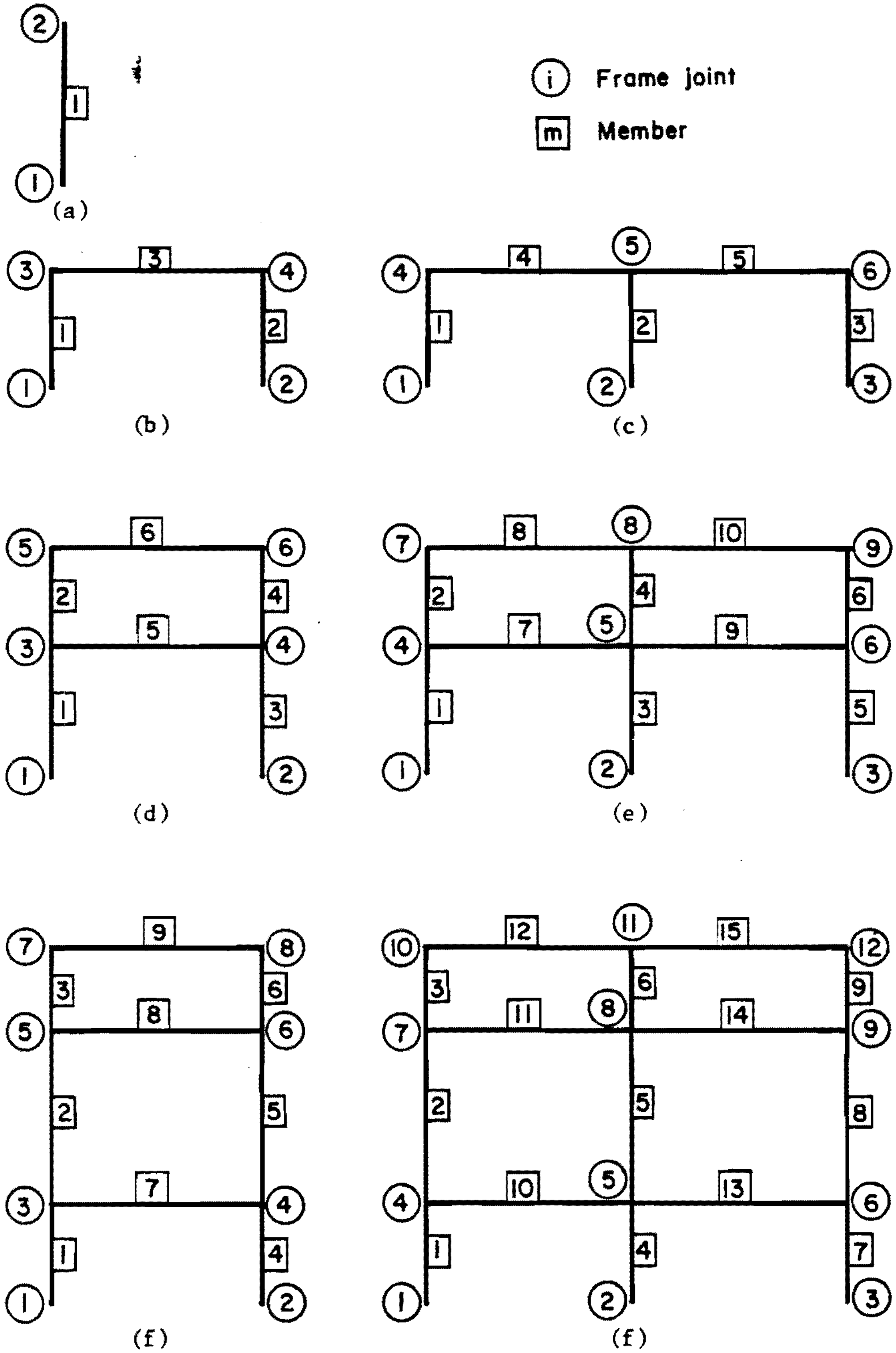


Fig. C.2 Numbering of frame joints and members for input and output

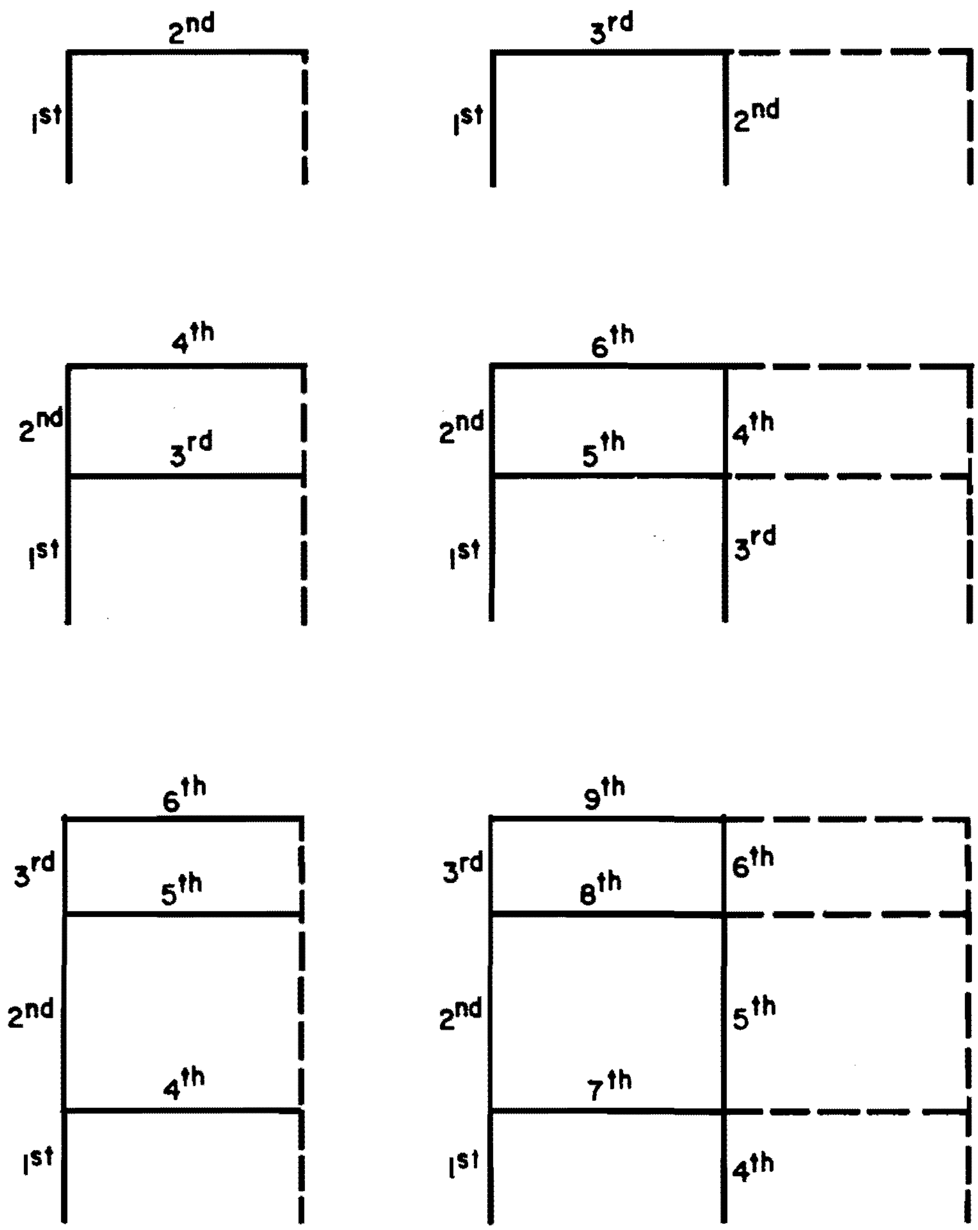


Fig. C.3 Sequence to input member characteristics

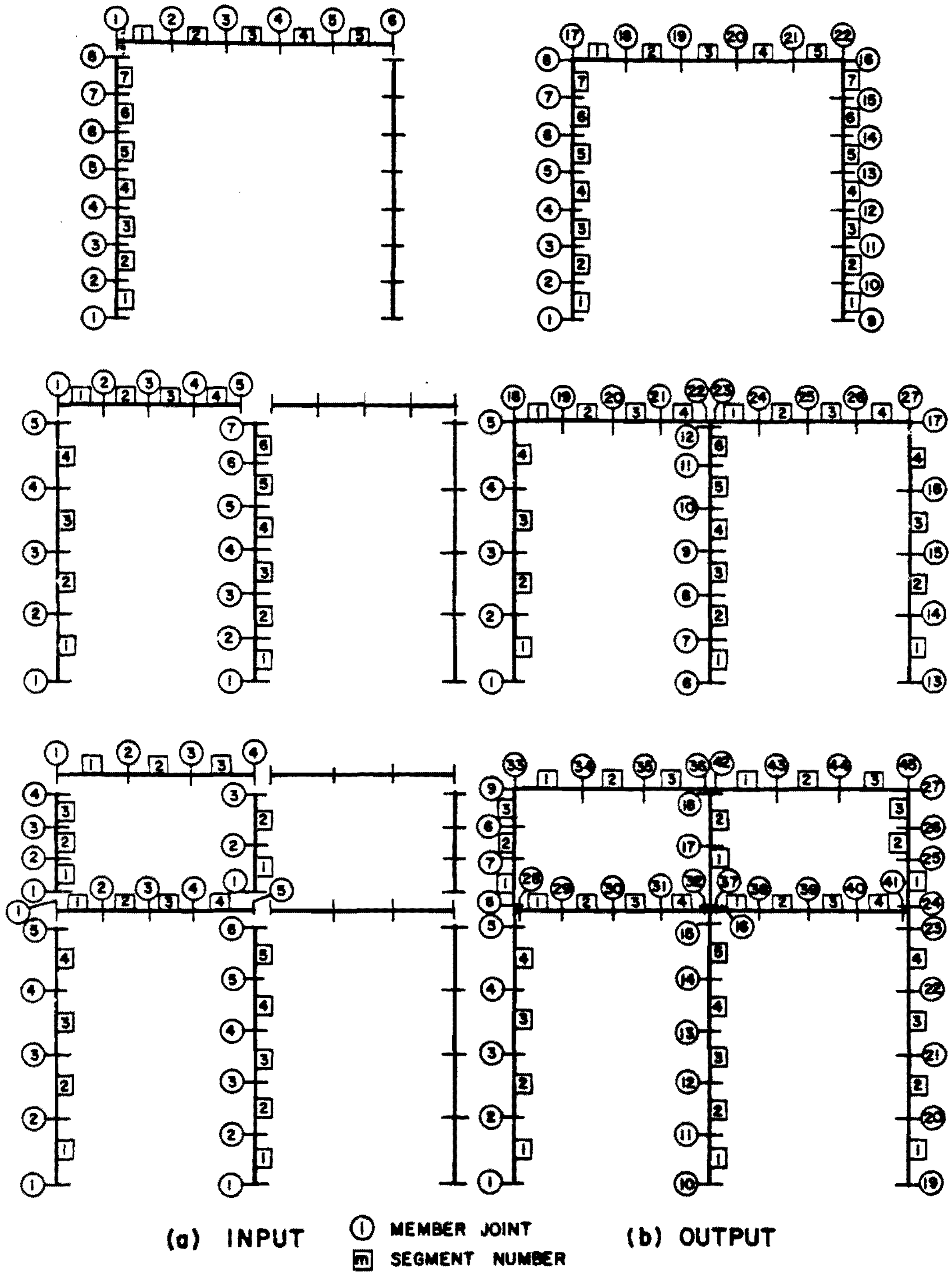


Fig. C.4 Numbering of segments and member joints

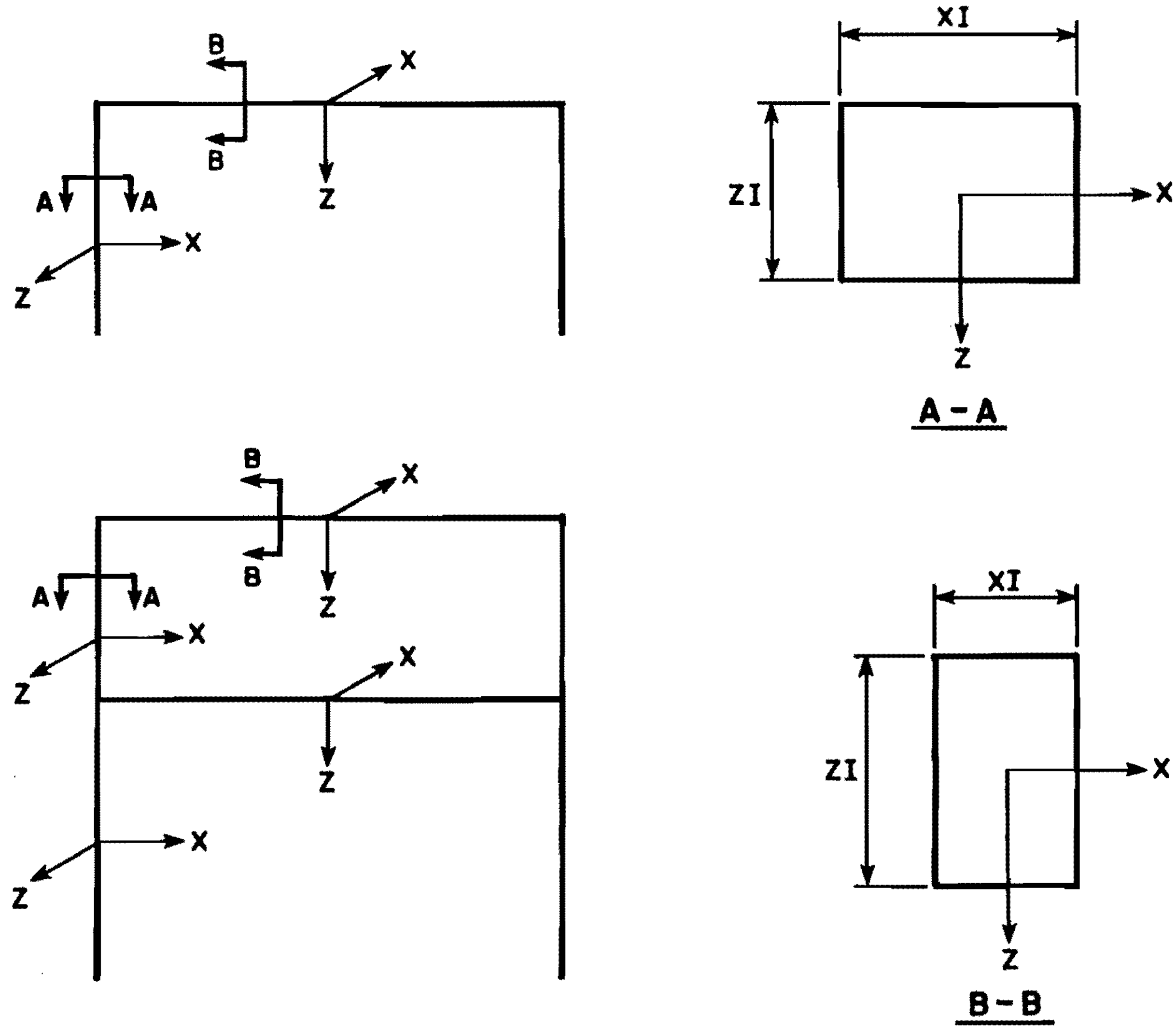
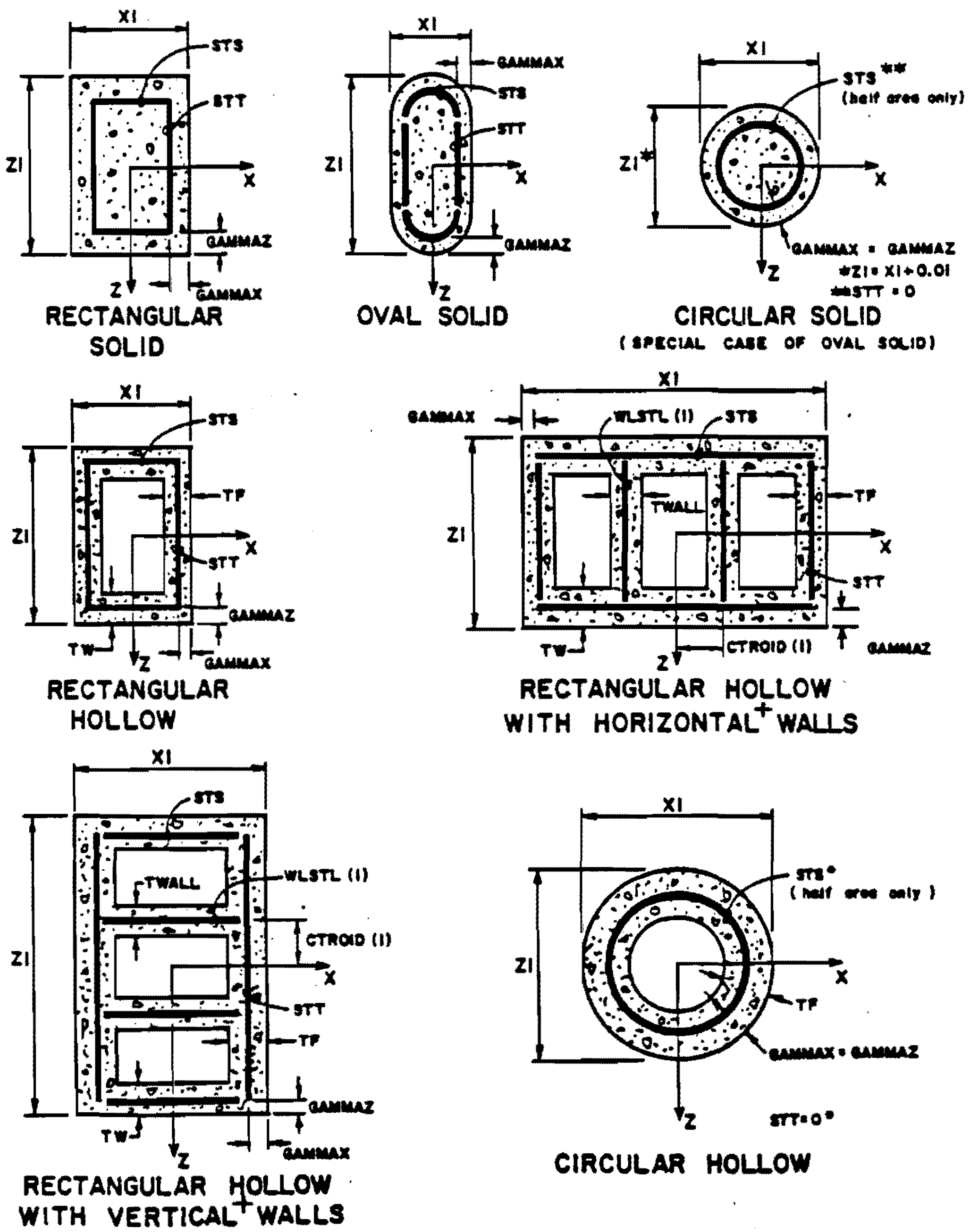


Fig. C.5 Location of axes for section properties



+ Interior walls are not oriented (vertical and horizontal) as seen in this figure because of the global coordinate system used in FPIER.

Fig. C.6 Section characteristics

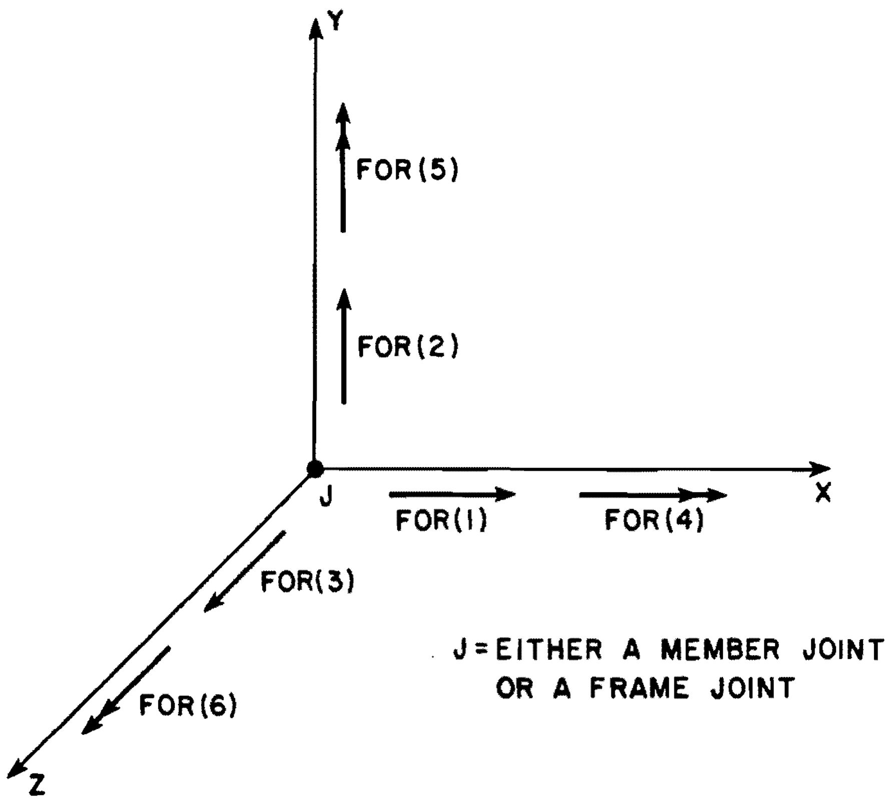


Fig. C.7 Positive convention of forces

TABLE C.1 Creep Factor, BETA

TIME (days)	BETA*
7	0.88
30	1.46
60	1.75
90	1.92
120	2.04
150	2.13
180	2.21
365 (1 year)	2.50
548	2.68
730 (2 y)	2.80
1095	3.91
1460 (4 y)	4.04
1825 (5 y)	4.11
2190 (6 y)	4.17
2555 (7 y)	4.21
2920 (8 y)	4.24
3285 (9 y)	4.27
3650 (10 y)	4.30
5475 (15 y)	4.39
7300 (20 y)	4.45
9125 (25 y)	4.49
10950 (30 y)	4.53

* Reference: "Analysis of Reinforced Concrete Columns Under Sustained Load," by V. Chovichien, M. J. Gutzwiller, and R. H. Lee, ACI Journal, October 1973, pp. 692-699.

Table C.2 Load Groups and Types of Load

ARRAY BFAC (Beta Coefficient)

Loading Group: Program	Loading Group: AASHTO	Load Type PRO: 1 AASHTO: D*	2 D*	3 L+I	4 CF	5 E*	6 E*	7 B	8 SF	9 W	10 WL	11 LF	12 R+S+T	13 EQ	14 ICE	γ Values
1	I	β_d=1.0	β_d=0.75	1.67	1.0	1.3	0.5	1.0	1.0	0	0	0	0	0	0	1.3
2	IA			2.20	0	0	0	0	0	0	0	0	0	0	0	1.3
3	II			0	0	1.3	0.5	1.0	1.0	1.0	0	0	0	0	0	1.3
4	III			1	1			1.0	1.0	0.3	1.0	1.0	0	0	0	1.3
5	IV			1	1			1.0	1.0	0	0	0	1.0	0	0	1.3
6	V			0	0			1.0	1.0	1.0	0	0	1.0	0	0	1.25
7	VI			1	1			1.0	1.0	0.3	1.0	1.0	1.0	0	0	1.25
8	VII			0	0			1.0	1.0	0	0	0	0	1.0	0	1.3
9	VIII			1	1			1.0	1.0	0	0	0	0	0	1.0	1.3
10	IX			0	0	1.3	0.5	1.0	1.0	1.0	0	0	0	0	1.0	1.20

D = Dead Load
L = Live Load
I = Live Load Impact
E = Earth Pressure
B = Buoyancy

W = Wind Load on Structure
WL = Wind Load on Live Load
(100 lbs. per lin. ft.)
(1458 N/m)
LF = Longitudinal Force from Live Load

CF = Centrifugal Force
R = Rib Shortening
S = Shrinkage
T = Temperature
EQ = Earthquake
SF = Stream Flow Pressure
ICE = Ice Pressure

* See AASHTO manual Section 1.2.22;
only 1 value of dead load and 1 value of earth pressure may be applied per problem.

C.5 Listing of FPier

```
C        DATA SET T224174    AT LEVEL 005 AS OF 01/20/84
      PROGRAM FPIER
C     *********************************************************
C     *                                                       *
C     *              >> PROGRAM FPIER <<                      *
C     *                                                       *
C     *  THE PRIMARY OBJECTIVE OF THE  PROGRAM  IS  THE       *
C     *  STRUCTURAL ANALYSIS OF A  REINFORCED  CONCRETE       *
C     *  FRAME UP TO TWO BAYS AND THREE STORIES (TAKING       *
C     *  INTO ACCOUNT SYMMETRY). IT COULD  BE USED  FOR       *
C     *  THE ANALYSIS OF A REINFORCED CONCRETE BEAM-COL       *
C     *  UMN(ZERO BAY,ONE STORY) TOO. THE  PROGRAM  HAS       *
C     *  BEEN DEVELOPED FOR STATIC LOADING ONLY.              *
C     *                                                       *
C     *********************************************************
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      CHARACTER*4,ASHTO(10),IHOLL,IHOLL1,IH1,ITITLE(20),ITYPE(2),
     .            ISOLID,IOVAL,ICHOLL,INC1,IV1,ISOLI1,IOVAL1,ICHOL1,
     .            INTTP(9,2),GREG(27),IOK(3),INO,IH,IV,ICODE
      DIMENSION AKPX(12),AKPY(12),AKPZ(12),AKX(12),AKY(12),AKZ(12),
     1AMAT(3,3),AX*(150),BMAT(3,3,150),BW(2),CPA(4000),COMPX(4000),
     2COMPZ(4000),DFORA(6,60),DFORB(6,60),DU(72),DUA(6),DUB(6),
     3EPS(150),EIC(200),EIS(200),PIY(150),PIZ(150),POR(6),FORA(6,60),
     4FORB(6,60),FORM(60,15),H(3),HH(10,9),ICCON(9),
     5INSECT(9),INSEG(9),ISEM(15),JR(12),NPC(10,9),
     6MPS(10,9),NIC(10,9),NIS(10,9),NSEM(15),POIS(72),PPOR(72),
     7ROT(6,6),SA(6,6,60),SB(6,6,60),SC(6,6,60),SKAA(6,6,60),
     8SKAB(6,6,60),SKBB(6,6,60),STAA(6,6),STAB(6,6),STBA(6,6),STBB(6,6),
     9STK(72,72),STLA(2000),STLB(2000),STLZ(2000),TKAA(6,6,15),
     *TKAB(6,6,15),TKBB(6,6,15),U(72),UT(6,60),UU(6,60)
      DIMENSION GAMMA(12),FRAC(10,16)
      COMMON /ONE/ ZI,TF,KI,TW
      COMMON /TWO/ STRGIC(5),YWALL(5),NW,NWE,NWT
      COMMON /THREE/ ICR,IAO,IH,IV
      COMMON /FOUR/ STT(3),SIS(3),GAMMAZ(3),GAMPAX(3),WLSTL(3),NRING
      COMMON /FIVE/ ISOLID,IHOLL,IOVAL,ICHOLL
      COMMON/LO/ SIG
      COMMON /L1/ ITL,AK,EC,ENO,EW
      COMMON /L2/ FY,ET,RATIO(10)
      COMMON /STF/ DA11,DA12,DA13,DA21,DA22,DA23,DA31,DA32,DA33
      COMMON /SUP/ IPRNT
      COMMON /SAME/ KCTYPE,KSTYPE,KPY,KAY,NA,NT,NV,NM,NZ
      COMMON/M1/ SIGO,MLOAD,MINC,MSIG,MSECT
      COMMON /D1/ ASHTO
      COMMON /D2/ GAMMA,FRAC,IPPP
      COMMON/M2/ SIGS,JLOAD,JINC,JSEG,JSECT
      COMMON/DSIGN/CUMMF(60,15),CUMM(72)
      DATA GREG/'IASH','TO E','QUAT','IONS',' ','AMMA','(SD=',
     1          'D=SL','=(L+','D)+E','C=CF','+SE=','E+SS','+S+E',
     2          'S+SP','+E+W','+SE+','L+AL','+ELA','LP+S','R=(R',
     3          '+SET',')+SE','L+SE','+EIC','S+IC','E) '/
C .... INO,IH,ISOLID,IHOLL,IOVAL,IONE,IZERO, ARE PARAMETERS(VALUES AS
C      DEFINED) USED IN THE PROGRAM TO MAKE DECISIONS.
      DATA INC1/'NO  '/,IH1/'HORI'/,IV1/'VERT'/,ISOLI1/'SOLI'/,
     .     IHOLL1/'HOLL'/,IOVAL1/'OVAL'/,ICHOL1/'CIRH'/
      DATA IONE/1/,IZERO/0/
      JT=1
      CALL TICTOC (JT)
      INO=INC1
      IH=IH1
      IV=IV1
      ISOLID=ISOLI1
      IHOLL=IHOLL1
      IOVAL=IOVAL1
      ICHOLL=ICHOL1
      KCTYPE=0
      KSTYPE=0
C .... READ PROBLEM TITLE AND ECHO PRINT IT ....
      READ (*,1000,END=10000)(ITITLE(I),I=1,20)
 1000 FORMAT(8X,20A4)
10000 PRINT 2000,(ITITLE(I),I=1,20)
 2000 FORMAT('1',4X,20A4)
C .... READ AND ECHO PRINT NUMBER OF COLUMN LINES(NCOL) AND NUMBER OF
C      LEVELS(NLVL).
      READ (*,1010,END=10001)NCOL,NLVL
 1005 FORMAT(8X,I10)
10001 PRINT 1007,NCOL,NLVL
 1007 FORMAT('0','NUMBER OF COLUMN LINES=',I2,/,
     1' NUMBER OF LEVELS       =',I2,/)
 1010 FORMAT(8X,2I10)
C .... CALCULATE NUMBER OF BAYS (NBY) ....
      NBY=NCOL-1
C .... READ COLUMN HEIGTHS PER LEVEL (H(I)) ....
      READ (*,1020,END=10002)(H(I),I=1,NLVL)
 1020 FORMAT(8X,8F10.0)
C .... IF NBY DIFFERENT THAN ZERO READ BEAM LENGTHS PER BAY(BW(I)).
C      IF NBY EQUAL ZERO MAKE THE PARAMETER NR EQUAL NBY.
10002 IF(NBY.EQ.0) GO TO 10
      READ (*,1020,END=   10)(BW(I),I=1,NBY)
   10 NR=NBY
C .... EVALUATE NUMBER OF TOTAL JOINTS,NJT ....
      NJT=NCOL*(NLVL+1)
      NMEM=NLVL*(2*NCOL-1)
C .... READ AND ECHO PRINT THE NUMBER OF JOINTS WITH CONSTRAINT OR
C      SPECIFIED DISPLACEMENTS (NCJ),AND THE NUMBER OF JOINTS WITH
C      SPRINGS (NJSP).
      READ (*,1010,END=10003)NCJ,NJSP
10003 PRINT 2001,NCJ,NJSP
 2001 FORMAT(///,3X,'NUMBER OF JOINTS WITH CONSTRAINED',/,
     1             3X,' OR SPECIFIED DISPLACEMENTS    =',I3,/,
     2             3X,'NUMBER OF JOINTS WITH SPRINGS =',I3)
C .... INITIALIZE FOR EACH JOINT THE VECTORS OF ROTATIONAL SPRING
C      STIFFNESSES, THE VECTORS  OF TRANSLATIONAL  SPRING  STIFF
C      NESSES, AND THE VECTOR OF JOINT  RELEASE  CODES OF  NODAL
C      POINT I.
      DO 13 I=1,NJT
      AKX(I)=0.
      AKY(I)=0.
      AKZ(I)=0.
      AKPX(I)=0.
      AKPY(I)=0.
      AKPZ(I)=0.
   13 JR(I)=0
C .... IF NCJ DIFFERENT THAN ZERO READ AND ECHO PRINT THE JOINT
C      NUMBER,J, AND ITS RESPECTIVE VECTOR OF JOINT RELEASE  CO
C      DES,JR(J), OTHERWISE GO TO CHECK IF THERE ARE JOINTS WITH
C      SPRINGS.
      IF(NCJ.EQ.0) GO TO 15
      PRINT 2002
 2002 FORMAT(///,3X,' JOINT NO.',4X,'RELEASE CODE')
      DO 14 I=1,NCJ
      READ (*,1010,END=   14)J,JR(J)
   14 PRINT 2003,J,JR(J)
 2003 FORMAT(4X,I3,7X,I10)
C .... IF NJSP DIFFERENT THAN ZERO READ AND ECHO PRINT THE JOINT
C      NUMBER AND ITS RESPECTIVE VECTORS OF SPRING  STIFFNESSES,
```

```
C       OTHERWISE GO TO READ MATERIAL PROPERTIES.
C
   15 IF(NJSP.EQ.0) GO TO 17
      PRINT 2004
 2004 FORMAT (///,16X,'>>>>>TRANSLATIONAL SPRINGS <<<<<',5X,
     1'>>>>>ROTATIONAL SPRINGS <<<<<',//,3X,'JOINT NO.',6X,'X-DIR',
     210X,'Y-DIR',10X,'Z-DIR',7X,'AROUND X-AXIS',2X,'AROUND Y-AXIS',
     32X,'AROUND Z-AXIS')
      DO 16 I=1,NJSP
      READ (*,1025,END=   16)J,AKX(J),AKY(J),AKZ(J),AKFX(J),AKFY(J),AKFZ
     *(J)
   16 PRINT 2005,J,AKX(J),AKY(J),AKZ(J),AKFX(J),AKFY(J),AKFZ(J)
 2005 FORMAT(4X,I5,8X,3(1PE12.5,3X),2X,3(1PE12.5,3X))
 1025 FORMAT(5I,I10,7F10.0)
C .... READ AND ECHO PRINT MATERIAL PROPERTIES .....
   17 READ (*,1020,END=10004)CYL,FY,RK,EC,ET,EU,BETA
C....... EVALUATE ENO THE STRAIN CORRESPONDING TO RK*CYL(REDUCE  ....
C....... CONCRETE STRENGTH)                                        ...
10004 ENO=(1.0+BETA)*RK*2.0*CYL/EC
      IF(EU.EQ.0.) EU=1.5*ENO
      IF(EU.LE.ENO) PRINT 6100
      IF(EU.LE.ENO) STOP 5200
 6100 FORMAT('0',3X,62HULTIMATE STRAIN IS NOT GREATER THAN STRAIN
     .          ,13H AT MAXIMUM STRESS)
      PRINT 2006,CYL,FY,RK,EC,ET,EU,BETA
 2006 FORMAT(///,3X,'CONCRETE STRENGTH           =',F10.2,/,
     1           3X,'STEEL YIELD POINT           =',F10.2,/,
     2           3X,'STRENGTH REDUCTION FACTOR =',F10.2,/,
     3           3X,'CONCRETE MODULUS            =',1PE12.4,/,
     4           3X,'STEEL MODULUS               =',1PE12.4,/,
     5           3X,'CONCRETE ULTIMATE STRAIN  =',1PE12.4,/,
     6           3X,'CREEP FACTOR                =',F10.2)
C
C
C .... FROM HERE THROUGH JUST BEFORE SENTENCE #130 THE CHARACTERISTICS
C       OF  EACH MEMBER ARE EVALUATED ( NUMBER  OF SEGMENTS, NUMBER  OF
C       DIFFERENT SECTIONS,TYPE OF SECTION, NUMBER OF TOTAL CONCRETE FI
C       BERS, NUMBER OF TOTAL STEEL FIBERS, MOMENTS OF INERTIA.
C
C
C             IN = MEMBER NUMBER
C           MARY = THIS PARAMETER ALLOWS TO DIFFERENTIATE BETWEEN
C                  COLUMNS(MARY=1), AND BEAMS(MARY=2).
C           NCFS = STORES TOTAL NUMBER OF CONCRETE FIBERS.
C           NSFS = STORES TOTAL NUMBER OF STEEL FIBERS.
C              I = CONTROLS COLUMN LINE NUMBER(LOOP OVER BAYS)
C              J = CONTROLS LEVEL NUMBER(LOOP OVER FLOORS).
C             N2 = CONTROLS THE NUMBER OF COLUMNS TO BE PROCESSED
C                  TAKING INTO ACCOUNT SYMMETRY.
C
C
      IF(N2.EQ.0) N=1
      IN=0
      MARY=1
      NCFS=1
      NSFS=1
      I=1
   20 J=1
   30 IN=IN+1
C
C .... READ NUMBER OF SEGMENTS,INSEG(IN), AND NUMBER OF DIFFERENT
C       SECTIONS,INSECT(IN),OF MEMBER IN. STORE THE VALUES IN NSEG
C       AND NSECT RESPECTIVELY. ECHO PRINT.
C
      READ (*,1010,END=10005)INSEG(IN),INSECT(IN)
      IF(INSECT(IN).EQ.0) INSECT(IN)=1
10005 NSEG=INSEG(IN)
      NSECT=INSECT(IN)
      PRINT 2010,I,J,NSEG,NSECT
 2010 FORMAT(///,3X,'MEMBER',2X,I2,'  LEVEL',2X,I2,
     1       ' HAS ',2X,I2,' SEGMENTS AND ',2X,I2,
     2       ' DIFFERENT SECTIONS ',/)
C
C .... IF NSEG = 1 OR 0 THE LENGTH OF THE MEMBER EQUAL THE MEMBER
C       LENGTH. OTHERWISE READ AND ECHO PRINT SEGMENT LENGTHS,HH.
C
      IF(NSEG.LE.1) GO TO 40
      READ (*,1020,END=10006)(HH(L,IN),L=1,NSEG)
 1026 FORMAT(8X,15F5.0)
10006 PRINT 2020,(HH(L,IN),L=1,NSEG)
 2020 FORMAT(/,3X,'SEGMENT LENGTHS',10F10.2,//)
C
C .... READ SECTION TYPE,THE CONCRETE STATE CODE,STEEL FIBERS CODE
C       GENERATION, AND THE PRINTER OPTION.
C
   40 IF(NSEG.LE.1) HH(1,IN)=H(IN)
      READ (*,1030,END=10007)ITYPE,ICON,ISTAT,IPRNT
 1030 FORMAT(5X,2A4,2X,3I10)
10007 ICCON(IN)=ICON
      DO 45 IT=1,2
   45 INTYP(IN,IT)=ITYPE(IT)
      PRINT 2030,ITYPE
 2030 FORMAT(///,3X,'THE MEMBER CROSS-SECTION IS',2X,2A4,/)
C
C .... IF ICON = 0 CONCRETE IS CONSIDERED UNCONFINED, IF ICON = 1
C       CONCRETE IS CONSIDERED CONFINED.
C
      IF(ICON.EQ.0) PRINT 2040
      IF(ICON.EQ.1) PRINT 2050
 2040 FORMAT('0',2X,'CONCRETE FIBERS WILL BE MODELLED AS',
     1' UNCONFINED CONCRETE')
 2050 FORMAT('0',2X,'CONCRETE FIBERS WILL BE MODELLED AS',
     1' CONFINED CONCRETE')
C
C .... IF ISTAT = 0 THE PROGRAM WILL GENERATED STEEL FIBERS,
C       IF ISTAT = 1 THE USER WILL READ IN INDIVIDUAL REINFORCEMENT.
C
      IF (ISTAT.EQ.IZERO) PRINT 2060
      IF(ISTAT.EQ.IONE) PRINT 2070
 2060 FORMAT('0',2X,'STEEL FIBERS ARE GENERATED BY THE PROGRAM')
 2070 FORMAT('0',2X,'STEEL FIBERS ARE READ IN BY THE USER')
C
C .... IF IPRNT = 0 PRINT OUT OF FIBER PROPERTIES WILL BE SUPRESSED ....
C
      IF(IPRNT.EQ.IZERO) PRINT 2080
 2080 FORMAT('0',2X,'PRINT OUT OF FIBER PROPERTIES',
     1' WILL BE SUPPRESSED')
C
C..... LOOP IN SECTIONS TO FIND THEIR PROPERTIES(TYPE,STEEL
C..... FIBERS,CONCRETE FIBERS,MOMENTS OF INERTIA,AREA)
C
      DO 120 JAY=1,NSECT
C .... READ FIRST(I1) AND LAST(I2) SEGMENTS WITH EQUAL SECTION ....
      READ (*,1010,END=10008)I1,I2
10008 PRINT 2090,JAY
 2090 FORMAT('0',2X,'CROSS-SECTION TYPE ',I3)
```

```
C .... READ SECTION DIMENSIONS AS INDICATED BY ITYPE (SOLID,OVAL,
C       HOLLOW) ....
      IF(ITYPE(1).EQ.ISOLID) GO TO 50
      IF(ITYPE(1).EQ.IOVAL) GO TO 50
      IF(ITYPE(1).EQ.IHOLL) GO TO 60
      IF(ITYPE(1).EQ.ICHOLL) GO TO 65
      PRINT 2100
 2100 FORMAT('0',2X,'MISTAKE MADE IN INPUT OF SECTION TYPE')
   50 READ (*,1020,END=10009)ZI,XI
10009 PRINT 2110,ZI,XI
 2110 FORMAT(3X,'0',2X,'Z-DIMENSION =',F10.2,/,
     1                3X,'X-DIMENSION =',F10.2)
      GO TO 75
   60 READ  (*,1020,END=10010)ZI,TF,XI,TW
10010 PRINT 2120,ZI,TF,XI,TW
 2120 FORMAT('0',2X,'Z-DIMENSION=',F10.2,/,
     1              3X,'FLANGE THICKNESS=',F10.2,/,
     2              3X,'X-DIMENSION=',F10.2,/,
     3              3X,'WEB THICKNESS    =',F10.2)
      READ (*,1030,END=10011)ICODE
C .... IF ICODE IS NOT EQUAL TO NO READ ORIENTATION AND NUMBER
C       OF INTERIOR WALLS.
10011 IF(ICODE.EQ.INO) GO TO 75
      READ (*,1035,END=10012)IOR,NW
 1035 FORMAT(2X,3A6,4X,I10)
10012 READ (*,1020,END=10013)(CTROID(L),TWALL(L),L=1,NW)
10013 PRINT 2130,NW,IOR
 2130 FORMAT(///,3X,'NUMBER OF INNER WALLS=',I5,/,
     1              3X,'WALLS ARE ORIENTED',2X,3A6)
      PRINT 2140
 2140 FORMAT(//,3X,'NO.',5X,'Z OR X CENTROID',6X,'THICKNESS')
      DO 70 L=1,NW
   70 PRINT 2150,L,CTROID(L),TWALL(L)
 2150 FORMAT(1X,I5,5X,F10.2,6X,F10.2)
      GO TO 75
   65 READ (*,2110,END=10014)ZI,TF
10014 PRINT 2125,ZI,TF
 2125 FORMAT('0',2X,'DIAMETER OF SECTION=',F10.2,/,
     1              3X,'WALL THICKNESS      =',F10.2)
   75 CONTINUE
C .... EVALUATE THE NUMBER OF CONCRETE FIBERS AND THEIR PROPERTIES
C       ACCORDINGLY TO ITYPE(1).
      IF( ITYPE(1).EQ.ISOLID) CALL SOLSCR(CONFI(NCFS),CONFX(NCFS)
     1,CFA(NCFS),NFIB)
      IF(ITYPE(1).EQ.IHOLL) CALL CSCRIA(CONFI(NCFS),CONFX(NCFS),
     1CFA(NCFS),ICODE,NFIB)
      IF(ITYPE(1).EQ.IOVAL) CALL OSCRIF(CONFI(NCFS),CONFX(NCFS),
     1CFA(NCFS),NFIB)
      IF(ITYPE(1).EQ.ICHOLL) CALL CIRSCR(CONFI(NCFS),CONFX(NCFS),
     1 CFA(NCFS),NFIB)
C .... STORE THE INITIAL,NIC(L,IN),AND THE FINAL,NFC(L,IN)
C .... CONCRETE FIBER OF SEGMENT L MEMBER IN.
      DO 40 L=1,II
      NIC(L,IN)=NCFS
   40 NFC(L,IN)=NFIB
C .... EVALUATE THE TOTAL NUMBER OF CONCRETE FIBERS,NCFS.
C .... IF NCFS GREATER THAN 6000 PRINT ERROR MESSAGE.
      NCFS=NCFS+NFIB
      IF(NCFS.GT.6000) GO TO 962
C .... FROM HERE THROUGH SENTENCE 85 THE PROGRAM EVALUATES THE
C .... MOMENT OF INERTIA OF THE CONCRETE SECTION AROUND  THE
C .... Z-AXIS AND THE X-AXIS.
C               KAT = CONTINUOUS LOOP OVER NUMBER OF FIBERS.
C               KF = FIBER NUMBER.
C               XC = STORES X-CENTROID OF CONCRETE FIBER KK.
C               ZC = STORES Z-CENTROID OF CONCRETE FIBER KK.
C               AR = STORES AREA OF CONCRETE FIBER KK.
C
      ZZZ=0.
      XIX=0.
      DO 85 KAT=1,NFIB
      KK=NIC(I1,IN)+KAT-1
      XC=CONFX(KK)
      ZC=CONFI(KK)
      AR=CFA(KK)
      ZZZ=ZZZ+XC*XC*AR
   85 XIX=XIX+ZC*ZC*AR
      PRINT 2160,ZZZ,XIX
 2160 FORMAT('0',2X,'MOMENT OF INERTIA ABOUT Z-AXIS=',1PE10.3,/,
     1              3X,'MOMENT OF INERTIA ABOUT X-AXIS=',1PE10.3,/)
C .... IF ISTAT = 0 READ THE NUMBER OF STEEL RINGS,NRING. ALSO READ
C .... THE STEEL END AREA,STT(L), THE SIDE STEEL AREA,STS(L),   THE
C .... Z-COVER,GAMMAZ(L), THE X-COVER,GAMMAX(L) FOR EACH RING. IF I
C .... TYPE EQUAL HOLLOW AND ICODE EQUAL YES, READ IN ADDITION  THE
C .... STEEL AREA OF INTERIOR WALLS.
C
C .... IF ISTAT = 1 CALL SUBROUTINE RECS AND READ IN STEEL FIBERS.
      IF(ISTAT.EQ.IZERO) GO TO 90
      IF(ISTAT.EQ.IONE) GO TO 102
   90 READ (*,1025,END=10015)NRING,(STT(L),STS(L),L=1,NRING)
10015 PRINT 2170,NRING
 2170 FORMAT(///,3X,'NUMBER OF STEEL RINGS=',I5,/,
     1              3X,'NO.',5X,'AREA OF END STEEL',5X,
     2              'AREA OF SIDE STEEL')
      PRINT 2180,(L,STT(L),STS(L),L=1,NRING)
 2180 FORMAT(1X,I3,7X,F10.2,16X,F10.2)
      READ (*,1020,END=10016)(GAMMAZ(L),GAMMAX(L),L=1,NRING)
10016 PRINT 2190
 2190 FORMAT(//,3X,'NO.',6X,'Z COVER',5X,'X COVER')
      PRINT 2200,(K,GAMMAZ(K),GAMMAX(K),K=1,NRING)
 2200 FORMAT(1X,I3,5X,F10.2,2X,F10.2)
      IF(ITYPE(1).EQ.IHOLL.AND.ICODE.NE.INO) GO TO 95
      GO TO 100
   95 READ (*,1020,END=10017)(WLSTL(L),L=1,NW)
10017 PRINT 2210
 2210 FORMAT(1X,I3,12X,F10.2)
  100 CONTINUE
C .... EVALUATE THE NUMBER OF STEEL FIBERS AND THEIR PROPERTIES
C .... ACCORDINGLY TO ITYPE(SOLID,HOLLOW,OVAL).
      IF(ITYPE(1).EQ.ISOLID) CALL SSCRIF(STLZ(NSFS),STLX(NSFS),
     1STLA(NSFS),ICODE,NSTOT,ITYPE(1))
      IF(ITYPE(1).EQ.IHOLL) CALL SSCRIF(STLZ(NSFS),STLX(NSFS),
     1STLA(NSFS),ICODE,NSTOT,ITYPE(1))
      IF(ITYPE(1).EQ.IOVAL) CALL OSSTL(STLZ(NSFS),STLX(NSFS),
     1STLA(NSFS),NSTOT)
      IF(ITYPE(1).EQ.ICHOLL) CALL CIRSTL(STLZ(NSFS),STLX(NSFS),
     1    STLA(NSFS),NSTOT)
      GO TO 105
C .... SUBROUTINE RED. IS USED TO READ IN STEEL FIBERS BY THE USER ....
  102 CALL RECS(STLZ(NSFS),STLX(NSFS),STLA(NSFS),NSTOT)
C .... LOOP IN SEGMENTS OF EQUAL SECTION TO STORE THE INITIAL,NIS(L,IN)
C .... AND THE FINAL,NFS(L,IN) STEEL FIBER OF SEGMENT L MEMBER IN.
  105 DO110 L=I1,I2
      NIS(L,IN)=NSFS
  110 NFS(L,IN) =NSTOT
C .... EVALUATE TOTAL NUMBER OF STEEL FIBERS,NSFS. IF NSFS GREATER
```

```
C .... THAN 2000 PRINT ERROR MESSAGE.
      NSFS=NSFS+NSTOT
      IF(NSFS.GT.2000) GO TO 961
  120 CONTINUE
      KCTYPE=0
      KSTYPE=0
C.........TORSIONAL PARAMETERS TO BE USED IN EVALUATING........
C           THE TORSIONAL STIFFNESS

      IF(ITTYPE(1).NE.IOVAL)  GO TO 122
      XI2=XI*XI
      ZI2=ZI*ZI
      CNOT=XI2*ZI2/(XI2+ZI2)
      CJ2=-2.0*CNOT/ZI2
      CJ1=-2.0*CNOT/XI2
      CJT=CJ1-CJ2
      GO TO 124
 122  IF(XI.GE.ZI) RRA=ZI/XI
      IF(XI.LT.ZI) RRA=XI/ZI
      CJT=4.0*RRA-3.35*RRA*RRA+0.277333*RRA*RRA*RRA-4.0/(3.0*RRA)
C..... END OF LOOP OVER SECTIONS .....
C .... INCREASE J TO EVALUATE THE NEXT COLUMN. IF J LESS OR
C .... EQUAL THAN NLVL REPEAT THE PROCEDURE FROM SENTENCE 30
C .... TO SENTENCE 120. OTHERWISE CHECK MARY.
  124 J=J+1
      IF(J.LE.NLVL)GO TO 30
C .... IF MARY =1 INCREASE THE NUMBER OF COLUMN LINE,I. OTHERWISE
C .... GO OUT OF THE LOOP (SENTENCE 130)
      GO TO(125,130),MARY
      CALL WOTGER
  125 I=I+1
C .... IF I LESS OR EQUAL THAN NR REPEAT THE PROCEDURE FROM SENTENCE
C .... 20 TROUGH SENTENCE 125 TO PROCESS THE REMAINING COLUMNS. OTHER
C .... CHECK NSY.
      IF(I.LE.NR) GO TO 20
C .... IF NSY DIFFERENT THAN ZERO MAKE MARY=2, AND BEGIN TO PROCESS
C .... THE GEOMETRIC PROPERTIES OF THE BEAMS. OTHERWISE  GO TO  130
C .... (OUT OF THE LOOP)
      IF(NSY.EQ.0) GO TO 130
      MARY=2
      GO TO 20
C .... DETERMINE THE MAXIMUM NUMBER OF SECTIONS PER SEGMENT,MNSSG,FOR
C .... WHICH THE NUMBER OF MEMBERS,NMEM, THE  NUMBER  OF COLUMNS, NCM
C .... AND THE NUMBER OF TOTAL SEGMENTS, NTSG ARE FIRST EVALUATED.
  130 NMEM=(NCOL+NSY)*NLVL
      INN=IN
      NCM=NLVL*NSY
      NTSG=0
      I=1
  140 NTSG=NTSG+INSEG(I)
      IF(I.LE.NLVL.AND.NSY.GT.0) NTSG=NTSG+INSEG(I)
      IF(I.GT.NCM.AND.NSY.EQ.0)  NTSG=NTSG+INSEG(I)
      I=I+1
      IF(I.LE.INN) GO TO 140
      MNSSG=130/NTSG
      LGP=0
      READ(MM,1005,END=1001)NLPOG
 1001 IF(NLPOG.GT.0) ILP=1
      IF(NLPOG.LE.0) ILP=0
C..... LOOP OVER MEMBERS TO FIND STIFFNESS MATRICES PER SEGMENT .......
  142 NCLA=NTSG-NMEM
      ISEM(1)=1
      MARY=1
      II=0
      NR=NSY
      NR1=NR+1
      IF(NR.EQ.0) NR=1
      IN=0
  145 I=1
  150 J=1
  155 IN=IN+1
      SIG=1.0
      II=I
      IF(II.GT.NR) II=1
      IF(MARY.EQ.1) IM=(II-1)*NLVL+J
      IF(MARY.EQ.2) IM=NCM+NLVL+J
      IF(MARY.EQ.1.AND.IM.NE.IN) SIG=-1.0
      NSEG=INSEG(IM)
      NSEM(IN)=NSEG*MNSSG
      IF(IN.LT.NMEM) ISEM(IN+1)=ISEM(IN)+NSEM(IN)
C..... LOOP OVER SEGMENTS TO RECALL NFIS,NSTOT,NSFS,NCFS
C                 AND DEFINE THE PARAMETER RATIO EQUAL
C                 STEEL AREA OVER CONCRETE AREA
      DO 170 LL=1,NSEG
      L=LL
      IF(MARY.EQ.2.AND.I.EQ.2) L=NSEG-LL+1
      AST=0.
      AG=0.
      NFIS=NFC(L,IM)
      NSTOT=NFS(L,IM)
      NSFS=NIS(L,IM)-1
      NCFS=NIC(L,IM)-1
      DO 160 K=1,NFIS
  160 AG=AG+CFA(NCFS+K)
      DO 165 K=1,NSTOT
  165 AST=AST+STLA(NSFS+K)
      RATIO(L)=AST/AG
  170 CONTINUE
C...... END OF LOOP OVER SEGMENTS .....
      NJ=NSEG+1
C ...... INITIALIZE THE CONCRETE STRAIN,EZC, AND THE STEEL
C               STRAIN,EZS,IN EACH FIBER
      DO 190 L=1,NFIS
  190 EZC(L)=0.
      DO 200 L=1,NSTOT
  200 EZS(L)=0.
      ICON=ICCON(IM)
      JJ=ISEM(IN)-1
C ..... LOOP OVER SEGMENTS .....
      DO 220 LL=1,NSEG
      L=LL
      IF(MARY.EQ.2.AND.I.EQ.2) L=NSEG-LL+1
      NSFS=NIS(L,IM)
      NCFS=NIC(L,IM)
      CALL CNSTF(CONFZ(NCFS),CONFX(NCFS),CFA(NCFS),STLZ(NSFS),
     1    STLX(NSFS),STLA(NSFS),EZC,EZS,NFIS,NSTOT,L,ICON)
      NSCT=NSEM(IM)
C ..... ASSEMBLE STIFFNESS MATRIX OF EACH SEGMENT ....
      DO 210 K=1,MNSSG
      JJ=JJ+1
      BMAT(1,1,JJ)=GA11
      BMAT(1,2,JJ)=GA12
      BMAT(1,3,JJ)=-GA13
      BMAT(2,1,JJ)=GA21
      BMAT(2,2,JJ)=GA22
      BMAT(2,3,JJ)=-GA23
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      SMAT(3,1,JJ)=-CA31
      SMAT(3,2,JJ)=-DA32
      SMAT(3,3,JJ)=DA33
      IF(MART.EQ.1) GO TO 210
      SMAT(1,2,JJ)=-SMAT(1,2,JJ)
      SMAT(2,1,JJ)=-SMAT(2,1,JJ)
      SMAT(1,3,JJ)=-SMAT(1,3,JJ)
      SMAT(3,1,JJ)=-SMAT(3,1,JJ)
C..... APPLY FORWARD PASS TO SMAT AND STORE RESULTS IN SMAT ....
  210 CALL PSINV(SMAT(1,1,JJ),3,3)
  220 CONTINUE
      J=J+1
      IF(J.LE.NLVL) GO TO 155
      I=I+1
      IF(I.LE.NR1) GO TO  150
      IF(NAT.EQ.0) GO TO 230
      IF(MART.EQ.2) GO TO 230
      MART=2
      NR1=NR1-1
      GO TO 145
  230 DO 235 L=1,6
      DO 235 K=1,NCLR
      UC(L,K)=0.
  235 UT(L,K)=0.
      DO 240 I=1,130
      ARP(I)=0.
      APS(I)=0.
      PIY(I)=0.
  240 PIZ(I)=0.
      DO 250 J=1,60
      DO 250 I=1,6
      FORA(I,J)=0.
      FORB(I,J)=0.
      DFORA(I,J)=0.
  250 DFORB(I,J)=0.
      NDF=6*NJT
      IF(NLPOG.GT.0) CALL DDSIN(NLPOG,ILP,NDF,NLOAD,KPRNT,LGP,NMEM)
      IF(NLPOG.LE.0) READ (*,100,END=10019)NLOAD
10019 IF(NLPOG.GT.0) PRINT 2700,ASMYC(LGP),(GAEC(I),I=1,27),
     . GAMMA(LGP),EPAC(LGP,IPSY),(EPAC(LGP,I),I=3,4),
     . (EPAC(LGP,I),I=8,14)
      PRINT 2230, NLOAD
 2230 FORMAT(///,3X,'NUMBER OF LOAD CASES=',I3)
      BIGC=1000.
      BIGS=-1000
      DO 950 JAT=1,NLOAD
      PRINT 2235,JAT
 2235 FORMAT('1',15(/),20X,'*** LOAD CASE ',I3,' ***')
      DO 260 I=1,66
      DO 260 J=1,NMEM
  260 FORM(I,J)=0.
      DO 270 I=1,NDF
      FFOR(I)=0.
  270 FOIS(I)=0.
      IF(NLPOG.GT.0) GO TO 332
      READ (*,1010,END=10020)NLINC,NFL,KPRNT
10020 IF(KPRNT.EQ.0) KPRNT=1
      PRINT 2240,NLINC,NML,KPRNT
 2240 FORMAT(///,3X,'NUMBER OF LOAD INCREMENTS=',I5,/,
     1             3X,'NUMBER OF MEMBERS LOADED =',I5,/,
     2             3X,'PRINTING INDEX            =',I5)
      IF(NML.EQ.0) GO TO 300
      DO 290 N=1,NML
      READ (*,1010,END=10021)I,NPJL
10021 PRINT 2250,I
      DO 290 NJL=1,NPJL
      READ (*,1040,END=10022)J,(FOR(L),L=1,6)
 1040 FORMAT(5X,I10,6F10.0)
 2250 FORMAT(///,4X,' LOADED NO.',I2,//,4X,' JOINT NO.',
     14X,'FORCE-X',4X,'FORCE-Y',4X,'FORCE-Z',4X,'MOMENT-X',4X,
     2'MOMENT-Y',4X,'MOMENT-Z')
10022 PRINT 2255,J,(FOR(L),L=1,6)
 2255 FORMAT(8X,I5,4X,3(1X,1PE10.3),3(2X,1PE10.3))
      J1=6*(J-1)
      DO 280 L=1,6
  280 FORM(J1+L,I)=FOR(L)
  290 CONTINUE
  300 READ (*,1005,END=10023)NFJL
10023 PRINT 2270,NFJL
 2270 FORMAT(///,3X,'NUMBER OF FRAME JOINTS LOADED=',I5)
      IF(NFJL.EQ.0) GO TO 342
      PRINT 2280
 2280 FORMAT(///,4X,'JOINT NO.',4X,'FORCE-X',4X,'FORCE-Y',4X,'FORCE-Z',
     14X,'MOMENT-X',4X,'MOMENT-Y',4X,'MOMENT-Z')
      DO 330 I=1,NFJL
      READ (*,1040,END=10024)J,(FOR(L),L=1,6)
10024 PRINT 2290,J,(FOR(L),L=1,6)
 2290 FORMAT(6X,I5,4X,3(1X,1PE10.3),3(2X,1PE10.3))
      JJ=6*(J-1)
      JMASK=1000000
      K=JR(J)
      DO 330 L=1,6
      JJ=JJ+1
      JMASK=JMASK/10
      K=K-JMASK
      IF(K) 310,320,320
  310 FFOR(JJ)=FOR(L)
      K=K+JMASK
      GO TO 330
  320 FOIS(JJ)=FOR(L)
  330 CONTINUE
      GO TO 342
C      ..... FORCES FROM LOADING GROUPS  .....
  332 DO 333 IJ=1,NDF
      FFOR(IJ)=CUMFF(IJ)
      IF(JAT.EQ.2) FFOR(IJ)=5.0*CUMFF(IJ)/12.0
  333 CONTINUE
      DO 334 IJ=1,66
      DO 334 IK=1,NMEM
      FORM(IJ,IK)=CUMMF(IJ,IK)
  334 IF(JAT.EQ.2) FORM(IJ,IK)=5.0*CUMMF(IJ,IK)/12.0
      NLINC=50-10*JAT
      IF(JAT.EQ.1) NLINC=50
      PRINT 2720
      IN=0
      MART=1
      NR=NBT
      NR1=NR+1
      IF(NR.EQ.0) NR=1
  335 I=1
  336 J=1
  337 IN=IN+1
      II=I
      IF(II.GT.NR) II=1
      IF(MART.EQ.1) IN=(II-1)*NLVL+J
      IF(MART.EQ.2) IN=NR*NLVL+J
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      NSEG=INSEG(IN)
      NNJ=NSEG+1
      PRINT 2730,IN
      PRINT 2740
      DO 335 IJ=1,NNJ
      L2=6*IJ
      L1=L2-5
  335 PRINT 2290,IJ,(PORM(K,IN),K=L1,L2)
      J=J+1
      IF(J.LE.NLVL) GO TO 337
      I=I+1
      IF(I.LE.NR1) GO TO 335
      IF(NBT.EQ.0) GO TO 339
      IF(NARY.EQ.2) GO TO 339
      NARY=2
      NR1=NR1-1
      GO TO 335
  339 PRINT 2730
      PRINT 2740
      DO 341 IJ=1,NJT
      L2=6*IJ
      L1=L2-5
  341 PRINT 2290,IJ,(PPDR(JI),JI=L1,L2)
C.....LOOP ON LOAD INCREMENTS ......
  342 DO 443 IL=1,NLINC
      NDP=6*NJT
      DO 345 I=1,NDP
      DO 345 J=1,NDP
  345 STA(I,J)=0.
      DO 346 I=1,NDP
  346 DL(I)=PPDR(I)
      IN=0
      NARY=1
      N2=NBT
      ISG=0
      NR1=NR+1
      IF(NR.EQ.0) NR=1
  350 I=1
  360 J=1
  370 IN=IN+1
      II=I
      SIG=1.0
      IF(II.GT.NR) II=1
      IF(NARY.EQ.1) IN=(II-1)*NLVL+J
      IF(NARY.EQ.2) IN=NR*NLVL+J
      IF(NARY.EQ.1.AND.IM.NE.IN) SIG=-1.0
      NSEG=INSEG(IN)
      NSEC=NSEP(IM)
      ICON=ICCON(IM)
      ITYPE(1)=IPTYP(IN,1)
      JJ=ISEN(IN)-1
      DO 380 NE=1,NSE
      ISEG=NE
      IF(NARY.EQ.2.AND.I.EQ.2) ISEG=NSEG+1-NE
      ISG=ISG+1
      DO 380 L=1,6
      DO 380 K=1,6
  380 STLL(L,K)=0.
      STPJ1=0.
      DO 430 ISECT=1,NNISS
      POR(1)=CPORS(1,ISG)
      JJ=JJ+1
      AI=ISECT-1
      AJ=NNSSG-1
      AI=AI/AJ
      DAL=HH(ISEG,IN)
      DI=DAL/AJ
      DIS=DAL*(1.-AI)
      POR(2)=CPORS(2,ISG)-DIS*CPORS(3,ISG)
      POR(3)=CPORS(4,ISG)+DIS*CPORS(2,ISG)
      CALL PSMULT(SMAT(1,1,JJ),POR,3,3,3,1)
      EPS(JJ)=EPS(JJ)+POR(1)
      FIY(JJ)=FIY(JJ)+POR(2)
      FIZ(JJ)=FIZ(JJ)+POR(3)
      NFIB=NPC(ISEG,IN)
      NSTOT=NPS(ISEG,IN)
      NSPS=NIS(ISEG,IN)-1
      NCPS=NIC(ISEG,IN)-1
      DO 390 L=1,NFIB
      IF(NARY.EQ.1) EIC(L)=EPS(JJ)+COMPY(NCPS+L)*FIY(JJ)-
     1COMPZ(NCPS+L)*FIZ(JJ)
      IF(NARY.EQ.2) EIC(L)=EPS(JJ)-COMPY(NCPS+L)*FIY(JJ)+
     1COMPZ(NCPS+L)*FIZ(JJ)
      IF(EIC(L).LT.-EU) GO TO 444
      IF(EIC(L).GE.BIGC) GO TO 390
      MLOAD=JAY
      MINC=IL
      MSEG=ISEG
      MSECT=ISECT
      BIGC=EIC(L)
      IMEM=IN
  390 CONTINUE
      DO 395 L=1,NSTOT
      IF(NARY.EQ.1) EIS(L)=EPS(JJ)+STLY(NSPS+L)*FIY(JJ)+SIG*
     1STLZ(NSPS+L)*FIZ(JJ)
      IF(NARY.EQ.2) EIS(L)=EPS(JJ)-STLY(NSPS+L)*FIY(JJ)+
     1STLZ(NSPS+L)*FIZ(JJ)
      IF(EIS(L).GT.0.01) GO TO 443
      IF(EIS(L).LE.BIGS) GO TO 395
      JLOAD =JAY
      JINC=IL
      JSEG=ISEG
      JSECT=ISECT
      BIGS=EIS(L)
      JMEM=IN
  395 CONTINUE
      AG=0.
      AST=0.
      DO 400 K=1,NFIS
  400 AG=AG+CPA(NCPS+K)
      DO 405 K=1,NSTOT
  405 AST=AST+STLA(NSPS+K)
      RATIO(ISEG)=AST/AG
      NSPS=NIS(ISEG,IN)
      NCPS=NIC(ISEG,IN)
      CALL CMSTF(COMPZ(NCPS),COMPY(NCPS),CPA(NCPS),STLZ(NSPS),
     1STLY(NSPS),STLA(NSPS),EIC,EIS,NFIB,NSTOT,ISEG,ICON)
      SMAT(1,1,JJ)= DA11
      SMAT(1,2,JJ)= DA12
      SMAT(1,3,JJ)=-DA13
      SMAT(2,1,JJ)= DA21
      SMAT(2,2,JJ)= DA22
      SMAT(2,3,JJ)=-DA23
      SMAT(3,1,JJ)=-DA31
      SMAT(3,2,JJ)=-DA32
      SMAT(3,3,JJ)= DA33
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      IF(MART.EQ.1) GO TO 406
      SMAT(1,2,JJ)=-SMAT(1,2,JJ)
      SMAT(2,1,JJ)=-SMAT(2,1,JJ)
      SMAT(1,3,JJ)=-SMAT(1,3,JJ)
      SMAT(3,1,JJ)=-SMAT(3,1,JJ)
  406 STPJ1=STPJ1+(DA22+DA33-DA32*DJT)*0.4
      CALL PSINV(SMAT(1,1,JJ),3,3)
      DO 415 L=1,3
      DO 410 K=1,3
  410 AMAT(L,K)=0.
  415 AMAT(L,L)=1.
      CALL PSMULT(SMAT(1,1,JJ),AMAT,3,3,3,3)
      C=1.
      IF(ISECT.EQ.1.OR.ISECT.EQ.MASSG) C=0.5
      IF(MART.EQ.2) GO TO 420
      STBB(2,2)=STBB(2,2)+C*C2*AMAT(1,1)
      STBB(2,3)=STBB(2,3)+C*C2*DIS*AMAT(1,3)
      STBB(1,2)=STBB(1,2)-C*C2*DIS*AMAT(1,2)
      STBB(2,6)=STBB(2,6)+C*C2*AMAT(1,2)
      STBB(2,4)=STBB(2,4)+C*C2*AMAT(1,3)
      STBB(3,3)=STBB(3,3)+C*C2*DIS*DIS*AMAT(3,3)
      STBB(1,3)=STBB(1,3)-C*C2*DIS*DIS*AMAT(3,2)
      STBB(3,6)=STBB(3,6)+C*C2*DIS*AMAT(3,2)
      STBB(3,4)=STBB(3,4)+C*C2*DIS*AMAT(3,3)
      STBB(1,1)=STBB(1,1)+C*C2*DIS*DIS*AMAT(2,2)
      STBB(1,6)=STBB(1,6)-C*C2*DIS*AMAT(2,2)
      STBB(1,4)=STBB(1,4)-C*C2*DIS*AMAT(2,3)
      STBB(6,6)=STBB(6,6)+C*C2*AMAT(2,2)
      STBB(4,6)=STBB(4,6)+C*C2*AMAT(2,3)
      STBB(4,4)=STBB(4,4)+C*C2*AMAT(3,3)
      GO TO 430
  420 STBB(1,1)=STBB(1,1)+C*C2*AMAT(1,1)
      STBB(1,2)=STBB(1,2)+C*C2*DIS*AMAT(1,3)
      STBB(1,3)=STBB(1,3)-C*C2*DIS*AMAT(1,2)
      STBB(1,5)=STBB(1,5)+C*C2*AMAT(1,2)
      STBB(1,6)=STBB(1,6)+C*C2*AMAT(1,3)
      STBB(2,2)=STBB(2,2)+C*C2*DIS*DIS*AMAT(3,3)
      STBB(2,3)=STBB(2,3)-C*C2*DIS*DIS*AMAT(3,2)
      STBB(2,5)=STBB(2,5)+C*C2*DIS*AMAT(3,2)
      STBB(2,6)=STBB(2,6)+C*C2*DIS*AMAT(3,3)
      STBB(3,3)=STBB(3,3)+C*C2*DIS*DIS*AMAT(2,2)
      STBB(3,5)=STBB(3,5)-C*C2*DIS*AMAT(2,2)
      STBB(3,6)=STBB(3,6)-C*C2*DIS*AMAT(2,3)
      STBB(5,5)=STBB(5,5)+C*C2*AMAT(2,2)
      STBB(5,6)=STBB(5,6)+C*C2*AMAT(2,3)
      STBB(6,6)=STBB(6,6)+C*C2*AMAT(3,3)
  430 CONTINUE
      STPJ=STPJ1/MASSG
      DO 440 L=1,6
      DO 440 K=1,6
  440 STBB(K,L)=STBB(L,K)
      IF(MART.EQ.1) STBB(3,3)=1.
      IF(MART.EQ.2) STBB(4,4)=1.
      CALL PSINV(STBB,6,6)
      DO 450 L=1,6
      DO 445 K=1,6
      STAA(L,K)=STBB(L,K)
  445 STBB(L,K)=0.
  450 STBB(L,L)=1.
      CALL PSMULT(STAA,STBB,6,6,6,6)
      IF(MART.NE.1) GO TO 452
      TRAT=[illegible]=(STBB(4,6)+STBB(6,4))/STBB(4,4)
      STBB(5,5)=STPJ/DAL
  452 IF(MART.EQ.2) STBB(4,4)=STPJ/DAL
      I1=ISG+IN-1
      I2=I1+1
      IF(MART.EQ.2) GO TO 465
      DO 460 L=1,6
      DO 455 K=1,6
  455 STBA(L,K)=-STBB(L,K)
      STBA(L,5)=-STBB(L,5)
      STBA(L,6)=-STBB(L,6)+DAL*STBB(L,1)
  460 STBA(L,4)=-STBB(L,4)-DAL*STBB(L,3)
      DO 470 K=1,6
      DO 465 L=1,3
      STAB(L,K)=-STBA(L,K)
  465 STAB(L,K)=-STBB(L,K)
      STAB(5,K)=-STBB(5,K)
      STAB(3,K)=-STBB(3,K)
      STAA(6,K)=-STBA(6,K)+DAL*STBA(1,K)
      STAB(6,K)=-STBB(6,K)+DAL*STBB(1,K)
      STAA(4,K)=-STBA(4,K)-DAL*STBA(3,K)
  470 STAB(4,K)=-STBB(4,K)-DAL*STBB(3,K)
      C1=(UT(1,I2)-UT(1,I1))/DAL
      C2=(UT(3,I2)-UT(3,I1))/DAL
      DO 480 L=1,6
      DO 475 K=1,6
  475 ROT(L,K)=0.
  480 ROT(L,L)=1.
      ROT(1,2)=-C1
      ROT(2,1)= C1
      ROT(2,3)= C2
      ROT(3,2)=-C2
      ROT(4,5)=-C1
      ROT(5,4)= C1
      ROT(5,6)= C2
      ROT(6,5)=-C2
      CALL MRCT(STBB,ROT)
      CALL MRCT(STAA,ROT)
      CALL MRCT(STAB,ROT)
      CALL MRCT(STBA,ROT)
      DAX=FORC(2,ISG)/DAL
      DMX=(FORC(4,IS2)-FORC(4,ISG))/(2.*DAL)
      DMZ=(FORC(6,IS2)-FORC(6,ISG))/(2.*DAL)
      STBB(1,5)=STBB(1,5)+DMX
      STBB(3,5)=STBB(3,5)-DMZ
      STBB(5,1)=STBB(1,5)
      STBB(5,3)=STBB(3,5)
      STAA(1,5)=STAA(1,5)+DMX
      STAA(3,5)=STAA(3,5)-DMZ
      STAA(5,1)=STAA(1,5)
      STAA(5,3)=STAA(3,5)
      STAB(1,5)=STAB(1,5)-DMX
      STAB(3,5)=STAB(3,5)+DMZ
      STAB(5,1)=STAB(5,1)-DMX
      STAB(5,3)=STAB(5,3)+DMZ
      STBA(1,5)=STAB(5,1)
      STBA(3,5)=STAB(5,3)
      STBA(5,1)=STAB(1,5)
      STBA(5,3)=STAB(3,5)
      STBB(1,1)=STBB(1,1)+DAX
      STBB(3,3)=STBB(3,3)+DAX
      STAA(1,1)=STAA(1,1)+DAX
      STAA(3,3)=STAA(3,3)+DAX
      STAB(1,1)=STAB(1,1)-DAX
      STAB(3,3)=STAB(3,3)-DAX
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      STBA(1,1)=STBA(1,1)-DAK
      STBA(3,3)=STBA(3,3)-DAK
      STBB(5,3)=STBB(5,3)+DAK*TRAT
      STAA(5,5)=STAA(5,5)+DAK*TRAT
      STAB(5,5)=STAB(5,5)-DAK*TRAT
      STBA(5,5)=STBA(5,5)-DAK*TRAT
      GO TO 515
485   DO 495 L=1,6
      DO 490 K=1,6
490   STBA(L,K)=-STBB(L,K)
      STBA(L,3)=STBA(L,3)+DAL*STBB(L,3)
495   STBA(L,6)=STBA(L,6)-DAL*STBB(L,2)
      DO 510 K=1,6
      DO 500 L=1,6
      STAA(L,K)=-STBA(L,K)
500   STAB(L,K)=STBA(K,L)
      STAA(3,K)=STAA(3,K)+DAL*STBB(3,K)
510   STAA(6,K)=STAA(6,K)-DAL*STBA(2,K)
515   CONTINUE
      DO 550 L=1,6
      DO 550 K=1,6
      SKAA(L,K,ISG)=STAA(L,K)
      SKAB(L,K,ISG)=STAB(L,K)
      SKBB(L,K,ISG)=STBB(L,K)
      IF(ME-1) 520,520,530
520   SA(L,K,I1)=STAA(L,K)
      GO TO 540
530   SA(L,K,I1)=SA(L,K,I1)+STAA(L,K)
540   SA(L,K,I2)=STBB(L,K)
      SB(L,K,ISG)=STAB(L,K)
550   SC(L,K,ISG)=STAB(K,L)
      DO 560 L=1,6
      J1=(ME-1)*6
      UU(L,I1)=FORM(J1+L,IN)
      IF(ME.LT.NSEG) GO TO 560
      J1=NSEG*6
      UU(L,I2)=FORM(J1+L,IN)
560   CONTINUE
580   CONTINUE
      N1=NSEG
      N2=N1-1
      JSG=I1-NSEG
      JJSG=ISG-NSEG
      DO 615 L=2,N2
      DO 605 K=1,6
      DO 605 M=1,6
605   STAA(K,M)=0.
      KSG=JSG+L
      KSJ=KSG+1
      KKSG=JJSG+L
      LSG=KKSG-1
      KKSJ=KKSJ+1
      CALL PSINV(SA(1,1,KSG),6,6)
      CALL PSMULT(SA(1,1,KSG),SB(1,1,KKSG),6,6,6,6)
      CALL MATMUL(SC(1,1,KKSG),SB(1,1,KKSG),SA(1,1,KSJ),6,6,6,6)
      CALL PSMULT(SA(1,1,KSG),SC(1,1,LSG),6,6,6,6)
      CALL PSMULT(SA(1,1,KSG),UU(1,KSG),6,6,6,1)
      CALL MATMUL(SC(1,1,KKSG),UU(1,KSG),UU(1,KSJ),6,6,6,1)
      CALL MATMUL (SC(1,1,KKSG),SC(1,1,LSG),STAA,6,6,6,6)
      DO 610 K=1,6
      DO 610 M=1,6
610   SC(K,M,KKSG)=STAA(K,M)
615   CONTINUE
      CALL PSINV(SA(1,1,KSJ),6,6)
      CALL PSMULT(SA(1,1,KSJ),UU(1,KSJ),6,6,6,1)
      CALL PSMULT(SA(1,1,KSJ),SB(1,1,KKSJ),6,6,6,6)
      CALL PSMULT(SA(1,1,KSJ),SC(1,1,KKSG),6,6,6,6)
      DO 630 L=2,N2
      DO 620 K=1,6
      DO 620 M=1,6
620   STAA(K,M)=0.
      LL=N1-L+1
      KKSG=JJSG+LL
      KKSJ=KKSG+1
      LSG=KKSG-1
      KSG=JSG+LL
      KSJ=KSG+1
      CALL MATMUL(SB(1,1,KKSG),UU(1,KSJ),UU(1,KSG),6,6,6,1)
      CALL MATMUL(SB(1,1,KKSG),SC(1,1,KKSG),SC(1,1,LSG),6,6,6,6)
      CALL MATMUL(SB(1,1,KKSG),SB(1,1,KKSJ),STAA,6,6,6,6)
      DO 625 K=1,6
      DO 625 M=1,6
625   SB(K,M,KKSG)=STAA(K,M)
630   CONTINUE
      IN1=JSG+1
      INS=JSG+NSEG
      IINS=JJSG+NSEG
      JNS=INS-1
      IN2=IN1+1
      INS2=INS+1
      JIN1=JJSG+1
      JINS=JJSG+NSEG
      JIN2=JIN1+1
      JJNS=IINS-1
      CALL MATMUL(SB(1,1,JIN1),SC(1,1,JIN1),SA(1,1,IN1),6,6,6,6)
      CALL MATMUL(SC(1,1,JINS),SB(1,1,JINS),SA(1,1,INS2),6,6,6,6)
      CALL MATMUL(SB(1,1,JIN1),UU(1,IN2),UU(1,IN1),6,6,6,1)
      CALL MATMUL(SC(1,1,JINS),UU(1,INS),UU(1,INS2),6,6,6,1)
      DO 635 K=1,6
      DO 635 M=1,6
635   STAA(K,M)=0.
      CALL MATMUL(SB(1,1,JIN1),SB(1,1,JIN2),STAA,6,6,6,6)
      DO 645 K=1,6
      DO 645 M=1,6
      SB(K,M,JIN1)=STAA(K,M)
645   STAA(K,M)=0.
      CALL MATMUL(SC(1,1,JINS),SC(1,1,JJNS),STAA,6,6,6,6)
      DO 655 K=1,6
      DO 655 M=1,6
655   SC(K,M,JINS)=STAA(K,M)
      DO 660 K=1,6
      DO 660 M=1,6
      TRAA(K,M,IN)=SA(K,M,IN1)
      TRAB(K,M,IN)=SB(K,M,JIN1)
660   TRAE(K,M,IN)=SA(K,M,INS2)
      JM=(J-1)*NCOL
      JP=JM+NCOL
      IF(MART.EQ.1) GO TO 670
      JP=JM+NCOL
      JP=JM+1
      KM=JM
670   II=6*(JP-1)
      JJ=6*(JP-1)
      DO 675 L=1,6
      DO 675 M=1,6
      II=II+L
```

```
      JJJ=II+M
      STK(III,JJJ)=STK(III,JJJ)+TK44(L,M,IN)
      JJJ=JJ+M
      STK(III,JJJ)=STK(III,JJJ)+TK45(L,M,IN)
      STK(JJJ,III)=STK(III,JJJ)
      III = JJ+L
  575 STK(III,JJJ) = STK(III,JJJ)+TK56(L,M,IN)
      DO 600 L=1,6
      III=II+L
      JJJ = JJ+L
      DU (III)=DU(III)+UU(L,IN1)
  600 DU(JJJ)=DU(JJJ)+UU(L,INS2)
      J=J+1
      IF(J.LE.NLVL) GO TO 370
      I=I+1
      IF(I.LE.NR1) GO TO 360
      IF(NAV.EQ.0) GO TO 690
      IF (MART.EQ.2) GO TO  690
      MART =2
      NR1=NR1-1
      GO TO 350
  690 CONTINUE
      DO 700 I=1,NJT
      II=6*(I-1)
      STK(II+1,II+1)=STK (II+1,II+1)+AKX(I)
      STK(II+2,II+2)=STK(II+2,II+2)+AKY (I)
      STK(II+3,II+3)=STK(II+3,II+3)+AKZ(I)
      STK(II+4,II+4)=STK(II+4,II+4)+AKPX(I)
      STK(II+5,II+5)=STK(II+5,II+5)+AKPY(I)
  700 STK(II+6,II+6)=STK(II+6,II+6)+AKPZ(I)
      DO 725 I=1,NJT
      IA=I-1
      II=6*IA
      JMASK=1000000
      K= JR(I)
      DO 720 L=1,6
      II=II+1
      JMASK=JMASK/10
      KK=K-JMASK
      IF (K) 705,710,710
  705 K=K+JMASK
      GO TO  720
  710 AA=PDIS(II)
      DO 715 J=1,NDF
      DU(J)=DU(J)-STK(J,II)*AA
      STK(J,II)=0.
  715 STK (II,J)=0.
      STK (II,II)=1.
      DU(II)=AA
  720 CONTINUE
  725 CONTINUE
      SPV=1.
      CALL PSINV (STK,72,NDF)
      DO 726 MAN=1,NJT
      MAN1=(MAN-1)*6+1
      MAN5=MAN1+4
      MAN5=MAN1+4
      MAN6=MAN1+5
      DO 726 IMAN=MAN1,MAN3
      IF(STK(IMAN,IMAN).LT.0.) SPV=-SPV
  726 CONTINUE
      IF(STK(MAN5,MAN5).LT.0.) SPV=-SPV
      IF(STK(MAN6,MAN6).LT.0.) SPV=-SPV
      IF(SPV.LT.0.) GO TO 905
      CALL PSPULT (STK,DU,72,NDF,72,1)
      IN=0
      MART = 1
      NR=NAT
      ISG=0
      NR1=NR-1
      IF(NR.EQ.0) NR=1
  730 I=1
  735 J=1
  740 IN=IN+1
      II=I
      IF(II.GT.NR)II=1
      IF(MART.EQ.1)IN=(II-1)*NLVL+J
      IF(MART.EQ.2) IN=NR*NLVL+J
      NSEG=INSEG(IN)
      NSEC=NSEN(IN)
      ISEC=ISEM(IN)
      ICCN=ICCON(IN)
      ITYPE(1)=INTYP(IN,1)
      JM=I+(J-1)*NCOL
      JP=JM+NCOL
      IF(MART.EQ.1) GO TO 745
      JM=JM+NCOL
      JP=JM+1
      KM=JM
  745 II=6*(JM-1)
      JJ=6*(JP-1)
      IN1=ISG+IN
      INS2=IN1+NSEG
      DO 750 L=1,6
      UU(L,IN1)=DU(II+L)
  750 UU(L,INS2)=DU(JJ+L)
      N1=NSEG
      N2=N1-1
      JSG=ISG+1
      I1=JSG+IN-1
      DO 760 L=2,NSEG
      JSG=JSG+1
      I1=I1+1
      KSG=JSG-1
      CALL MATMUL(SC(1,1,KSG),UL(1,IN1),LU(1,I1),6,6,6,1)
  760 CALL MATMUL(SB(1,1,JSG),UU(1,INS2),UU(1,I1),6,6,6,1)
      DO 790 ISEG=1,NSEG
      ISG=ISG+1
      I1=ISG+IN-1
      I2=I1+1
      DO 780 L=1,6
      CUM=0.
      SUM=0.
      DO 770 M=1,6
      SUM=SUM+SKAA(L,M,ISG)*UU(M,I1)+SKAB(L,M,ISG)*UU(M,I2)
  770 CUM=CUM+SKAB(M,L,ISG)*UU(M,I1)+SKBB(L,M,ISG)*UU(M,I2)
      FORA(L,ISG)=FORA(L,ISG)+SUM
      FOR(L)=CUM
  780 FORB(L,ISG)=FORB(L,ISG)+CUM
      IF(MART.EQ.2) GO TO 781
      DAL=H*(ISEG,IP)
      C1=(UT(1,I2)-UT(1,I1))/DAL
      C2=(UT(3,I2)-UT(3,I1))/DAL
      DUS(1)=FOR(1)-C1*FOR(3)
      DUS(2)=C1*FOR(1)+FOR(2)+C2*FOR(3)
      DUS(3)=-C2*FOR(2)+FOR(3)
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      DUB(4)=FOR(6)-C1*FOR(5)
      DUB(5)=C1*FOR(4)+FOR(5)+C2*FOR(6)
      DUB(6)=-C2*FOR(5)+FOR(6)
      DFORB(1,ISG)=DUB(2)
      DFORB(3,ISG)=DUB(1)
      DFORB(2,ISG)=DUB(3)
      DFORB(5,ISG)=DUB(6)
      DFORB(6,ISG)=DUB(4)
      AXP(ISG)=AXP(ISG)+DUB(2)
      GO TO 790
  781 DO 782 ME=1,6
  782 DFORB(ME,ISG)=FOR(ME)
  790 CONTINUE
      JSG=ISG-NSEG
      DO 800 ISEG=1,NSEG
      JSG=JSG+1
      I1=JSG+IN-1
      DO 800 L=1,6
  800 UT(L,I1)=UT(L,I1)+UL(L,I1)
      I1=I1+1
      DO 805 L=1,6
  805 UT(L,I1)=UT(L,I1)+UU(L,I1)
C.....PRINTING BLOCK ......
      IF(IL.EQ.1) GO TO 619
      JPRNT=IL-KPRNT*(IL/KPRNT)
      IF(JPRNT.NE.0) GO TO 852
  619 CONTINUE
      PRINT 2320,IN
 2320 FORMAT('1',20X,' MEMBER ',I3,/,20X,'*************',//)
      PRINT 2400,IL
      JSG=ISG-NSEG
      DO 920 ISEG=1,NSEG
      JSG=JSG+1
      I1=JSG+IN-1
      PRINT 2420
  920 PRINT 2425,I1,(UT(L,I1),L=1,6)
      I1=I1+1
      PRINT 2420
      PRINT 2425,I1,(UT(L,I1),L=1,6)
      JSG=ISG-NSEG
      PRINT 2426
      DO 951 ISEG=1,NSEG
      JSG=JSG+1
      PRINT 2430,ISEG
      PRINT 2435,(FORA(L,JSG),L=1,6)
  951 PRINT 2440,(FORB(L,JSG),L=1,6)
  952 J=J+1
      IF(J.LE.NLVL) GO TO 740
      I=I+1
      IF(I.LE.NR1) GO TO 735
      IF(NBY.EQ.0) GO TO 850
      IF(MARY.EQ.2) GO TO 850
      MARY=2
      NR1=NR1-1
      GO TO 720
  850 CONTINUE
  945 CONTINUE
  950 CONTINUE
      PRINT 2445,ISEC,NLOAD,INBM,PINC,PSEC,ASECT
      PRINT 2450,ISGS,JLOAD,JMEM,JINC,JSEG,JSECT
      GO TO 960
  961 PRINT 2340
 2340 FORMAT('0','NUMBER OF STEEL FIBERS EXCEEDED')
      GO TO 960
  962 PRINT 2350
 2350 FORMAT('0','NUMBER OF CONCRETE FIBERS EXCEEDED')
      GO TO 960
  963 IFAIL=-1
      CALL FAIL(LGP,JAY,IL,IFAIL,ISECT,ISEG,IN)
      GO TO 966
  964 IFAIL=1
      CALL FAIL(LGP,JAY,IL,IFAIL,ISECT,ISEG,IN)
      GO TO 964
  965 IFAIL=0
      CALL FAIL(LGP,JAY,IL,IFAIL,ISECT,ISEG,IN)
C.....PRINTING OF FORCES AND DISPLACEMENTS PREVIOUS TO FAILURE
  966 PRINT 2600
      IL=IL-1
      IN=0
      MARY=1
      NR=NBY
      ISG=0
      NR1=NR+1
      IF(NR.EQ.0) NR=1
  967 I=1
  968 J=1
  969 IN=IN+1
      II=I
      IF(II.GT.NR) II=1
      IF(MARY.EQ.1) IN=(II-1)*NLVL+J
      IF(MARY.EQ.2) IN=NR*NLVL+J
      NSEG=INSEG(IN)
      PRINT 2320,IN
      PRINT 2400,IL
      JSG=ISG
      DO 971 ISEG=1,NSEG
      JSG=JSG+1
      I1=JSG+IN-1
      PRINT 2420
  971 PRINT 2425,I1,(UT(L,I1),L=1,6)
      I1=I1+1
      PRINT 2420
      PRINT 2425,I1,(UT(L,I1),L=1,6)
      JSG=ISG
      PRINT 2426
      DO 972 ISEG=1,NSEG
      JSG=JSG+1
      PRINT 2430,ISEG
      PRINT 2435,(FORA(L,JSG),L=1,6)
  972 PRINT 2440,(FORB(L,JSG),L=1,6)
      ISG=ISG+NSEG
      J=J+1
      IF(J.LE.NLVL) GO TO 969
      I=I+1
      IF(I.LE.NR1) GO TO 968
      IF(NBY.EQ.0) GO TO 960
      IF(MARY.EQ.2) GO TO 960
      MARY=2
      NR1=NR1-1
      GO TO 967
  960 ILF=ILF+1
      IF(ILF.LE.NLADS) GO TO 143
      STOP
 2400 FORMAT('0',2X,'LOAD INCREMENT',I3,/,
     1             3X,'*******************',
     2//,20X,'>>> DISPLACEMENTS <<<',/)
```

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2420 FORMAT(//,3X,'JOINT NO.',6X,'DISPL.-X',6X,'DISPL.-Y',6X,
    1'DISPL.-Z',2X,'ROTATION-X',2X,'ROTATION-Y',2X,'ROTATION-Z')
2425 FORMAT(5X,I3,2X,6(2X,1PE10.3))
2428 FORMAT(//,30X,'>>> FORCES <<<',/)
2430 FORMAT(//,4X,'SEG.NO.',5X,'FORCE-X',5X,'FORCE-Y',5X,'FORCE-Z',
    16X,'MOMENT-X',6X,'MOMENT-Y',6X,'MOMENT-Z',/,6X,I5)
2435 FORMAT(7X,'END-A',6(2X,1PE10.3),/)
2440 FORMAT(7X,'END-B',6(2X,1PE10.3),/)
2445 FORMAT('1',2X,'AT THE END OF ALL LOADING',/,
    1          3X,'*************************',///,
    2          3X,'MAX. COMPRESSIVE CONCRETE STRAIN=',1PE10.3,/,
    3          3X,'OCCURED IN LOAD CASE          =',I5,/,
    4          3X,'OCCURED IN MEMBER             =',I5,/,
    5          3X,'OCCURED IN INCREMENT          =',I5,/,
    6          3X,'OCCURED IN SEGMENT            =',I5,/,
    7          3X,'OCCURED IN SECTION            =',I5,/)
2450 FORMAT('0',2X,'MAX. TENSILE STEEL STRAIN=',1PE10.3,/,
    1          3X,'OCCURED IN LOAD CASE     =',I5,/,
    2          3X,'OCCURED IN MEMBER        =',I5,/,
    3          3X,'OCCURED IN INCREMENT     =',I5,/,
    4          3X,'OCCURED IN SEGMENT       =',I5,/,
    5          3X,'OCCURED IN SECTION       =',I5,/)
2460 FORMAT('1',20(/),40X,'**************************************',
    *        '**************',/,40X,'**                         ',
    *        '                      **',/40X,'** FORCES AND DISPLA',
    *        'CEMENTS PREVIOUS TO FAILURE **',/40X,'**              ',
    *        '                      **'/,40X,
    *        '****************************************************')
2700 FORMAT('1',20X,'*********************',/,21X,'*  LOAD GROUP',
    1          A6,'   *',/,21X,'*********************',///,2X,27A6,///,
    2          5X,'GAMMA VALUE =',F5.2,//,6X,'BETA VALUES:',2X,'BU =',
    3 F5.2,  /,20X,'BL =',F5.2,/,20X,'BC =',F5.2,/,20X,'BE =',F5.2,/,
    4          20X,'BI =',F5.2,/,20X,'BS =',F5.2,/,20X,'BW =',F5.2,/,
    5          20X,'BWLM',F5.2,/,20X,'BL =',F5.2,/,20X,'BR =',F5.2,/,
    6          20X,'BEQ=',F5.2,/,19X,'BICE=',F5.2)
2720 FORMAT('0',4X,'THE FACTORED INCREMENTAL FORCES ',
    *        'FOR THIS LOAD CASE ARE:',///,8X,
    *        '* MEMBER FORCES *',//)
2730 FORMAT('0',30X,'MEMBER',I3,//)
2740 FORMAT('0',4X,'JOINT NO.',6X,'FORCE-X',4X,'FORCE-Y',6X,
    *        'FORCE-Z',6X,'MOMENT-X',6X,'MOMENT-Y',4X,'MOMENT-Z',/)
2750 FORMAT('1',//////,5X,'* FRAME JOINT FORCES *',//)
     END
C*P45V1PG*
     SUBROUTINE MROT(A,B)
     IMPLICIT DOUBLE PRECISION(A-H,O-Z)
     DIMENSION A(6,6),B(6,6),C(6)
     DO 1 I=1,6
     DO 2 J=1,6
     SUM=0.
     DO 3 K=1,6
3    SUM=SUM+A(I,K)*B(K,J)
2    C(J)=SUM
     DO 4 J=1,6
4    A(I,J)=C(J)
1    CONTINUE
     DO 5 J=1,6
     DO 6 I=1,6
     SUM=0.
     DO 7 K=1,6
7    SUM=SUM+A(K,J)*B(K,I)
6    C(I)=SUM
     DO 6 I=1,6
6    A(I,J)=C(I)
5    CONTINUE
     RETURN
C**  THIS PROGRAM VALID ON FTN4 AND FTN5 **
     END
C*P45V1PG*
     SUBROUTINE MATMUL(A,B,C,NDIM,N,MDIM,M)
     IMPLICIT DOUBLE PRECISION(A-H,O-Z)
     DIMENSION A(NDIM,NDIM),B(MDIM,MDIM),C(NDIM,NDIM)
     DO 10 I=1,N
     DO 10 J=1,M
     SUM=0.
     DO 11 K=1,N
11   SUM=SUM+A(I,K)*B(K,J)
10   C(I,J)=C(I,J)-SUM
     RETURN
C**  THIS PROGRAM VALID ON FTN4 AND FTN5 **
     END
C*P45V1PD*
     SUBROUTINE PSINV(A,NDIM,N)
     IMPLICIT DOUBLE PRECISION(A-H,O-Z)
     DIMENSION A(NDIM,NDIM)
     N1=N-1
     DO 10 I=1,N1
     C=1./A(I,I)
     I1=I+1
     DO 11 J=I1,N
11   A(I,J)=A(I,J)*C
     DO 12 K=I1,N
     DO 12 J=I1,N
12   A(K,J)=A(K,J)-A(K,I)*A(I,J)
10   CONTINUE
     RETURN
C**  THIS PROGRAM VALID ON FTN4 AND FTN5 **
     END
C*P45V1PD*
     SUBROUTINE PSMULT(A,B,NDIM,N,MDIM,M)
     IMPLICIT DOUBLE PRECISION(A-H,O-Z)
     DIMENSION A(NDIM,NDIM),B(MDIM,MDIM)
     N1=N-1
     DO 10 I=1,N1
     C=1./A(I,I)
     DO 11 J=1,M
11   B(I,J)=B(I,J)*C
     I1=I+1
     DO 12 K=I1,N
     DO 12 J=1,M
12   B(K,J)=B(K,J)-A(K,I)*B(I,J)
10   CONTINUE
     C=1./A(N,N)
     DO 13 J=1,M
13   B(N,J)=B(N,J)*C
     DO 14 II=1,N1
     I=N-II
     I1=I+1
     DO 15 K=I1,N
     DO 15 J=1,M
15   B(I,J)=B(I,J)-A(I,K)*B(K,J)
14   CONTINUE
     RETURN
C**  THIS PROGRAM VALID ON FTN4 AND FTN5 **
     END
C*P45V1PD*
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      SUBROUTINE DSCRIS(CONPX,CONPY,CPA,ICODE,NPIS)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      CHARACTER*4,IOR(3),INO,IH,IV,ICODE
      DIMENSION CONPX(200),CONPY(200),CPA(200)
      COMMON /ONE/ XI,TP,YI,TW
      COMMON /TWO/ CTROID(5),TWALL(5),NW,NWX,NWY
      COMMON /THRE/ IOR,INO,IH,IV
      COMMON /SUP/ IPRNT
      COMMON /SAME/ KCTYPE,KSTYPE,KPY,KWX,NX,NY,KY,KW,NI
      REAL INNERX,INNERY
      IXI=XI
      IYI=YI
      KPX=2
      KWY=2
      FACT=16.
      IF(ICODE.NE.INO) FACT=16.-NX*16.
      IF(KCTYPE.GT.0) GO TO 25
      XKPY=FACT/(4.*(1.+YI/XI))
      KPY=XKPY
      XKWX=YI/XI*XKPY
      KWX=XKWX
   25 XPINC=(XI-2.0*TW)/KPY
      YPINC=TP/KPX
      XWINC=TW/KWY
      YWINC=(YI-2.0*TP)/KWX
      NPIS=2*KPX*KPY+2*KWX*KWY+4*KPX*KWY
      INNERX=XI-2.0*TW
      INNERY=YI-2.0*TP
      HIX=INNERX/2.0
      HIY=INNERY/2.0
      STATX=-XI/2.0+XWINC/2.0
      STATY=YI/2.0-YPINC/2.0
      KC=1
      Y=STATY
      IT=0
98    DO 12 I=1,KPX
      X=STATX
      DO  20 J=1,KWY-1
      CONPX(KC)=X
      CONPY(KC)=Y
      CPA(KC)=XWINC*YPINC
      X=X+XWINC
20    KC=KC+1
      CONPX(KC)=X
      CONPY(KC)=Y
      CPA(KC)=XWINC*YPINC
      KC=KC+1
      X=X+XPINC/2.0+XWINC/2.0
      CONPX(KC)=X
      CONPY(KC)=Y
      CPA(KC)=XPINC*YPINC
      DO 30 K=1,KPY-1
      X=X+XPINC
      KC=KC+1
      CPA(KC)=XPINC*YPINC
      CONPX(KC)=X
30    CONPY(KC)=Y
      X=X+XPINC/2.0+XWINC/2.0
      KC=KC+1
      CONPX(KC)=X
      CONPY(KC)=Y
      CPA(KC)=XWINC*YPINC
      KC=KC+1
      DO 40 JK=1,KWY-1
      X=X+XWINC
      CONPX(KC)=X
      CONPY(KC)=Y
      CPA(KC)=XWINC*YPINC
40    KC=KC+1
10    Y=Y-YPINC
      IF(IT.NE.0) GO TO 99
      Y=Y+YPINC/2.0-YWINC/2.0
      DO 90 L=1,KWX
      X=STATX
      DO 60 M=1,KWY-1
      CONPX(KC)=X
      CONPY(KC)=Y
      CPA(KC)=XWINC*YWINC
      X=X+XWINC
60    KC=KC+1
      CONPX(KC)=X
      CONPY(KC)=Y
      CPA(KC)=XWINC*YWINC
      KC=KC+1
      X=X+INNERX+XWINC
      CONPX(KC)=X
      CONPY(KC)=Y
      CPA(KC)=XWINC*YWINC
      KC=KC+1
      DO 70 N=1,KWY-1
      X=X+XWINC
      CONPX(KC)=X
      CONPY(KC)=Y
      CPA(KC)=XWINC*YWINC
70    KC=KC+1
90    Y=Y-YWINC
      Y=Y+YWINC/2.0-YPINC/2.0
      IT=1
      GO TO 98
99    CONTINUE
      NPP=KPX*KPY*KPX*KWY*2
      NWP=KWX*KWY
      IF(IPRNT.EQ.0) GO TO 113
      PRINT 100
100   FORMAT (///,10X,'CONCRETE PIECE PROPERTIES',
     1//,3X,'NO.',6X,'X',12X,'Y',12X,'AREA')
      DO 110 MC=1,NPIS
      PRINT 102,MC,CONPX(MC),CONPY(MC),CPA(MC)
102   FORMAT (I5,3X,F10.2,2X,F10.2,3X,F10.2)
110   CONTINUE
113   CONTINUE
      IF(ICODE.EQ.INO) GO TO 199
      IF(IOR(1).EQ.IH) GO TO 140
      IF(IOR(1).EQ.IV) GO TO 130
130   NWX=2
      NWY=5
      WALLX=INNERX/NWY
      ANWX=NWX
      DO 150 L=1,NW
      WALLY=TWALL(L)/ANWX
      YSTWL=CTROID(L)+(ANWX-1.0)*.5*WALLY
      WY=YSTWL+WALLY
      DO 160 K=1,NWX
      WY=WY-WALLY
      WX=-XI/2.0+TW-WALLX/2.0
      DO 170 M=1,NWY
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      WX=WX+WALLX
      COMPX(KC)=WX
      COMPY(KC)=WY
      CPA(KC)=WALLX*WALLY
170   KC=KC+1
160   CONTINUE
150   CONTINUE
      NWLPI6=NW*NWX*NWY
      INDEX=NPI3+1
      NPI6=NPI6+NWLPI5
      IF(IPRNT.EQ.0) GO TO 199
      DO 190 I=INDEX,NPI3
      PRINT 102,I,COMPX(I),COMPY(I),CPA(I)
190   CONTINUE
      GO TO 199
140   NWX=5
      NWY=2
      ANWY=NWY
      ANWX=NWX
      WALLY=INNERY/ANWX
      DO 210 L=1,NW
      WALLX=TWALL(L)/ANWY
      XSTWL=CTADIS(L)-(ANWY-1.0)*.5*WALLX
      WX=XSTWL-WALLX
      DO 220 K=1,NWY
      WX=WX+WALLX
      WY=YI/2.0-TP+WALLY/2.0
      DO 230 M=1,NWX
      WY=WY-WALLY
      COMPX(KC)=WX
      COMPY(KC)=WY
      CPA(KC)=WALLX*WALLY
230   KC=KC+1
220   CONTINUE
210   CONTINUE
      NWLPI6=NW*NWX*NWY
      INDEX=NPI6+1
      NPI6=NPI3+NWLPI2
      IF(IPRNT.EQ.0) GO TO 199
      DO 250 I=INDEX,NPI3
      PRINT 102,I,COMPX(I),COMPY(I),CPA(I)
250   CONTINUE
199   CONTINUE
      KCTYPE=KCTYPE+1
      RETURN
      END
C*PASVIP2*
      SUBROUTINE SSCALE(STLX,STLY,STLA,ICODE,NSTOT,ITYP)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      CHARACTER*4,IOP(3),IND,IP,IV,ISOLID,IHOLL,IOVAL,
     * IWHOLL,ITYP,ICODE
      DIMENSION STLX(200),STLY(200),STLA(200)
      COMMON /ONE/ XI,YI,TI,TW
      COMMON /TWO/ CTADIS(5),TWALL(5),NW,NWX,NWY
      COMMON /THREE/ IOP,IND,IP,IV
      COMMON /FOUR/ STT(3),STS(3),GAMMAX(3),GAMMAY(3),NUSTL(5),NRING
      COMMON /FIVE/ ISOLID,IHOLL,IOVAL,IWHOLL
      COMMON /SIX/ IPRNT
      COMMON /SEVEN/ KCTYPE,KSTYPE,KPT,NAX,NX,NY,NV,KN,NZ
      REAL INNERX,INNERY
      NWX=10
      NWY=10
      KC=1
      IF(KSTYPE.GT.0) GO TO 25
      VAL=200./NRING
      IF(ITYP.EQ.IHOLL.AND.ICODE.NE.IND) VAL=(200.-NW*10.)/NRING
      IVAL=VAL
      XNX=VAL/(2.*(1.+YI/XI))
      NX=XNX
      NY=YI/XI*NX
25    NSTOT=(2*(NX+NY))*NRING
      DO 10 I=1,NRING
      GX=XI-2.0*GAMMAX(I)
      GY=YI-2.0*GAMMAY(I)
      HT=STT(I)/GX
      HS=STS(I)/GY
      XINC=GX/NX
      YINC=GY/NY
      STATX=-XI/2.0+GAMMAX(I)
      STATY=YI/2.0-GAMMAY(I)
      X=STATX-XINC/2.0
      Y=STATY
      DO 20 J=1,NX
      X=X+XINC
      STLX(KC)=X
      STLY(KC)=Y
      STLA(KC)=XINC*HT
20    KC=KC+1
      X=STATX-XINC/2.0
      Y=-STATY
      DO 30 K=1,NX
      X=X+XINC
      STLX(KC)=X
      STLY(KC)=Y
      STLA(KC)=XINC*HT
30    KC=KC+1
      X=STATX
      Y=STATY+YINC/2.0
      DO 40 L=1,NY
      Y=Y-YINC
      STLX(KC)=X
      STLY(KC)=Y
      STLA(KC)=YINC*HS
40    KC=KC+1
      X=-STATX
      Y=STATY+YINC/2.0
      DO 50 M=1,NY
      Y=Y-YINC
      STLX(KC)=X
      STLY(KC)=Y
      STLA(KC)=YINC*HS
50    KC=KC+1
10    CONTINUE
      IF(IPRNT.EQ.0) GO TO 213
      PRINT 100
100   FORMAT (///,10X,'STEEL FIBER PROPERTIES'
     1//,3X,'NO.',8X,'X',15X,'Y',10X,'AREA')
      PRINT 110,(K,STLX(K),STLY(K),STLA(K),K=1,NSTOT)
110   FORMAT (I5,2X,F10.2,6X,F10.2,5X,F10.2)
213   CONTINUE
      IF(ITYP.EQ.ISOLID) GO TO 199
      IF(ICODE.EQ.IND) GO TO 199
      IF(IOP(1).EQ.IP) GO TO 140
      IF(IOP(1).EQ.IV) GO TO 120
130   INNERX=XI-2.0*TW
      WALLX=INNERX/NWY
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      DO 130 L=1,NW
      HW=WLSTL(L)/INNERX
      YSTWL=CTROIL(L)
      WX=-XI/2.0+TW-WALLX/2.0
      DO 160 K=1,NWY
      WX=WX+WALLX
      STLX(KC)=WX
      STLY(KC)=YSTWL
      STLA(KC)=HW*WALLX
160   KC=KC+1
150   CONTINUE
      INDEX=NSTOT+1
      NSTOT=NSTOT+NW*NWY
      IF(IPRNT.EQ.0) GO TO 199
      PRINT 110,(I,STLX(I),STLY(I),STLA(I),I=INDEX,NSTOT)
      GO TO 199
140   INNERY=YI-2.0*TF
      WALLY=INNERY/NWX
      DO 170 K=1,NW
      HW=WLSTL(K)/INNERY
      XSTWL=CTROID(K)
      WY=YI/2.0-TF+WALLY/2.0
      DO 180 M=1,NWX
      WY=WY-WALLY
      STLX(KC)=XSTWL
      STLY(KC)=WY
      STLA(KC)=HW*WALLY
180   KC=KC+1
170   CONTINUE
      INDEX=NSTOT+1
      NSTOT=NSTOT+NW*NWX
      IF(IPRNT.EQ.0) GO TO 199
      PRINT 110,(I,STLX(I),STLY(I),STLA(I),I=INDEX,NSTOT)
199   CONTINUE
      KSTYPE=KSTYPE+1
      RETURN
      END
C*F45V1P0*
      SUBROUTINE CNSTF(CONFX,CONFY,CFA,STLX,STLY,STLA,EIC,EIS,
     1NFIB,NSTOT,ISEG,ICON)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      DIMENSION CONFX(200),CONFY(200),CFA(200),STLX(200),STLY(200),
     1STLA(200),EIC(200),EIS(200)
      COMMON /STF/ DA11,DA12,DA13,DA21,DA22,DA23,DA31,DA32,DA33
      COMMON /LC/ SIG
      DA11=0.0
      DA12=0.0
      DA13=0.0
      DA21=0.0
      DA22=0.0
      DA23=0.0
      DA31=0.0
      DA32=0.0
      DA33=0.0
      DO 10 I=1,NFIB
      ECON=EIC(I)
      IF(ICON.EQ.0) CALL FC(ECON,SC,ETC)
      IF(ICON.EQ.1) CALL FCON(ECON,SC,ETC)
      DA11=DA11+ETC*CFA(I)
      DA12=DA12+ETC*CONFY(I)*CFA(I)
      DA13=DA13+ETC*CONFX(I)*CFA(I)
      DA21=DA21+ETC*CONFY(I)*CFA(I)
      DA22=DA22+ETC*CONFY(I)**2*CFA(I)
      DA23=DA23+ETC*CONFX(I)*CONFY(I)*CFA(I)
      DA31=DA31+ETC*CONFX(I)*CFA(I)
      DA32=DA32+ETC*CONFX(I)*CONFY(I)*CFA(I)
      DA33=DA33+ETC*CONFX(I)**2*CFA(I)
10    CONTINUE
      DO 20 J=1,NSTOT
      ESTL=EIS(J)
      CALL FS(ESTL,SS,ETS,ISEG)
      IF(ICON.EQ.0) CALL FC(ESTL,SSS,ESUB)
      IF(ICON.EQ.1) CALL FCON(ESTL,SSS,ESUB)
      EEFF=ETS-ESUB
      DA11=DA11+EEFF*STLA(J)
      DA12=DA12+EEFF*STLY(J)*STLA(J)*SIG
      DA13=DA13+EEFF*STLX(J)*STLA(J)
      DA21=DA21+EEFF*STLY(J)*STLA(J)*SIG
      DA22=DA22+EEFF*STLY(J)**2*STLA(J)
      DA23=DA23+EEFF*STLX(J)*STLY(J)*STLA(J)*SIG
      DA31=DA31+EEFF*STLX(J)*STLA(J)
      DA32=DA32+EEFF*STLX(J)*STLY(J)*STLA(J)*SIG
      DA33=DA33+EEFF*STLX(J)**2*STLA(J)
20    CONTINUE
      RETURN
C**   THIS PROGRAM VALID ON FTN4 AND FTN5 **
      END
C*F45V1P0*
      SUBROUTINE SOLSCR(CONFZ,CONFX,CFA,NFIB)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      DIMENSION CONFZ(200),CONFX(200),CFA(200)
      COMMON /CNE/ ZI,TF,XI,TW
      COMMON /SUP/ IPRNT
      COMMON /SAME/ KCTYPE,KSTYPE,KFT,FAX,NX,NY,KV,KH,NZ
      IF(KCTYPE.GT.0) GO TO 25
      FACTOR=SQRT(200./(XI/ZI))
      KV=FACTOR
      KH=XI/ZI*KV
25    NFIB=KV*KH
      IF(NFIB.GT.200) STOP 555
      HINC=ZI/KV
      VINC=XI/KH
      HSTRT=-ZI/2.0+HINC/2.0
      VSTRT=XI/2.0-VINC/2.0
      KC=1
      V=VSTRT
      DO 10 I=1,KH
      H=HSTRT
      DO 20 J=1,KV
      CONFZ(KC)=H
      CONFX(KC)=V
      CFA(KC)=HINC*VINC
      H=H+HINC
20    KC=KC+1
10    V=V-VINC
      IF(IPRNT.EQ.0) GO TO 115
      PRINT 100
100   FORMAT(///,10X,'CONCRETE FIBER PROPERTIES',
     1//,3X,'NO.',3X,'Z',12X,'X',13X,'AREA')
      DO 110 M=1,NFIB
      PRINT 102,M,CONFZ(M),CONFX(M),CFA(M)
102   FORMAT(I5,3(3X,F10.2))
110   CONTINUE
115   CONTINUE
      KCTYPE=KCTYPE+1
      RETURN
```

```
      END
C*FASVIRC*
      SUBROUTINE OSCRIB(CONPI,CONPX,CPA,NPIS)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      DIMENSION CONPI(200),CONPX(200),CPA(200)
      COMMON /ONE/ ZI,TP,XI,TW
      COMMON /SUP/ IPRNT
      COMMON /SAME/ KCTYPE,KSTTPE,KPT,KMX,NK,NY,KV,KM,NZ
      SI=ZI-XI
      IF(SI.LE.0.) GO TO 15
      UI=XP
      VI=ZI
      GO TO 35
   15 UI=ZI
      VI=XI
   35 SI=ABS(SI)
      IF(SI.LE.1.0) GO TO 53
      IF(KCTYPE.GT.0) GO TO 25
      FACTOR=SQRT(125./(UI/SI))
      KV=FACTOR
      KM=UI/SI*KV
   25 NPIS=KV*KM
      HINC=SI/KV
      VINC=UI/KM
      HSTRT=-SI/2.0+HINC/2.0
      VSTRT=UI/2.0-VINC/2.0
      KC=1
      V=VSTRT
      DO 10 I=1,KM
      H=HSTRT
      DO 20 J=1,KV
      CONPI(KC)=H
      CONPX(KC)=V
      CPA(KC)=HINC*VINC
      H=H+HINC
   20 KC=KC+1
   10 V=V-VINC
      RADIUS=UI/2.0
      PI=3.141592?
      H=1.0/5.0*RADIUS
      ANG=75.0*PI/180.
      C1=.5*30.*PI/180.*H
      ALPHA=15.*PI/180.
      CON=2.*SIN(ALPHA)/(3.*ALPHA)
      DO 30 I=1,3
      R1=RADIUS
      R2=3./4.*RADIUS
      DO 40 J=1,4
      A=C1*(R1+R2)
      ONE=CON*R1*ALPHA*R1**2
      TWO=CON*R2*ALPHA*R2**2
      RAD=(ONE-TWO)/A
      Z=VI/2.0-RADIUS+RAD*COS(ANG)
      X=RAD*SIN(ANG)
      CONPI(KC)=Z
      CONPX(KC)=X
      CPA(KC)=A
      KC=KC+1
      CONPI(KC)=-Z
      CONPX(KC)=X
      CPA(KC)=A
      KC=KC+1
      CONPI(KC)=-Z
      CONPX(KC)=-X
      CPA(KC)=A
      KC=KC+1
      CONPI(KC)=Z
      CPA(KC)=A
      KC=KC+1
      CONPI(KC)=Z
      COMPX(KC)=-X
      CPA(KC)=A
      KC=KC+1
      CONPI(KC)=Z
      CONPX(KC)=-X
      CPA(KC)=A
      KC=KC+1
      R1=R1-H
   40 R2=R2-H
   30 ANG=ANG-30.*PI/180.
      NPIS=NPIS+72
      GO TO 90
   50 KC=1
      RADIUS=UI/2.0
      PI=3.141592?
      H=1.0/5.0*RADIUS
      ANG=82.5*PI/180.
      C1=.5*15.*PI/180.*H
      ALPHA=7.5*PI/180.
      CON=2.*SIN(ALPHA)/(3.*ALPHA)
      DO 70 I=1,6
      R1=RADIUS
      R2=7./8.*RADIUS
      DO 60 J=1,5
      A=C1*(R1+R2)
      ONE=CON*R1*ALPHA*R1**2
      TWO=CON*R2*ALPHA*R2**2
      RAD=(ONE-TWO)/A
      Z=RAD*COS(ANG)
      X=RAD*SIN(ANG)
      CONPI(KC)=Z
      CONPX(KC)=X
      CPA(KC)=A
      KC=KC+1
      CONPI(KC)=-Z
      CONPX(KC)=X
      CPA(KC)=A
      KC=KC+1
      CONPI(KC)=-Z
      CONPX(KC)=-X
      CPA(KC)=A
      KC=KC+1
      CONPI(KC)=Z
      CONPX(KC)=-X
      CPA(KC)=A
      KC=KC+1
      R1=R1-H
   60 R2=R2-H
   70 ANG=ANG-15.*PI/180.
      NPIS=120
   90 IF(NPIS.GT.200) STOP 999
      IF(IPRNT.EQ.0) GO TO 113
      PRINT 100
  100 FORMAT(///,10X,'CONCRETE FIBER PROPERTIES',
     1//,3X,'NO.',5X,'Z',12X,'X',12X,'AREA')
      DO 110 M=1,NPIS
      PRINT 102,M,CONPI(M),CONPX(M),CPA(M)
  102 FORMAT(I5,3E13.4,F10.3))
  110 CONTINUE
  113 CONTINUE
      KCTYPE=KCTYPE+1
      RETURN
      END
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C*F*SV1PO*
      SUBROUTINE DSSTL(STLZ,STLX,STLA,NSTOT)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      DIMENSION STLZ(200),STLX(200),STLA(200)
      COMMON /ONE/ ZI,TF,XI,TW
      COMMON /FOUR/ STT(3),STS(3),GAMMAZ(3),GAMMAX(3),WLSTL(5),NRING
      COMMON /SUP/ IPRNT
      COMMON /SAME/ KCTYPE,KSTYPE,KFY,KWX,NX,NY,KV,KH,NZ
      KC=1
      IF(KSTYPE.GT.0) GO TO 25
      VAL=200.-24.*NRING
      PI=3.1415927
      ZZ=(VAL/NRING)/2.0
      NZ=ZZ
 25   NSTOT=(2*NZ)*NRING+24*NRING
      DO 10 I=1,NRING
      GZ=ZI-XI
      RAD=XI/2.0-GAMMAZ(I)
      GCIR=PI*RAD
      WT=STT(I)/GZ
      WCIR=STS(I)/GCIR
      ZINC=GZ/NZ
      CIRINC=GCIR/12.0
      STRTZ=-ZI/2.0+XI/2.0+ZINC/2.0
      STRTX=XI/2.0-GAMMAX(I)
      Z=STRTZ
      X=STRTX
      DO 20 J=1,NZ
      STLZ(KC)=Z
      STLX(KC)=X
      STLA(KC)=ZINC*WT
      KC=KC+1
      STLZ(KC)=Z
      STLX(KC)=-X
      STLA(KC)=ZINC*WT
      Z=Z+ZINC
 20   KC=KC+1
      ANG=82.5*PI/180.
      DO 30 K=1,6
      Z=ZZ/2.0-XI/2.0+RAD*COS(ANG)
      X=RAD*SIN(ANG)
      A=CIRINC*WCIR
      STLZ(KC)=Z
      STLX(KC)=X
      STLA(KC)=A
      KC=KC+1
      STLZ(KC)=-Z
      STLX(KC)=X
      STLA(KC)=A
      KC=KC+1
      STLZ(KC)=-Z
      STLX(KC)=-X
      STLA(KC)=A
      KC=KC+1
      STLZ(KC)=Z
      STLX(KC)=-X
      STLA(KC)=A
      KC=KC+1
 30   ANG=ANG-15.*PI/180.
 10   CONTINUE
      IF(IPRNT.EQ.0) GO TO 111
      PRINT 100
 100  FORMAT(///,10X,'STEEL FIBER PROPERTIES'
     1//,3X,'NO.',5X,'Z',13X,'Y',10X,'AREA')
      PRINT 110,(K,STLZ(K),STLX(K),STLA(K),K=1,NSTOT)
 110  FORMAT(1X,I4,3X,F10.2,4X,F10.2,3X,F10.2)
 111  CONTINUE
      KSTYPE=KSTYPE+1
      RETURN
      END
C*F*SV1PO*
      SUBROUTINE CIRSCF(CONFX,CONFY,CFA,NFIB)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      DIMENSION CONFX(200),CONFY(200),CFA(200)
      COMMON /ONE/ XI,TF,YI,XF
      KC=1
      PI=3.14156
      ANG=9.0*PI/180.0
      RADIUS=XI/2.0
      H=TF/5.0
      STANG=ANG/2.0
      R1=RADIUS
      R2=RADIUS-H
      DO 10 I=1,5
      AREA=0.5*ANG*H*(R1+R2)
      C1=2.0/3.0*SIN(STANG)/STANG
      C2=R1**3-R2**3
      C3=R1**2-R2**2
      RCEN=C1*C2/C3
      TANG=STANG
      DO 20 J=1,10
      X=RCEN*COS(TANG)
      Y=RCEN*SIN(TANG)
      CFA(KC)=AREA
      CONFX(KC)=X
      CONFY(KC)=Y
      KC=KC+1
      CFA(KC)=AREA
      CONFX(KC)=X
      CONFY(KC)=-Y
      KC=KC+1
      CFA(KC)=AREA
      CONFX(KC)=-X
      CONFY(KC)=Y
      KC=KC+1
      CFA(KC)=AREA
      CONFX(KC)=-X
      CONFY(KC)=-Y
      KC=KC+1
      TANG=TANG+ANG
 20   CONTINUE
      R1=R1-H
      R2=R2-H
 10   CONTINUE
      NFIB=KC-1
      IF(NFIB.GT.200) STOP 510
      PRINT 100
 100  FORMAT(///,10X,'CONCRETE FIBER PROPERTIES',
     1//,3X,'NO.',5X,'X',12X,'Y',10X,'AREA')
      DO 110 M=1,NFIB
      PRINT 102,M,CONFX(M),CONFY(M),CFA(M)
 102  FORMAT(I5,3(3X,F10.3))
 110  CONTINUE
      RETURN
      END
C*F*SV1PO*
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      SUBROUTINE STRSTL(STLX,STLY,STLA,NSTOT)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      DIMENSION STLX(200),STLY(200),STLA(200)
      COMMON /CHE/ AI,TP,YI,TW
      COMMON /FOLR/ STT(3),STS(3),GAMMAX(3),GAMMAY(3),WLSTL(5),NRING
      PI=3.14156
      VAL=200.0/NRING
      KVAL=VAL
      KVAL=KVAL/4
      ANG=(PI/2.0)/KVAL
      RADIUS=AI/2.0
      STANG=ANG/2.0
      KC=1
      DO 10 I=1,NRING
      R=RADIUS-GAMMAX(I)
      ST=0.5*STS(I)
      AREA=ST/KVAL
      RCEN=R*SIN(STANG)/(STANG)
      TANG=STANG
      DO 20 J=1,KVAL
      X=RCEN*COS(TANG)
      Y=RCEN*SIN(TANG)
      STLA(KC)=AREA
      STLX(KC)=X
      STLY(KC)=Y
      KC=KC+1
      STLA(KC)=AREA
      STLX(KC)=X
      STLY(KC)=-Y
      KC=KC+1
      STLA(KC)=AREA
      STLX(KC)=-X
      STLY(KC)=Y
      KC=KC+1
      STLA(KC)=AREA
      STLX(KC)=-X
      STLY(KC)=-Y
      KC=KC+1
      TANG=TANG+ANG
   20 CONTINUE
   10 CONTINUE
      NSTOT=KC-1
      IF(NSTOT.GT.200) STOP 570
      PRINT 100
  100 FORMAT(///,10X,'STEEL FIBER PROPERTIES',
     1//,3X,'NO.',6X,'X',13X,'Y',10X,'AREA')
      PRINT 110,(K,STLX(K),STLY(K),STLA(K),K=1,NSTOT)
  110 FORMAT(1X,I4,3X,F10.4,2X,F10.4,3X,F10.7)
      RETURN
      END
C*FWSV1PO*
      SUBROUTINE FSUB(STLX,STLY,STLA,NSTOT)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      DIMENSION STLX(200),STLY(200),STLA(200)
      READ (*,100,END=10000)NSTOT
  100 FORMAT(3X,I10)
10000 PRINT 150,NSTOT
  150 FORMAT(///,10X,'NUMBER OF READ IN STEEL FIBERS',I5)
      PRINT 200
  200 FORMAT(///,10X,'STEEL FIBER PROPERTIES',
     1//,3X,'NO.',6X,'X',13X,'Y',10X,'AREA')
      READ (*,300,END=10001)(STLX(I),STLY(I),STLA(I),I=1,NSTOT)
  300 FORMAT(3D20.10)
10001 PRINT 110,(K,STLX(K),STLY(K),STLA(K),K=1,NSTOT)
  110 FORMAT(1X,I4,3X,F10.4,2X,F10.4,3X,F10.7)
      RETURN
      END
C*FW5V1PO*
      SUBROUTINE LOSIN(NLPOS,ILP,NDF,NLOAD,JFRNT,LOP,NMEM)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      CHARACTER*4,A(14,13),ASHTL(10)
      DIMENSION FOF(6),MLOAD(66,15,14),FFAC(10,14),GAMMA(10),
     1          FLOAD(72,14),LGI(10),FLCAD(72,14)
      COMMON/DISNO/DUMMF(66,15),DUMFF(72)
      COMMON /C1/ ASHTO
      COMMON /D2/GAMMA,FAC,IERR
      DATA (A(1,J),J=1,13)/'DEAD',' LOA','D (P','ETA ','D SG',
     1     'UAL ','1.0)','00*','    '/
      DATA (A(2,J),J=1,13)/'DEAD',' LOA','D (U','ETA ','D EQ',
     1     'UAL ','0.75)','00*','    '/
      DATA (A(3,J),J=1,13)/'LIVE',' LOA','D AN','D OP',' LIV',
     1     'E LO','AD I','MPAC','T   ','00*','    '/
      DATA (A(4,J),J=1,13)/'CENT','RIFU','GAL ','FORC','E   ',
     1     '0*','    '/
      DATA (A(5,J),J=1,13)/'EART','H PR','ESSU','RE  ','00*','    ',
     1(A(6,J),J=1,13)/'EART','H PR','ESSU','RE(F','RAME','S)  ',
     2     '0*','    ',(A(7,J),J=1,13)/'BUOY','ANCY','11*','    '/
      DATA (A(8,J),J=1,13)/'STRE','AM F','LOW ','PRES','SURE','8*','    ',
     1     (A(9,J),J=1,13)/'WIND',' LOA','D ON',' STR','UCTU','RE  ',
     2     '9*','    '/
      DATA (A(10,J),J=1,13)/'WIND',' LOA','D ON',' LIV','E LO','AD  ',
     1     '0*','    ',(A(11,J),J=1,13)/'LONG','ITUD','INAL',' FOR',
     2     'CE F','ROM ','LIVE',' LOA','D   ','00*','    '/
      DATA (A(12,J),J=1,13)/'RIB ','SHOR','TENI','NG A','ND-S','R SH',
     1     'RINK','AGE ','AND-','OR T','EMPE','RATU','RE  '/
      DATA (A(13,J),J=1,13)/'EART','HQUA','KE  ','12*','    '/,
     1     (A(14,J),J=1,13)/'ICE ','PRES','SURE','13*','    '/
      IF(ILP.GT.1) GO TO 50
      DO 10 K=1,66
      DO 10 J=1,NMEM
      DO 10 L=1,14
   10 MLOAD(K,J,L)=0.0
      DO 15 K=1,NDF
      DO 15 L=1,14
   15 FLOAD(K,L)=0.0
      READ (*,1000,END=10000)JFRNT,NLTYP,(LGI(I),I=1,NLPOS)
10000 PRINT 2000,NLPOS
      DO 20 I=1,NLPOS
      I1=LGI(I)
   20 PRINT 2005,ASHTL(I1)
      PRINT 2010,NLTYP
      DO 70 I=1,NLTYP
      READ (*,1000,END=10001)LTI,NML
10001 IPRS=1
      IF(LTI.EQ.2)IPRS=2
      PRINT 2020,I,(A(LTI,J),J=1,13),NML
      IF(NML.EQ.0) GO TO 50
      DO 40 NMJ=1,NML
      READ (*,1000,END=10002)IM,NMJL
10002 PRINT 2030,IM,NMJL
      PRINT 2033
      DO 40 NJL=1,NMJL
      READ (*,1000,END=10003)J,(TIP(L),L=1,6)
10003 PRINT 2040,J,(FOR(L),L=1,6)
            JJ=6*J-6
      DO 30 L=1,6
```

```
      JJ=JJ+1
   30 MLOAD(JJ,IP,LTI)=FOR(L)
   40 CONTINUE
   50 READ (*,1000,END=10004)NFJL
10004 PRINT 2050,(A(LTI,J),J=1,10),NFJL
      IF(NFJL.EQ.0) GO TO 70
      PRINT 2035
      DO 60 NF=1,NFJL
      READ (*,1020,END=10005)J,(FOR(L),L=1,6)
10005 PRINT 2040,J,(FOR(L),L=1,6)
      JJ=6*J-6
      DO 60 L=1,6
      JJ=JJ+1
   60 PLOAD(JJ,LTI)=FOR(L)
   70 CONTINUE
   80 LGP=LGI(ILP)
      DO 90 K=1,65
      DO 90 J=1,NMEM
      CUMMF(K,J)=0.0
      DO 90 L=1,16
      FORLS(K,J,L)=MLOAD(K,J,L)*BFAC(LGP,L)*GAMMA(LGP)
   90 CUMMF(K,J)=CUMMF(K,J)+FORLS(K,J,L)
      DO 100 K=1,NOP
      CUMFF(K)=0.0
      DO 100 J=1,16
      FORLG(K,1,J)=PLLAD(K,J)*BFAC(LGP,J)*GAMMA(LGP)
  100 CUMFF(K)=CUMFF(K)+FORLG(K,1,J)
      DO 110 K=1,65
      DO 110 J=1,NMEM
  110 CUMMF(K,J)=0.5*CUMMF(K,J)/50.
      DO 120 K=1,NOP
  120 CUMFF(K)=CUMFF(K)*0.5/50.
      NLOAD=3
 1000 FORMAT(2X,I10)
 1020 FORMAT(2X,I10,6F10.0)
 2030 FORMAT('1',4X,'THE FOLLOWING',I3,' AASHTO LOAD GROUPS WILL BE',
     1       ' CONSIDERED :',///)
 2005 FORMAT('0',10X,/,'LOAD GROUP',A4)
 2010 FORMAT('0',4X,//,'THERE ARE',I3,' DIFFERENT LOAD TYPES:',///)
 2020 FORMAT('0',//,20X,'  **********  ','LOAD TYPE',I3,'     ******',
     1'****',//,4X,' MEMBERS LOADED WITH ',13A4,' =',I3,/////)
 2030 FORMAT('0',10X,'MEMBER =',I3,//,11X,'NUMBER OF JOINTS LOADED =',
     .        I3,//)
 2035 FORMAT('0',20X,'*** INPUT UNFACTORED LOAD VALUES ***',//,
     .        4X,'JOINT NO.',4X,'FORCE-X',4X,'FORCE-Y',4X,'FORCE-Z',
     .        4X,'MOMENT-X',4X,'MOMENT-Y',4X,'MOMENT-Z',//)
 2040 FORMAT('0',4X,I3,4X,3(1X,1PE10.3),3(2X,1PE10.3))
 2050 FORMAT('1',4X,' FRAME JOINTS LOADED WITH ',13A4,' =',I3,/////)
      RETURN
      END
C*PASV1PG*
      SUBROUTINE FAIL(LGP,JAY,IL,IFAIL,ISECT,ISEG,MI)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      CHARACTER*4 ASHTO(10)
      COMMON/C1/ ASHTO
      COMMON/F1/ BIGC,MLOAD,MINC,MSEG,MSECT
      COMMON/F2/ BIGS,JLOAD,JINC,JSEG,JSECT
 2000 FORMAT('1',5X,'THE STIFFNESS MATRIX HAS BECOME NEGATIVE',
     . ', NO FURTHER CALCULATIONS POSSIBLE',/,5X,
     .         '*****************************************')
 2010 FORMAT('1',4X,'THE PIER-BENT HAD A CONCRETE COMPRESSION FAILURE',/
     .         '*******************************************')
 2020 FORMAT('1',4X,'THE PIER-BENT HAD A STEEL TENSION FAILURE',/,5X,
     .         '*************************************')
 2030 FORMAT('0',5X,'IN MEMBER',I5,/,5X,'IN SECTION',I5,/,5X,
     .         'IN SEGMENT',I5,/)
 2040 FORMAT('0',5X,'IN LOAD INCREMENT',I5,/,5X,'OF LOAD CASE',I5)
 2050 FORMAT('0',///,10X,'JUST BEFORE FAILURE',/,
     . 10X,'********************',///,
     1        5X,'MAX. COMPRESSIVE CONCRETE STRAIN=',1PE10.3,/,
     2        5X,'OCCURED IN LOAD CASE            =',I5,/,
     3             5X,'OCCURED IN INCREMENT            =',I5,/,
     .             5X,'OCCURED IN MEMBER               =',I5,/,
     4             5X,'OCCURED IN SEGMENT              =',I5,/,
     5             5X,'OCCURED IN SECTION              =',I5,/)
 2060 FORMAT('0',5X,'MAX. TENSILE STEEL STRAIN=',1PE10.3,/,
     1             5X,'OCCURED IN LOAD CASE    =',I5,/,
     C             5X,'OCCURED IN INCREMENT    =',I5,/,
     .             5X,'OCCURED IN MEMBER       =',I5,/,
     2             5X,'OCCURED IN SEGMENT      =',I5,/,
     3             5X,'OCCURED IN  SECTION     =',I5)
 2070 FORMAT ('0',2X,'AT ',F6.2,' PERCENT OF THE REQUIRED ULTIMATE',
     1        2X,' LOAD CORRESPONDING TO AASHTO LOAD GROUP',A4)
 2080 FORMAT('0',///,2X,'THE PIER-BENT HAS BEEN GROSSLY UNDERDESIGNED',/)
 2082 FORMAT('0',///2X,'THE PIER-BENT HAS BEEN SIGNIFICANTLY',
     . 'UNDERDESIGNED',/)
 2085 FORMAT ('0',///,2X,'THE PIER-BENT HAS BEEN UNDERDESIGNED',/)
 2090 FORMAT('0',//,2X,'THE PIER-BENT HAS BEEN ACEPTABLY',
     . ' DESIGNED',/)
 2110 FORMAT('0',//,2X,'THE PIER-BENT HAS BEEN OVERDESIGNED',/)
 2112 FORMAT('0',//,2X,'THE PIER-BENT HAS BEEN SIGNIFICANTLY',
     . 'OVERDESIGNED',/)
 2115 FORMAT('0',//,2X,'THE PIER-BENT HAS BEEN GROSSLY OVERDESIGNED',/)
 2120 FORMAT('0',2X,'FOR LOAD GROUP',A4)
      PER=-1.0
      IF(LGP.LE.0) GO TO 50
      KJAY=JAY-2
      IF(KJAY) 20,30,40
   20 PER=IL*2/50.
      GO TO 50
   30 PER=80.+IL*.2/50.
      GO TO 50
   40 PER=100.+IL*.8/50.
   50 IF(IFAIL) 60,70,80
   60 PRINT 2000
      PRINT 2030,MI,ISECT,ISEG
      IF(LGP.LE.0) PRINT 2040,IL,JAY
      IF(LGP.GT.0) PRINT 2070,PER,ASHTO(LGP)
      GO TO 100
   70 PRINT 2000
      IF(LGP.LE.0) PRINT 2040,IL,JAY
      IF(LGP.GT.0) PRINT 2070,PER,ASHTO(LGP)
      GO TO 100
   80 PRINT 2010
      PRINT 2030,MI,ISECT,ISEG
      IF(LGP.LE.0) PRINT 2040,IL,JAY
      IF(LGP.GT.0) PRINT 2070,PER,ASHTO(LGP)
  100 PRINT 2050,BIGC,MLOAD,MINC,MI,MSEG,MSECT
      PRINT 2060,BIGS,JLOAD,JINC,MI,JSEG,JSECT
      IF(PER.LT.0) GO TO 110
      IF(PER.GE.0.AND.PER.LT.70.) PRINT 2080
      IF(PER.GE.70..AND.PER.LT.90.) PRINT 2082
      IF(PER.GE.90..AND.PER.LT.95.)PRINT 2085
      IF(PER.GE.95..AND.PER.LE.105.)PRINT 2090
      IF(PER.GT.105..AND.PER.LE.115.) PRINT 2110
      IF(PER.GT.115..AND.PER.LE.130.) PRINT 2112
```

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      IF(PSN.GT.15C.) PRINT 2113
      PRINT 2112,ASHTO(LGP)
  110 RETURN
      END
C*P45V1P0*
      SUBROUTINE FS(EP,SS,ETS,ISEG)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      COMMON /L2/ FY,EY,RATIO(10)
      EP=-EP
C     CALCULATION OF STEEL STRESS AND TANGENT STIFFNESS
      IF(EP.LT.-0.00010) GO TO 4
      SS=EY*EP
      ETS=EY
      IF(SS.GT.FY) SS=FY
      IF(SS.GE.FY) ETS=0.0
      GO TO 10
 4    A=ABS(EP/.0001)
      ADD=EY/535770./RATIO(ISEG)
      ADK=EY/2544.
      IF(RATIO(ISEG).LT..005) ADD=ADK+(ADD-ADK)*(RATIO(ISEG)/.005)
      SS=EP*EY-ADD*(1.0-1./A)
      ETS=EY-0.0001*ADD/(EP**2)
      IF(SS.LT.-FY) SS=-FY
      IF(SS.LE.-FY) ETS=0.0
 10   EP=-EP
      SS=-SS
      RETURN
C**  THIS PROGRAM VALID ON FTN4 AND FTN5 **
      END
C*P45V1P0*
      SUBROUTINE FC(EP,STRESS,ETC)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      COMMON /L1/ CYL,RK,EC,ENO,EU
C     CONCRETE STRESS AND TANGENT STIFFNESS USING HOGNESTAD STRESS BLOCK
      EUHOG=1.7*ENO
      EP=-EP
      IF(EP) 2,3,4
    2 SC=0.
      ETAN=EC/10.**10
      GO TO 1
 3    SC=0.0
      ETAN=EC
      GO TO 1
 4    SC=RK*CYL*(2.0*EP/ENO-(EP/ENO)**2)
      ETAN=RK*CYL*(2.0/ENO-2.0*(EP/ENO)*(1.0/ENO))
      IF(EP.GT.ENO) SC=RK*CYL*(1.0-.15*(EP-ENO)/(EUHOG-ENO))
      IF(EP.GT.ENO) ETAN=-.15*RK*CYL/(EUHOG-ENO)
 1    STRESS=-SC
      ETC=ETAN
      EP=-EP
      RETURN
C**  THIS PROGRAM VALID ON FTN4 AND FTN5 **
      END
C*P45V1P0*
      SUBROUTINE FCON(EP,STRESS,ETC)
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      COMMON /L1/ CYL,RK,EC,ENO,EU
C      CONCRETE STRESS AND TANGENT STIFFNESS FOR CONFINED
C         CONCRETE USING MODL STRESS-STRAIN CURVE
      EP=-EP
      IF(EP) 2,3,4
    2 SC=0.
      ETAN=EC/10.**10
      GO TO 1
 3    SC=0.0
      ETAN=EC
      GO TO 1
 4    SC=RK*CYL*(2.0*EP/ENO-(EP/ENO)**2)
      ETAN=RK*CYL*(2.0/ENO-2.0*(EP/ENO)*(1.0/ENO))
      IF(EP.GT.ENO) SC=RK*CYL*(1.0-20.0*(EP-ENO))
      IF(EP.GT.ENO) ETAN=-20.0*RK*CYL
 1    STRESS=-SC
      ETC=ETAN
      EP=-EP
      RETURN
C**  THIS PROGRAM VALID ON FTN4 AND FTN5 **
      END
      BLOCK DATA DLOAD
      IMPLICIT DOUBLE PRECISION(A-H,O-Z)
      CHARACTER*4 ASHTO(10)
      DIMENSION GAMMA(10),BFAC(10,14)
      COMMON /D2/GAMMA,BFAC,IPRE
      COMMON /D1/ ASHTO
      DATA (BFAC(I,1),I=1,10)/10*1.0/,(BFAC(I,2),I=1,10)/10*.75/
      DATA (BFAC(I,3),I=1,10)/1.67,2.2,0.,2*1.0,0.,1.,0.,1.,0./
      DATA (BFAC(I,4),I=1,10)/1.,2*0.,2*1.,0.,1.,0.,1.,0./
      DATA (BFAC(I,5),I=1,10)/1.3,0.,3*1.3/,(BFAC(I,6),I=1,10)/.5,0.,
     1      6*.5/
      DATA (BFAC(I,7),I=1,10)/1.,0.,8*1./,(BFAC(I,8),I=1,10)/1.,0.,8*1./
      DATA (BFAC(I,9),I=1,10)/2*0.,1.,.3,0.,1.,.3,2*0.,1./
      DATA (BFAC(I,10),I=1,10)/3*0.,1.,2*0.,1.,3*0./,
     1      (BFAC(I,11),I=1,10)/3*0.,1.,2*0.,1.,3*0./,
     2      (BFAC(I,12),I=1,10)/6*0.,3*1.,3*0./,
     3      (BFAC(I,13),I=1,10)/7*0.,1.,2*0./,
     4      (BFAC(I,14),I=1,10)/8*0.,2*1./
      DATA (GAMMA(I),I=1,10)/5*1.3,2*1.25,2*1.3,1.2/
      DATA ASHTO/'  I ',' IA ',' II ',' III',' IV ','  V ',' VI ',
     1           ' VII','VIII',' IX '/
      END
      SUBROUTINE GOTCER
      STOP "ERROR CONDITION (CALL TO GOTCER) STOPPED EXECUTION"
      END
```

C. 6 Example Problem

The following are the input data and the data output for the example frame model by Repa.

C. 6.1 Input Data for Example Problem

CARD		DATA FILE : REPALG							PAGE 1 of 2
N°	TYPE	COL 1	10	20	30	40	50	60	70
1	1	FRAME MODEL BY REPA							
2	2	3	1						
3	3	25.71							
4	4	31.5	31.5						
5	5	3	0						
6	6	1	111111						
7	6	2	111111						
8	6	3	111111						
9	8	4430.	37000.	0.85	3700000.	29000000.	0.004		
10	9	10	2						
11	10	2.46 2.46	2.60 2.60	2.60 2.60	2.60 2.60	2.60 2.60			
12	11	OVAL	0	1	1				
13	12	1	2						
14	13.b	4.39	4.38						
15	14.b.1	8							
16	14.b.2	1.86	0.0	0.040	-1.315	1.315	0.040		
17	14.b.2	-1.86	0.0	0.040	-1.315	-1.315	0.040		
18	14.b.2	0.0	1.86	0.040	1.315	-1.315	0.040		
19	14.b.2	0.0	-1.86	0.040	1.315	1.315	0.040		
20	12	3	10						
21	13.b	4.39	4.38						
22	14.b.1	8							
23	14.b.2	1.86	0.0	0.020	-1.315	1.315	0.020		
24	14.b.2	-1.86	0.0	0.020	-1.315	-1.315	0.020		
25	14.b.2	0.0	1.86	0.020	1.315	-1.315	0.020		
26	14.b.2	0.0	-1.86	0.020	1.315	1.315	0.020		
27	9	10	2						
28	10	2.46 2.46	2.60 2.60	2.60 2.60	2.60 2.60	2.60 2.60			
29	11	OVAL	0	1	1				
30	12	1	2						
31	13.b	4.39	4.38						
32	14.b.1	8							
33	14.b.2	1.86	0.0	0.040	-1.315	1.315	0.040		
34	14.b.2	-1.86	0.0	0.040	-1.315	-1.315	0.040		
35	14.b.2	0.0	1.86	0.040	1.315	-1.315	0.040		
36	14.b.2	0.0	-1.86	0.040	1.315	1.315	0.040		
37	12	3	10						
38	13.b	4.39	4.38						
39	14.b.1	8							
40	14.b.2	1.86	0.0	0.020	-1.315	1.315	0.020		
41	14.b.2	-1.86	0.0	0.020	-1.315	-1.315	0.020		
42	14.b.2	0.0	1.86	0.020	1.315	-1.315	0.020		
43	14.b.2	0.0	-1.86	0.020	1.315	1.315	0.020		
44	9	10	5						
45	10	4.36 2.64	2.81 4.10	1.75 2.25	3.00 5.05	1.05 3.5			
46	11	SOLID	0	1	1				
47	12	1	1						
48	13.a	5.60	4.38						
49	14.b.1	4							
50	14.b.2	2.13	-1.86	0.033	2.13	1.86	0.033		
51	14.b.2	-2.13	-1.86	0.054	-2.13	1.86	0.054		
52	12	2	3						
53	13.a	5.60	4.38						
54	14.b.1	4							
55	14.b.2	2.13	-1.86	0.043	2.13	1.86	0.043		
56	14.b.2	-2.13	-1.86	0.054	-2.13	1.86	0.054		
57	12	4	7						
58	13.a	5.60	4.38						
59	14.b.1	4							
60	14.b.2	2.13	-1.86	0.043	2.13	1.86	0.043		
61	14.b.2	-2.13	-1.86	0.033	-2.13	1.86	0.033		

DATA FILE: REPALG (CONT)

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CARD No	CARD TYPE	COL 1	11	21	31	41	51	61
62	12	0	0					
63	13A	5.60	4.38					
64	14.b.1	4						
65	14.b.2	2.13	-1.86	0.043	2.13	1.86	0.043	
67	14.b.2	-2.13	-1.86	0.103	-2.13	1.86	0.103	
68	12	0	10					
69	13.a	5.60	4.38					
70	14.b.1	4						
71	14.b.2	2.13	-1.86	0.033	2.13	1.86	0.033	
72	14.b.2	-2.13	-1.86	0.103	-2.13	1.86	0.103	
73	15	0						
74	16.a.1	3						
75	16.a.2	30	2	30				
76	16.a.3	4	4					
77	16.a.4	3		-34.6				
78	16.a.4	5		-34.6				
79	16.a.4	8		-34.6				
80	16.a.4	10		-34.6				
81	16.a.3	5	4					
82	16.a.4	3		-34.6				
83	16.a.4	5		-34.6				
84	16.a.4	8		-34.6				
85	16.a.4	10		-34.6				
86	16.a.5	2						
87	16.a.6	4		-69.2				242.30
88	16.a.6	6		-69.2				-242.30
89	16.a.2	90	1	12				
90	16.a.3	4	4					
91	16.a.4	3		-17.5				
92	16.a.4	5		-17.5				
93	16.a.4	8		-17.5				
94	16.a.4	10		-17.5				
95	16.a.5	1						
96	16.a.6	6		-22.5				-78.75
97	16.a.2	80	1	0				
98	16.a.3	4	4					
99	16.a.4	3		-7.00				
100	16.a.4	5		-7.00				
101	16.a.4	8		-7.00				
102	16.a.4	10		-7.00				
103	16.a.5	1						
104	16.a.6	6		-9.00				-31.50

FRAME MODEL BY REPA

NUMBER OF COLUMN LINES= 3
NUMBER OF LEVELS = 1

NUMBER OF JOINTS WITH CONSTRAINED
OR SPECIFIED DISPLACEMENTS = 3
NUMBER OF JOINTS WITH SPRINGS = 0

JOINT NO.	RELEASE CODE
1	111111
2	111111
3	111111

CONCRETE STRENGTH = 4430.00
STEEL YIELD POINT = 37000.00
STRENGTH REDUCTION FACTOR = .85
CONCRETE MODULUS = .3790E+07
STEEL MODULUS = .2900E+08
CONCRETE ULTIMATE STRAIN = .4000E-02
CREEP FACTOR = 0.00

MEMBER 1 LEVEL 1 HAS 10 SEGMENTS AND 2 DIFFERENT SECTIONS

SEGMENT LENGTHS 2.46 2.46 2.60 2.60 2.00
2.00 2.60 2.60 2.00 2.00

THE MEMBER CROSS-SECTION IS OVAL

CONCRETE FIBERS WILL BE MODELLED AS UNCONFINED CONCRETE

STEEL FIBERS ARE READ IN BY THE USER

CROSS-SECTION TYPE 1

Z-DIMENSION = 4.39
X-DIMENSION = 4.38

CONCRETE FIBER PROPERTIES

NO.	Z	X	AREA
1	.27	2.03	.15
2	-.27	2.03	.15
3	-.27	-2.03	.15
4	.27	-2.03	.15
5	.23	1.76	.13
6	-.23	1.76	.13
7	-.23	-1.76	.13
8	.23	-1.76	.13
9	.20	1.49	.11
10	-.20	1.49	.11
11	-.20	-1.49	.11
12	.20	-1.49	.11
13	.16	1.22	.09
14	-.16	1.22	.09
15	-.16	-1.22	.09
16	.16	-1.22	.09
17	.13	.95	.07
18	-.13	.95	.07
19	-.13	-.95	.07
20	.13	-.95	.07
21	.09	.69	.05
22	-.09	.69	.05
23	-.09	-.69	.05
24	.09	-.69	.05
25	.06	.42	.03
26	-.06	.42	.03
27	-.06	-.42	.03
28	.06	-.42	.03
29	.02	.18	.01
30	-.02	.18	.01
31	-.02	-.18	.01
32	.02	-.18	.01
33	.78	1.89	.15
34	-.78	1.89	.15
35	-.78	-1.89	.15
36	.78	-1.89	.15
37	.68	1.64	.13
38	-.68	1.64	.13
39	-.68	-1.64	.13
40	.68	-1.64	.13
41	.58	1.39	.11
42	-.58	1.39	.11
43	-.58	-1.39	.11
44	.58	-1.39	.11
45	.47	1.14	.09
46	-.47	1.14	.09
47	-.47	-1.14	.09
48	.47	-1.14	.09
49	.37	.89	.07
50	-.37	.89	.07
51	-.37	-.89	.07
52	.37	-.89	.07
53	.26	.64	.05
54	-.26	.64	.05
55	-.26	-.64	.05
56	.26	-.64	.05
57	.16	.39	.03
58	-.16	.39	.03
59	-.16	-.39	.03
60	.16	-.39	.03
61	.07	.17	.01
62	-.07	.17	.01
63	-.07	-.17	.01
64	.07	-.17	.01
65	1.25	1.63	.15
66	-1.25	1.63	.15
67	-1.25	-1.63	.15

68	1.25	-1.63	.15
69	1.08	1.41	.13
70	-1.08	1.41	.13
71	-1.08	-1.41	.13
72	1.08	-1.41	.13
73	.92	1.19	.11
74	-.92	1.19	.11
75	-.92	-1.19	.11
76	.92	-1.19	.11
77	.75	.98	.09
78	-.75	.98	.09
79	-.75	-.98	.09
80	.75	-.98	.09
81	.59	.76	.07
82	-.59	.76	.07
83	-.59	-.76	.07
84	.59	-.76	.07
85	.42	.55	.05
86	-.42	.55	.05
87	-.42	-.55	.05
88	.42	-.55	.05
89	.26	.34	.03
90	-.26	.34	.03
91	-.26	-.34	.03
92	.26	-.34	.03
93	.11	.14	.01
94	-.11	.14	.01
95	-.11	-.14	.01
96	.11	-.14	.01
97	1.63	1.25	.15
98	-1.63	1.25	.15
99	-1.63	-1.25	.15
100	1.63	-1.25	.15
101	1.41	1.08	.13
102	-1.41	1.08	.13
103	-1.41	-1.08	.13
104	1.41	-1.08	.13
105	1.19	.92	.11
106	-1.19	.92	.11
107	-1.19	-.92	.11
108	1.19	-.92	.11
109	.98	.75	.09
110	-.98	.75	.09
111	-.98	-.75	.09
112	.98	-.75	.09
113	.76	.59	.07
114	-.76	.59	.07
115	-.76	-.59	.07
116	.76	-.59	.07
117	.55	.42	.05
118	-.55	.42	.05
119	-.55	-.42	.05
120	.55	-.42	.05
121	.34	.26	.03
122	-.34	.26	.03
123	-.34	-.26	.03
124	.34	-.26	.03
125	.14	.11	.01
126	-.14	.11	.01
127	-.14	-.11	.01
128	.14	-.11	.01
129	1.89	.78	.15

130	-1.89	.78	.15
131	-1.89	-.78	.15
132	1.89	-.78	.15
133	1.64	.68	.13
134	-1.64	.68	.13
135	-1.64	-.68	.13
136	1.64	-.68	.13
137	1.39	.58	.11
138	-1.39	.58	.11
139	-1.39	-.58	.11
140	1.39	-.58	.11
141	1.14	.47	.09
142	-1.14	.47	.09
143	-1.14	-.47	.09
144	1.14	-.47	.09
145	.89	.37	.07
146	-.89	.37	.07
147	-.89	-.37	.07
148	.89	-.37	.07
149	.64	.26	.05
150	-.64	.26	.05
151	-.64	-.26	.05
152	.64	-.26	.05
153	.39	.16	.03
154	-.39	.16	.03
155	-.39	-.16	.03
156	.39	-.16	.03
157	.17	.07	.01
158	-.17	.07	.01
159	-.17	-.07	.01
160	.17	-.07	.01
161	2.03	.27	.15
162	-2.03	.27	.15
163	-2.03	-.27	.15
164	2.03	-.27	.15
165	1.76	.23	.13
166	-1.76	.23	.13
167	-1.76	-.23	.13
168	1.76	-.23	.13
169	1.49	.20	.11
170	-1.49	.20	.11
171	-1.49	-.20	.11
172	1.49	-.20	.11
173	1.22	.16	.09
174	-1.22	.16	.09
175	-1.22	-.16	.09
176	1.22	-.16	.09
177	.95	.13	.07
178	-.95	.13	.07
179	-.95	-.13	.07
180	.95	-.13	.07
181	.69	.09	.05
182	-.69	.09	.05
183	-.69	-.09	.05
184	.69	-.09	.05
185	.42	.06	.03
186	-.42	.06	.03
187	-.42	-.06	.03
188	.42	-.06	.03
189	.18	.02	.01
190	-.18	.02	.01
191	-.18	-.02	.01

192 .10 -.02 .01

MOMENT OF INERTIA ABOUT Z-AXIS= .170E+02
MOMENT OF INERTIA ABOUT X-AXIS= .170E+02

NUMBER OF READ IN STEEL FIBERS= 8
4

STEEL FIBER PROPERTIES

NO.	Z	X	AREA
1	1.86	0.00	.04
2	-1.32	1.32	.04
3	-1.86	0.00	.04
4	-1.32	-1.32	.04
5	0.00	1.86	.04
6	1.32	-1.32	.04
7	0.00	-1.86	.04
8	1.32	1.32	.04

CROSS-SECTION TYPE 2

Z-DIMENSION = 4.30
X-DIMENSION = 4.30

CONCRETE FIBER PROPERTIES

NO.	Z	X	AREA
1	.27	2.03	.15
2	-.27	2.03	.15
3	-.27	-2.03	.15
4	.27	-2.03	.15
5	.23	1.76	.13
6	-.23	1.76	.13
7	-.23	-1.76	.13
8	.23	-1.76	.13
9	.20	1.49	.11
10	-.20	1.49	.11
11	-.20	-1.49	.11
12	.20	-1.49	.11
13	.16	1.22	.09
14	-.16	1.22	.09
15	-.16	-1.22	.09
16	.16	-1.22	.09
17	.13	.95	.07
18	-.13	.95	.07
19	-.13	-.95	.07
20	.13	-.95	.07
21	.09	.69	.05
22	-.09	.69	.05
23	-.09	-.69	.05
24	.09	-.69	.05
25	.06	.42	.03
26	-.06	.42	.03
27	-.06	-.42	.03
28	.06	-.42	.03
29	.02	.18	.01
30	-.02	.18	.01
31	-.02	-.18	.01
32	.02	-.18	.01
33	.78	1.89	.15
34	-.78	1.89	.15
35	-.78	-1.89	.15
36	.78	-1.89	.15
37	.68	1.64	.13
38	-.68	1.64	.13
39	-.68	-1.64	.13
40	.68	-1.64	.13
41	.58	1.39	.11
42	-.58	1.39	.11
43	-.58	-1.39	.11
44	.58	-1.39	.11
45	.47	1.14	.09
46	-.47	1.14	.09
47	-.47	-1.14	.09
48	.47	-1.14	.09
49	.37	.89	.07
50	-.37	.89	.07
51	-.37	-.89	.07
52	.37	-.89	.07
53	.26	.64	.05
54	-.26	.64	.05
55	-.26	-.64	.05
56	.26	-.64	.05
57	.16	.39	.03
58	-.16	.39	.03
59	-.16	-.39	.03
60	.16	-.39	.03
61	.07	.17	.01
62	-.07	.17	.01
63	-.07	-.17	.01
64	.07	-.17	.01
65	1.25	1.63	.15
66	-1.25	1.63	.15
67	-1.25	-1.63	.15
68	1.25	-1.63	.15
69	1.08	1.41	.13
70	-1.08	1.41	.13
71	-1.08	-1.41	.13
72	1.08	-1.41	.13
73	.92	1.19	.11
74	-.92	1.19	.11
75	-.92	-1.19	.11
76	.92	-1.19	.11
77	.75	.98	.09
78	-.75	.98	.09
79	-.75	-.98	.09
80	.75	-.98	.09
81	.59	.76	.07
82	-.59	.76	.07
83	-.59	-.76	.07
84	.59	-.76	.07
85	.42	.55	.05
86	-.42	.55	.05
87	-.42	-.55	.05
88	.42	-.55	.05
89	.26	.34	.03
90	-.26	.34	.03

91	-.26	-.34	.03
92	.26	-.34	.03
93	.11	.14	.01
94	-.11	.14	.01
95	-.11	-.14	.01
96	.11	-.14	.01
97	1.63	1.25	.15
98	-1.63	1.25	.15
99	-1.63	-1.25	.15
100	1.63	-1.25	.15
101	1.41	1.00	.13
102	-1.41	1.00	.13
103	-1.41	-1.00	.13
104	1.41	-1.00	.13
105	1.19	.92	.11
106	-1.19	.92	.11
107	-1.19	-.92	.11
108	1.19	-.92	.11
109	.98	.75	.09
110	-.98	.75	.09
111	-.98	-.75	.09
112	.98	-.75	.09
113	.76	.59	.07
114	-.76	.59	.07
115	-.76	-.59	.07
116	.76	-.59	.07
117	.55	.42	.05
118	-.55	.42	.05
119	-.55	-.42	.05
120	.55	-.42	.05
121	.34	.26	.03
122	-.34	.26	.03
123	-.34	-.26	.03
124	.34	-.26	.03
125	.14	.11	.01
126	-.14	.11	.01
127	-.14	-.11	.01
128	.14	-.11	.01
129	1.89	.70	.15
130	-1.89	.70	.15
131	-1.89	-.70	.15
132	1.89	-.70	.15
133	1.64	.60	.13
134	-1.64	.60	.13
135	-1.64	-.60	.13
136	1.64	-.60	.13
137	1.39	.50	.11
138	-1.39	.50	.11
139	-1.39	-.50	.11
140	1.39	-.50	.11
141	1.14	.47	.09
142	-1.14	.47	.09
143	-1.14	-.47	.09
144	1.14	-.47	.09
145	.89	.37	.07
146	-.89	.37	.07
147	-.89	-.37	.07
148	.89	-.37	.07
149	.64	.26	.05
150	-.64	.26	.05
151	-.64	-.26	.05
152	.64	-.26	.05
153	.39	.16	.03
154	-.39	.16	.03
155	-.39	-.16	.03
156	.39	-.16	.03
157	.17	.07	.01
158	-.17	.07	.01
159	-.17	-.07	.01
160	.17	-.07	.01
161	2.03	.27	.15
162	-2.03	.27	.15
163	-2.03	-.27	.15
164	2.03	-.27	.15
165	1.76	.23	.13
166	-1.76	.23	.13
167	-1.76	-.23	.13
168	1.76	-.23	.13
169	1.49	.20	.11
170	-1.49	.20	.11
171	-1.49	-.20	.11
172	1.49	-.20	.11
173	1.22	.16	.09
174	-1.22	.16	.09
175	-1.22	-.16	.09
176	1.22	-.16	.09
177	.95	.13	.07
178	-.95	.13	.07
179	-.95	-.13	.07
180	.95	-.13	.07
181	.69	.09	.05
182	-.69	.09	.05
183	-.69	-.09	.05
184	.69	-.09	.05
185	.42	.06	.03
186	-.42	.06	.03
187	-.42	-.06	.03
188	.42	-.06	.03
189	.18	.02	.01
190	-.18	.02	.01
191	-.18	-.02	.01
192	.18	-.02	.01

MOMENT OF INERTIA ABOUT Z-AXIS= .179E+02
MOMENT OF INERTIA ABOUT X-AXIS= .179E+02

NUMBER OF READ IN STEEL FIBERS= 8

STEEL FIBER PROPERTIES

NO.	Z	X	AREA
1	1.86	0.00	.02
2	-1.32	1.32	.02
3	-1.86	0.00	.02
4	-1.32	-1.32	.02
5	0.00	1.86	.02
6	1.32	-1.32	.02
7	0.00	-1.86	.02
8	1.32	1.32	.02

MEMBER 2 LEVEL 1 HAS 10 SEGMENTS AND 2 DIFFERENT SECTIONS

SEGMENT LENGTHS				
2.46	2.46	2.40	2.40	2.40
2.40	2.40	2.40	2.40	2.40

THE MEMBER CROSS-SECTION IS OVAL

CONCRETE FIBERS WILL BE MODELLED AS UNCONFINED CONCRETE

STEEL FIBERS ARE READ IN BY THE USER

CROSS-SECTION TYPE 1

Z-DIMENSION = 4.30
X-DIMENSION = 4.30

CONCRETE FIBER PROPERTIES

NO.	Z	X	AREA
1	.27	2.03	.15
2	-.27	2.03	.15
3	-.27	-2.03	.15
4	.27	-2.03	.15
5	.23	1.76	.13
6	-.23	1.76	.13
7	-.23	-1.76	.13
8	.23	-1.76	.13
9	.20	1.49	.11
10	-.20	1.49	.11
11	-.20	-1.49	.11
12	.20	-1.49	.11
13	.16	1.22	.09
14	-.16	1.22	.09
15	-.16	-1.22	.09
16	.16	-1.22	.09
17	.13	.95	.07
18	-.13	.95	.07
19	-.13	-.95	.07
20	.13	-.95	.07
21	.09	.69	.05
22	-.09	.69	.05
23	-.09	-.69	.05
24	.09	-.69	.05
25	.06	.42	.03
26	-.06	.42	.03
27	-.06	-.42	.03
28	.06	-.42	.03
29	.02	.18	.01
30	-.02	.18	.01
31	-.02	-.18	.01
32	.02	-.18	.01
33	.78	1.89	.15
34	-.78	1.89	.15
35	-.78	-1.89	.15
36	.78	-1.89	.15
37	.68	1.64	.13
38	-.68	1.64	.13
39	-.68	-1.64	.13
40	.68	-1.64	.13
41	.58	1.39	.11
42	-.58	1.39	.11
43	-.58	-1.39	.11
44	.58	-1.39	.11
45	.47	1.14	.09
46	-.47	1.14	.09
47	-.47	-1.14	.09
48	.47	-1.14	.09
49	.37	.89	.07
50	-.37	.89	.07
51	-.37	-.89	.07
52	.37	-.89	.07
53	.26	.64	.05
54	-.26	.64	.05
55	-.26	-.64	.05
56	.26	-.64	.05
57	.16	.39	.03
58	-.16	.39	.03
59	-.16	-.39	.03
60	.16	-.39	.03
61	.07	.17	.01
62	-.07	.17	.01
63	-.07	-.17	.01
64	.07	-.17	.01
65	1.25	1.63	.15
66	-1.25	1.63	.15
67	-1.25	-1.63	.15
68	1.25	-1.63	.15
69	1.08	1.41	.13
70	-1.08	1.41	.13
71	-1.08	-1.41	.13
72	1.08	-1.41	.13
73	.92	1.19	.11
74	-.92	1.19	.11
75	-.92	-1.19	.11
76	.92	-1.19	.11
77	.75	.98	.09
78	-.75	.98	.09
79	-.75	-.98	.09
80	.75	-.98	.09
81	.59	.76	.07
82	-.59	.76	.07
83	-.59	-.76	.07
84	.59	-.76	.07
85	.42	.55	.05
86	-.42	.55	.05
87	-.42	-.55	.05
88	.42	-.55	.05
89	.26	.34	.03
90	-.26	.34	.03
91	-.26	-.34	.03
92	.26	-.34	.03
93	.11	.14	.01
94	-.11	.14	.01
95	-.11	-.14	.01

96	.11	-.14	.01
97	1.63	1.25	.15
98	-1.63	1.25	.15
99	-1.63	-1.25	.15
100	1.63	-1.25	.15
101	1.41	1.08	.13
102	-1.41	1.08	.13
103	-1.41	-1.08	.13
104	1.41	-1.08	.13
105	1.19	.92	.11
106	-1.19	.92	.11
107	-1.19	-.92	.11
108	1.19	-.92	.11
109	.98	.75	.09
110	-.98	.75	.09
111	-.98	-.75	.09
112	.98	-.75	.09
113	.76	.59	.07
114	-.76	.59	.07
115	-.76	-.59	.07
116	.76	-.59	.07
117	.55	.42	.05
118	-.55	.42	.05
119	-.55	-.42	.05
120	.55	-.42	.05
121	.34	.26	.03
122	-.34	.26	.03
123	-.34	-.26	.03
124	.34	-.26	.03
125	.14	.11	.01
126	-.14	.11	.01
127	-.14	-.11	.01
128	.14	-.11	.01
129	1.89	.78	.15
130	-1.89	.78	.15
131	-1.89	-.78	.15
132	1.89	-.78	.15
133	1.64	.68	.13
134	-1.64	.68	.13
135	-1.64	-.68	.13
136	1.64	-.68	.13
137	1.39	.58	.11
138	-1.39	.58	.11
139	-1.39	-.58	.11
140	1.39	-.58	.11
141	1.14	.47	.09
142	-1.14	.47	.09
143	-1.14	-.47	.09
144	1.14	-.47	.09
145	.89	.37	.07
146	-.89	.37	.07
147	-.89	-.37	.07
148	.89	-.37	.07
149	.64	.26	.05
150	-.64	.26	.05
151	-.64	-.26	.05
152	.64	-.26	.05
153	.39	.16	.03
154	-.39	.16	.03
155	-.39	-.16	.03
156	.39	-.16	.03
157	.17	.07	.01
158	-.17	.07	.01
159	-.17	-.07	.01
160	.17	-.07	.01
161	2.03	.27	.15
162	-2.03	.27	.15
163	-2.03	-.27	.15
164	2.03	-.27	.15
165	1.76	.23	.13
166	-1.76	.23	.13
167	-1.76	-.23	.13
168	1.76	-.23	.13
169	1.49	.20	.11
170	-1.49	.20	.11
171	-1.49	-.20	.11
172	1.49	-.20	.11
173	1.22	.16	.09
174	-1.22	.16	.09
175	-1.22	-.16	.09
176	1.22	-.16	.09
177	.95	.13	.07
178	-.95	.13	.07
179	-.95	-.13	.07
180	.95	-.13	.07
181	.69	.09	.05
182	-.69	.09	.05
183	-.69	-.09	.05
184	.69	-.09	.05
185	.42	.06	.03
186	-.42	.06	.03
187	-.42	-.06	.03
188	.42	-.06	.03
189	.18	.02	.01
190	-.18	.02	.01
191	-.18	-.02	.01
192	.18	-.02	.01

MOMENT OF INERTIA ABOUT Z-AXIS= .179E+02
MOMENT OF INERTIA ABOUT X-AXIS= .179E+02

NUMBER OF BARS IN STEEL FIBERS= 8

STEEL FIBER PROPERTIES

NO.	Z	X	AREA
1	1.86	0.00	.04
2	-1.32	1.32	.04
3	-1.86	0.00	.04
4	-1.32	-1.32	.04
5	0.00	1.86	.04
6	1.32	-1.32	.04
7	0.00	-1.86	.04
8	1.32	1.32	.04

CROSS-SECTION TYPE 2

Z-DIMENSION = 4.39
X-DIMENSION = 4.38

CONCRETE FIBER PROPERTIES

NO.	Z	X	AREA
1	.27	2.03	.15
2	-.27	2.03	.15
3	-.27	-2.03	.15
4	.27	-2.03	.15
5	.23	1.76	.13
6	-.23	1.76	.13
7	-.23	-1.76	.13
8	.23	-1.76	.13
9	.20	1.49	.11
10	-.20	1.49	.11
11	-.20	-1.49	.11
12	.20	-1.49	.11
13	.16	1.22	.09
14	-.16	1.22	.09
15	-.16	-1.22	.09
16	.16	-1.22	.09
17	.13	.95	.07
18	-.13	.95	.07
19	-.13	-.95	.07
20	.13	-.95	.07
21	.09	.69	.05
22	-.09	.69	.05
23	-.09	-.69	.05
24	.09	-.69	.05
25	.06	.42	.03
26	-.06	.42	.03
27	-.06	-.42	.03
28	.06	-.42	.03
29	.02	.16	.01
30	-.02	.16	.01
31	-.02	-.16	.01
32	.02	-.16	.01
33	.78	1.89	.15
34	-.78	1.89	.15
35	-.78	-1.89	.15
36	.78	-1.89	.15
37	.68	1.64	.13
38	-.68	1.64	.13
39	-.68	-1.64	.13
40	.68	-1.64	.13
41	.58	1.39	.11
42	-.58	1.39	.11
43	-.58	-1.39	.11
44	.58	-1.39	.11
45	.47	1.14	.09
46	-.47	1.14	.09
47	-.47	-1.14	.09
48	.47	-1.14	.09
49	.37	.89	.07
50	-.37	.89	.07
51	-.37	-.89	.07
52	.37	-.89	.07
53	.26	.64	.05
54	-.26	.64	.05
55	-.26	-.64	.05
56	.26	-.64	.05
57	.16	.39	.03
58	-.16	.39	.03
59	-.16	-.39	.03
60	.16	-.39	.03
61	.07	.17	.01
62	-.07	.17	.01
63	-.07	-.17	.01
64	.07	-.17	.01
65	1.25	1.63	.15
66	-1.25	1.63	.15
67	-1.25	-1.63	.15
68	1.25	-1.63	.15
69	1.08	1.41	.13
70	-1.08	1.41	.13
71	-1.08	-1.41	.13
72	1.08	-1.41	.13
73	.92	1.19	.11
74	-.92	1.19	.11
75	-.92	-1.19	.11
76	.92	-1.19	.11
77	.75	.98	.09
78	-.75	.98	.09
79	-.75	-.98	.09
80	.75	-.98	.09
81	.59	.76	.07
82	-.59	.76	.07
83	-.59	-.76	.07
84	.59	-.76	.07
85	.42	.55	.05
86	-.42	.55	.05
87	-.42	-.55	.05
88	.42	-.55	.05
89	.26	.34	.03
90	-.26	.34	.03
91	-.26	-.34	.03
92	.26	-.34	.03
93	.11	.14	.01
94	-.11	.14	.01
95	-.11	-.14	.01
96	.11	-.14	.01
97	1.63	1.25	.15
98	-1.63	1.25	.15
99	-1.63	-1.25	.15
100	1.63	-1.25	.15
101	1.41	1.08	.13
102	-1.41	1.08	.13
103	-1.41	-1.08	.13
104	1.41	-1.08	.13
105	1.19	.92	.11
106	-1.19	.92	.11
107	-1.19	-.92	.11
108	1.19	-.92	.11
109	.98	.75	.09
110	-.98	.75	.09
111	-.98	-.75	.09
112	.98	-.75	.09
113	.76	.59	.07
114	-.76	.59	.07
115	-.76	-.59	.07
116	.76	-.59	.07
117	.55	.42	.05
118	-.55	.42	.05

119	-.55	-.42	.05
120	.55	-.42	.05
121	.34	.26	.03
122	-.34	.26	.03
123	-.34	-.26	.03
124	.34	-.26	.03
125	.14	.11	.01
126	-.14	.11	.01
127	-.14	-.11	.01
128	.14	-.11	.01
129	1.89	.78	.15
130	-1.89	.78	.15
131	-1.89	-.78	.15
132	1.89	-.78	.15
133	1.64	.68	.13
134	-1.64	.68	.13
135	-1.64	-.68	.13
136	1.64	-.68	.13
137	1.39	.58	.11
138	-1.39	.58	.11
139	-1.39	-.58	.11
140	1.39	-.58	.11
141	1.14	.47	.09
142	-1.14	.47	.09
143	-1.14	-.47	.09
144	1.14	-.47	.09
145	.89	.37	.07
146	-.89	.37	.07
147	-.89	-.37	.07
148	.89	-.37	.07
149	.64	.26	.05
150	-.64	.26	.05
151	-.64	-.26	.05
152	.64	-.26	.05
153	.39	.16	.03
154	-.39	.16	.03
155	-.39	-.16	.03
156	.39	-.16	.03
157	.17	.07	.01
158	-.17	.07	.01
159	-.17	-.07	.01
160	.17	-.07	.01
161	2.03	.27	.15
162	-2.03	.27	.15
163	-2.03	-.27	.15
164	2.03	-.27	.15
165	1.76	.23	.13
166	-1.76	.23	.13
167	-1.76	-.23	.13
168	1.76	-.23	.13
169	1.49	.20	.11
170	-1.49	.20	.11
171	-1.49	-.20	.11
172	1.49	-.20	.11
173	1.22	.16	.09
174	-1.22	.16	.09
175	-1.22	-.16	.09
176	1.22	-.16	.09
177	.95	.13	.07
178	-.95	.13	.07
179	-.95	-.13	.07
180	.95	-.13	.07
181	.69	.09	.05
182	-.69	.09	.05
183	-.69	-.09	.05
184	.69	-.09	.05
185	.42	.06	.03
186	-.42	.06	.03
187	-.42	-.06	.03
188	.42	-.06	.03
189	.18	.02	.01
190	-.18	.02	.01
191	-.18	-.02	.01
192	.18	-.02	.01

MOMENT OF INERTIA ABOUT Z-AXIS= .170E+02
MOMENT OF INERTIA ABOUT X-AXIS= .170E+02

NUMBER OF READ IN STEEL FIBERS= 8

STEEL FIBER PROPERTIES

NO.	Z	X	AREA
1	1.86	0.00	.02
2	-1.32	1.32	.02
3	-1.86	0.00	.02
4	-1.32	-1.32	.02
5	0.00	1.86	.02
6	1.32	-1.32	.02
7	0.00	-1.86	.02
8	1.32	1.32	.02

MEMBER 3 LEVEL 1 HAS 10 SEGMENTS AND 5 DIFFERENT SECTIONS

SEGMENT LENGTHS	4.36	2.44	2.81	4.19	1.75
	2.25	3.00	5.05	1.95	3.50

THE MEMBER CROSS-SECTION IS SOLID

CONCRETE FIBERS WILL BE MODELLED AS UNCONFINED CONCRETE

STEEL FIBERS ARE READ IN BY THE USER

CROSS-SECTION TYPE 1

Z-DIMENSION = 5.68
X-DIMENSION = 4.38

CONCRETE FIBER PROPERTIES

NO.	Z	X	AREA
1	-2.61	1.99	.15
2	-2.24	1.99	.15
3	-1.87	1.99	.15
4	-1.49	1.99	.15
5	-1.12	1.99	.15
6	-.75	1.99	.15
7	-.37	1.99	.15
8	.00	1.99	.15
9	.37	1.99	.15
10	.75	1.99	.15
11	1.12	1.99	.15
12	1.49	1.99	.15
13	1.87	1.99	.15
14	2.24	1.99	.15
15	2.61	1.99	.15
16	-2.61	1.59	.15
17	-2.24	1.59	.15
18	-1.87	1.59	.15
19	-1.49	1.59	.15
20	-1.12	1.59	.15
21	-.75	1.59	.15
22	-.37	1.59	.15
23	.00	1.59	.15
24	.37	1.59	.15
25	.75	1.59	.15
26	1.12	1.59	.15
27	1.49	1.59	.15
28	1.87	1.59	.15
29	2.24	1.59	.15
30	2.61	1.59	.15
31	-2.61	1.19	.15
32	-2.24	1.19	.15
33	-1.87	1.19	.15
34	-1.49	1.19	.15
35	-1.12	1.19	.15
36	-.75	1.19	.15
37	-.37	1.19	.15
38	.00	1.19	.15
39	.37	1.19	.15
40	.75	1.19	.15
41	1.12	1.19	.15
42	1.49	1.19	.15
43	1.87	1.19	.15
44	2.24	1.19	.15
45	2.61	1.19	.15
46	-2.61	.80	.15
47	-2.24	.80	.15
48	-1.87	.80	.15
49	-1.49	.80	.15
50	-1.12	.80	.15
51	-.75	.80	.15
52	-.37	.80	.15
53	.00	.80	.15
54	.37	.80	.15
55	.75	.80	.15
56	1.12	.80	.15
57	1.49	.80	.15
58	1.87	.80	.15
59	2.24	.80	.15
60	2.61	.80	.15
61	-2.61	.40	.15
62	-2.24	.40	.15
63	-1.87	.40	.15
64	-1.49	.40	.15
65	-1.12	.40	.15
66	-.75	.40	.15
67	-.37	.40	.15
68	.00	.40	.15
69	.37	.40	.15
70	.75	.40	.15
71	1.12	.40	.15
72	1.49	.40	.15
73	1.87	.40	.15
74	2.24	.40	.15
75	2.61	.40	.15
76	-2.61	-.00	.15
77	-2.24	-.00	.15
78	-1.87	-.00	.15
79	-1.49	-.00	.15
80	-1.12	-.00	.15
81	-.75	-.00	.15
82	-.37	-.00	.15
83	.00	-.00	.15
84	.37	-.00	.15
85	.75	-.00	.15
86	1.12	-.00	.15
87	1.49	-.00	.15
88	1.87	-.00	.15
89	2.24	-.00	.15
90	2.61	-.00	.15
91	-2.61	-.40	.15
92	-2.24	-.40	.15
93	-1.87	-.40	.15
94	-1.49	-.40	.15
95	-1.12	-.40	.15
96	-.75	-.40	.15
97	-.37	-.40	.15
98	.00	-.40	.15
99	.37	-.40	.15
00	.75	-.40	.15
01	1.12	-.40	.15
02	1.49	-.40	.15
03	1.87	-.40	.15
04	2.24	-.40	.15
05	2.61	-.40	.15
06	-2.61	-.80	.15
07	-2.24	-.80	.15
08	-1.87	-.80	.15
09	-1.49	-.80	.15
10	-1.12	-.80	.15
11	-.75	-.80	.15
12	-.37	-.80	.15
13	.00	-.80	.15
14	.37	-.80	.15
15	.75	-.80	.15
16	1.12	-.80	.15
17	1.49	-.80	.15
18	1.87	-.80	.15
19	2.24	-.80	.15
20	2.61	-.80	.15
21	-2.61	-1.19	.15
22	-2.24	-1.19	.15
23	-1.87	-1.19	.15

24	-1.49	-1.19	.15
25	-1.12	-1.19	.15
26	-.75	-1.19	.15
27	-.37	-1.19	.15
28	.00	-1.19	.15
29	.37	-1.19	.15
30	.75	-1.19	.15
31	1.12	-1.19	.15
32	1.49	-1.19	.15
33	1.87	-1.19	.15
34	2.24	-1.19	.15
35	2.61	-1.19	.15
36	-2.61	-1.59	.15
37	-2.24	-1.59	.15
38	-1.87	-1.59	.15
39	-1.49	-1.59	.15
40	-1.12	-1.59	.15
41	-.75	-1.59	.15
42	-.37	-1.59	.15
43	.00	-1.59	.15
44	.37	-1.59	.15
45	.75	-1.59	.15
46	1.12	-1.59	.15
47	1.49	-1.59	.15
48	1.87	-1.59	.15
49	2.24	-1.59	.15
50	2.61	-1.59	.15
51	-2.61	-1.99	.15
52	-2.24	-1.99	.15
53	-1.87	-1.99	.15
54	-1.49	-1.99	.15
55	-1.12	-1.99	.15
56	-.75	-1.99	.15
57	-.37	-1.99	.15
58	.00	-1.99	.15
59	.37	-1.99	.15
60	.75	-1.99	.15
61	1.12	-1.99	.15
62	1.49	-1.99	.15
63	1.87	-1.99	.15
64	2.24	-1.99	.15
65	2.61	-1.99	.15

MOMENT OF INERTIA ABOUT Z-AXIS= .389E+02
MOMENT OF INERTIA ABOUT X-AXIS= .638E+02

NUMBER OF BEAD IN STEEL FIBERS= 4

STEEL FIBER PROPERTIES

NO.	Z	X	AREA
1	2.13	-1.86	.83
2	2.13	1.86	.83
3	-2.13	-1.86	.85
4	-2.13	1.86	.85

CROSS-SECTION TYPE 2

Z-DIMENSION = 5.60
X-DIMENSION = 4.30

CONCRETE FIBER PROPERTIES

NO.	Z	X	AREA
1	-2.61	1.99	.15
2	-2.24	1.99	.15
3	-1.87	1.99	.15
4	-1.49	1.99	.15
5	-1.12	1.99	.15
6	-.75	1.99	.15
7	-.37	1.99	.15
8	.00	1.99	.15
9	.37	1.99	.15
10	.75	1.99	.15
11	1.12	1.99	.15
12	1.49	1.99	.15
13	1.87	1.99	.15
14	2.24	1.99	.15
15	2.61	1.99	.15
16	-2.61	1.59	.15
17	-2.24	1.59	.15
18	-1.87	1.59	.15
19	-1.49	1.59	.15
20	-1.12	1.59	.15
21	-.75	1.59	.15
22	-.37	1.59	.15
23	.00	1.59	.15
24	.37	1.59	.15
25	.75	1.59	.15
26	1.12	1.59	.15
27	1.49	1.59	.15
28	1.87	1.59	.15
29	2.24	1.59	.15
30	2.61	1.59	.15
31	-2.61	1.19	.15
32	-2.24	1.19	.15
33	-1.87	1.19	.15
34	-1.49	1.19	.15
35	-1.12	1.19	.15
36	-.75	1.19	.15
37	-.37	1.19	.15
38	.00	1.19	.15
39	.37	1.19	.15
40	.75	1.19	.15
41	1.12	1.19	.15
42	1.49	1.19	.15
43	1.87	1.19	.15
44	2.24	1.19	.15
45	2.61	1.19	.15
46	-2.61	.80	.15
47	-2.24	.80	.15
48	-1.87	.80	.15
49	-1.49	.80	.15
50	-1.12	.80	.15
51	-.75	.80	.15
52	-.37	.80	.15
53	.00	.80	.15

54	.37	.80	.15
55	.75	.80	.15
56	1.12	.80	.15
57	1.49	.80	.15
58	1.87	.80	.15
59	2.24	.80	.15
60	2.61	.80	.15
61	-2.61	.40	.15
62	-2.24	.40	.15
63	-1.87	.40	.15
64	-1.49	.40	.15
65	-1.12	.40	.15
66	-.75	.40	.15
67	-.37	.40	.15
68	.00	.40	.15
69	.37	.40	.15
70	.75	.40	.15
71	1.12	.40	.15
72	1.49	.40	.15
73	1.87	.40	.15
74	2.24	.40	.15
75	2.61	.40	.15
76	-2.61	-.00	.15
77	-2.24	-.00	.15
78	-1.87	-.00	.15
79	-1.49	-.00	.15
80	-1.12	-.00	.15
81	-.75	-.00	.15
82	-.37	-.00	.15
83	.00	-.00	.15
84	.37	-.00	.15
85	.75	-.00	.15
86	1.12	-.00	.15
87	1.49	-.00	.15
88	1.87	-.00	.15
89	2.24	-.00	.15
90	2.61	-.00	.15
91	-2.61	-.40	.15
92	-2.24	-.40	.15
93	-1.87	-.40	.15
94	-1.49	-.40	.15
95	-1.12	-.40	.15
96	-.75	-.40	.15
97	-.37	-.40	.15
98	.00	-.40	.15
99	.37	-.40	.15
00	.75	-.40	.15
01	1.12	-.40	.15
02	1.49	-.40	.15
03	1.87	-.40	.15
04	2.24	-.40	.15
05	2.61	-.40	.15
06	-2.61	-.80	.15
07	-2.24	-.80	.15
08	-1.87	-.80	.15
09	-1.49	-.80	.15
10	-1.12	-.80	.15
11	-.75	-.80	.15
12	-.37	-.80	.15
13	.00	-.80	.15
14	.37	-.80	.15
15	.75	-.80	.15
16	1.12	-.80	.15
17	1.49	-.80	.15
18	1.87	-.80	.15
19	2.24	-.80	.15
20	2.61	-.80	.15
21	-2.61	-1.19	.15
22	-2.24	-1.19	.15
23	-1.87	-1.19	.15
24	-1.49	-1.19	.15
25	-1.12	-1.19	.15
26	-.75	-1.19	.15
27	-.37	-1.19	.15
28	.00	-1.19	.15
29	.37	-1.19	.15
30	.75	-1.19	.15
31	1.12	-1.19	.15
32	1.49	-1.19	.15
33	1.87	-1.19	.15
34	2.24	-1.19	.15
35	2.61	-1.19	.15
36	-2.61	-1.59	.15
37	-2.24	-1.59	.15
38	-1.87	-1.59	.15
39	-1.49	-1.59	.15
40	-1.12	-1.59	.15
41	-.75	-1.59	.15
42	-.37	-1.59	.15
43	.00	-1.59	.15
44	.37	-1.59	.15
45	.75	-1.59	.15
46	1.12	-1.59	.15
47	1.49	-1.59	.15
48	1.87	-1.59	.15
49	2.24	-1.59	.15
50	2.61	-1.59	.15
51	-2.61	-1.99	.15
52	-2.24	-1.99	.15
53	-1.87	-1.99	.15
54	-1.49	-1.99	.15
55	-1.12	-1.99	.15
56	-.75	-1.99	.15
57	-.37	-1.99	.15
58	.00	-1.99	.15
59	.37	-1.99	.15
60	.75	-1.99	.15
61	1.12	-1.99	.15
62	1.49	-1.99	.15
63	1.87	-1.99	.15
64	2.24	-1.99	.15
65	2.61	-1.99	.15

MOMENT OF INERTIA ABOUT Z-AXIS= .389E+02
MOMENT OF INERTIA ABOUT X-AXIS= .638E+02

NUMBER OF READ IN STEEL FIBERS= 4

STEEL FIBER PROPERTIES

NO.	Z	X	AREA
1	2.13	-1.86	.84
2	2.13	1.86	.84
3	-2.13	-1.86	.85
4	-2.13	1.86	.85

CROSS-SECTION TYPE 3

Z-DIMENSION = 5.60
X-DIMENSION = 4.38

CONCRETE FIBER PROPERTIES

NO.	Z	X	AREA
1	-2.61	1.99	.15
2	-2.24	1.99	.15
3	-1.87	1.99	.15
4	-1.49	1.99	.15
5	-1.12	1.99	.15
6	-.75	1.99	.15
7	-.37	1.99	.15
8	.00	1.99	.15
9	.37	1.99	.15
10	.75	1.99	.15
11	1.12	1.99	.15
12	1.49	1.99	.15
13	1.87	1.99	.15
14	2.24	1.99	.15
15	2.61	1.99	.15
16	-2.61	1.59	.15
17	-2.24	1.59	.15
18	-1.87	1.59	.15
19	-1.49	1.59	.15
20	-1.12	1.59	.15
21	-.75	1.59	.15
22	-.37	1.59	.15
23	.00	1.59	.15
24	.37	1.59	.15
25	.75	1.59	.15
26	1.12	1.59	.15
27	1.49	1.59	.15
28	1.87	1.59	.15
29	2.24	1.59	.15
30	2.61	1.59	.15
31	-2.61	1.19	.15
32	-2.24	1.19	.15
33	-1.87	1.19	.15
34	-1.49	1.19	.15
35	-1.12	1.19	.15
36	-.75	1.19	.15
37	-.37	1.19	.15
38	.00	1.19	.15
39	.37	1.19	.15
40	.75	1.19	.15
41	1.12	1.19	.15
42	1.49	1.19	.15
43	1.87	1.19	.15
44	2.24	1.19	.15
45	2.61	1.19	.15
46	-2.61	.80	.15
47	-2.24	.80	.15
48	-1.87	.80	.15
49	-1.49	.80	.15
50	-1.12	.80	.15
51	-.75	.80	.15
52	-.37	.80	.15
53	.00	.80	.15
54	.37	.80	.15
55	.75	.80	.15
56	1.12	.80	.15
57	1.49	.80	.15
58	1.87	.80	.15
59	2.24	.80	.15
60	2.61	.80	.15
61	-2.61	.40	.15
62	-2.24	.40	.15
63	-1.87	.40	.15
64	-1.49	.40	.15
65	-1.12	.40	.15
66	-.75	.40	.15
67	-.37	.40	.15
68	.00	.40	.15
69	.37	.40	.15
70	.75	.40	.15
71	1.12	.40	.15
72	1.49	.40	.15
73	1.87	.40	.15
74	2.24	.40	.15
75	2.61	.40	.15
76	-2.61	-.00	.15
77	-2.24	-.00	.15
78	-1.87	-.00	.15
79	-1.49	-.00	.15
80	-1.12	-.00	.15
81	-.75	-.00	.15
82	-.37	-.00	.15
83	.00	-.00	.15
84	.37	-.00	.15
85	.75	-.00	.15
86	1.12	-.00	.15
87	1.49	-.00	.15
88	1.87	-.00	.15
89	2.24	-.00	.15
90	2.61	-.00	.15
91	-2.61	-.40	.15
92	-2.24	-.40	.15
93	-1.87	-.40	.15
94	-1.49	-.40	.15
95	-1.12	-.40	.15
96	-.75	-.40	.15
97	-.37	-.40	.15
98	.00	-.40	.15
99	.37	-.40	.15
100	.75	-.40	.15
101	1.12	-.40	.15
102	1.49	-.40	.15
103	1.87	-.40	.15
104	2.24	-.40	.15
105	2.61	-.40	.15
106	-2.61	-.80	.15
107	-2.24	-.80	.15

108	-1.87	-.80	.15
109	-1.49	-.80	.15
110	-1.12	-.80	.15
111	-.75	-.80	.15
112	-.37	-.80	.15
113	.00	-.80	.15
114	.37	-.80	.15
115	.75	-.80	.15
116	1.12	-.80	.15
117	1.49	-.80	.15
118	1.87	-.80	.15
119	2.24	-.80	.15
120	2.61	-.80	.15
121	-2.61	-1.19	.15
122	-2.24	-1.19	.15
123	-1.87	-1.19	.15
124	-1.49	-1.19	.15
125	-1.12	-1.19	.15
126	-.75	-1.19	.15
127	-.37	-1.19	.15
128	.00	-1.19	.15
129	.37	-1.19	.15
130	.75	-1.19	.15
131	1.12	-1.19	.15
132	1.49	-1.19	.15
133	1.87	-1.19	.15
134	2.24	-1.19	.15
135	2.61	-1.19	.15
136	-2.61	-1.59	.15
137	-2.24	-1.59	.15
138	-1.87	-1.59	.15
139	-1.49	-1.59	.15
140	-1.12	-1.59	.15
141	-.75	-1.59	.15
142	-.37	-1.59	.15
143	.00	-1.59	.15
144	.37	-1.59	.15
145	.75	-1.59	.15
146	1.12	-1.59	.15
147	1.49	-1.59	.15
148	1.87	-1.59	.15
149	2.24	-1.59	.15
150	2.61	-1.59	.15
151	-2.61	-1.99	.15
152	-2.24	-1.99	.15
153	-1.87	-1.99	.15
154	-1.49	-1.99	.15
155	-1.12	-1.99	.15
156	-.75	-1.99	.15
157	-.37	-1.99	.15
158	.00	-1.99	.15
159	.37	-1.99	.15
160	.75	-1.99	.15
161	1.12	-1.99	.15
162	1.49	-1.99	.15
163	1.87	-1.99	.15
164	2.24	-1.99	.15
165	2.61	-1.99	.15

MOMENT OF INERTIA ABOUT Z-AXIS= .309E+02
MOMENT OF INERTIA ABOUT X-AXIS= .638E+02

NUMBER OF BARS IN STEEL FIBERS= 4

STEEL FIBER PROPERTIES

NO.	Z	X	AREA
1	2.13	-1.86	.04
2	2.13	1.86	.04
3	-2.13	-1.86	.03
4	-2.13	1.86	.03

CROSS-SECTION TYPE 4

Z-DIMENSION = 5.60
X-DIMENSION = 4.38

CONCRETE FIBER PROPERTIES

NO.	Z	X	AREA
1	-2.61	1.99	.15
2	-2.24	1.99	.15
3	-1.87	1.99	.15
4	-1.49	1.99	.15
5	-1.12	1.99	.15
6	-.75	1.99	.15
7	-.37	1.99	.15
8	.00	1.99	.15
9	.37	1.99	.15
10	.75	1.99	.15
11	1.12	1.99	.15
12	1.49	1.99	.15
13	1.87	1.99	.15
14	2.24	1.99	.15
15	2.61	1.99	.15
16	-2.61	1.59	.15
17	-2.24	1.59	.15
18	-1.87	1.59	.15
19	-1.49	1.59	.15
20	-1.12	1.59	.15
21	-.75	1.59	.15
22	-.37	1.59	.15
23	.00	1.59	.15
24	.37	1.59	.15
25	.75	1.59	.15
26	1.12	1.59	.15
27	1.49	1.59	.15
28	1.87	1.59	.15
29	2.24	1.59	.15
30	2.61	1.59	.15
31	-2.61	1.19	.15
32	-2.24	1.19	.15
33	-1.87	1.19	.15
34	-1.49	1.19	.15
35	-1.12	1.19	.15
36	-.75	1.19	.15
37	-.37	1.19	.15

38	.00	1.19	.15
39	.37	1.19	.15
40	.75	1.19	.15
41	1.12	1.19	.15
42	1.49	1.19	.15
43	1.87	1.19	.15
44	2.24	1.19	.15
45	2.61	1.19	.15
46	-2.61	.80	.15
47	-2.24	.80	.15
48	-1.87	.80	.15
49	-1.49	.80	.15
50	-1.12	.80	.15
51	-.75	.80	.15
52	-.37	.80	.15
53	.00	.80	.15
54	.37	.80	.15
55	.75	.80	.15
56	1.12	.80	.15
57	1.49	.80	.15
58	1.87	.80	.15
59	2.24	.80	.15
60	2.61	.80	.15
61	-2.61	.40	.15
62	-2.24	.40	.15
63	-1.87	.40	.15
64	-1.49	.40	.15
65	-1.12	.40	.15
66	-.75	.40	.15
67	-.37	.40	.15
68	.00	.40	.15
69	.37	.40	.15
70	.75	.40	.15
71	1.12	.40	.15
72	1.49	.40	.15
73	1.87	.40	.15
74	2.24	.40	.15
75	2.61	.40	.15
76	-2.61	-.00	.15
77	-2.24	-.00	.15
78	-1.87	-.00	.15
79	-1.49	-.00	.15
80	-1.12	-.00	.15
81	-.75	-.00	.15
82	-.37	-.00	.15
83	.00	-.00	.15
84	.37	-.00	.15
85	.75	-.00	.15
86	1.12	-.00	.15
87	1.49	-.00	.15
88	1.87	-.00	.15
89	2.24	-.00	.15
90	2.61	-.00	.15
91	-2.61	-.40	.15
92	-2.24	-.40	.15
93	-1.87	-.40	.15
94	-1.49	-.40	.15
95	-1.12	-.40	.15
96	-.75	-.40	.15
97	-.37	-.40	.15
98	.00	-.40	.15
99	.37	-.40	.15

100	.75	-.80	.15
101	1.12	-.80	.15
102	1.49	-.80	.15
103	1.87	-.80	.15
104	2.24	-.80	.15
105	2.61	-.80	.15
106	-2.61	-.80	.15
107	-2.24	-.80	.15
108	-1.87	-.80	.15
109	-1.49	-.80	.15
110	-1.12	-.80	.15
111	-.75	-.80	.15
112	-.37	-.80	.15
113	.00	-.80	.15
114	.37	-.80	.15
115	.75	-.80	.15
116	1.12	-.80	.15
117	1.49	-.80	.15
118	1.87	-.80	.15
119	2.24	-.80	.15
120	2.61	-.80	.15
121	-2.61	-1.19	.15
122	-2.24	-1.19	.15
123	-1.87	-1.19	.15
124	-1.49	-1.19	.15
125	-1.12	-1.19	.15
126	-.75	-1.19	.15
127	-.37	-1.19	.15
128	.00	-1.19	.15
129	.37	-1.19	.15
130	.75	-1.19	.15
131	1.12	-1.19	.15
132	1.49	-1.19	.15
133	1.87	-1.19	.15
134	2.24	-1.19	.15
135	2.61	-1.19	.15
136	-2.61	-1.59	.15
137	-2.24	-1.59	.15
138	-1.87	-1.59	.15
139	-1.49	-1.59	.15
140	-1.12	-1.59	.15
141	-.75	-1.59	.15
142	-.37	-1.59	.15
143	.00	-1.59	.15
144	.37	-1.59	.15
145	.75	-1.59	.15
146	1.12	-1.59	.15
147	1.49	-1.59	.15
148	1.87	-1.59	.15
149	2.24	-1.59	.15
150	2.61	-1.59	.15
151	-2.61	-1.99	.15
152	-2.24	-1.99	.15
153	-1.87	-1.99	.15
154	-1.49	-1.99	.15
155	-1.12	-1.99	.15
156	-.75	-1.99	.15
157	-.37	-1.99	.15
158	.00	-1.99	.15
159	.37	-1.99	.15
160	.75	-1.99	.15
161	1.12	-1.99	.15

162	1.49	-1.99	.15
163	1.87	-1.99	.15
164	2.24	-1.99	.15
165	2.61	-1.99	.15

MOMENT OF INERTIA ABOUT Z-AXIS= .389E+02
MOMENT OF INERTIA ABOUT X-AXIS= .638E+02

4

NUMBER OF REAR IN STEEL FIBERS= 4

STEEL FIBER PROPERTIES

NO.	Z	X	AREA
1	2.13	-1.86	.84
2	2.13	1.86	.84
3	-2.13	-1.86	.18
4	-2.13	1.86	.18

CROSS-SECTION TYPE 5

Z-DIMENSION = 5.60
X-DIMENSION = 4.38

CONCRETE FIBER PROPERTIES

NO.	Z	X	AREA
1	-2.61	1.99	.15
2	-2.24	1.99	.15
3	-1.87	1.99	.15
4	-1.49	1.99	.15
5	-1.12	1.99	.15
6	-.75	1.99	.15
7	-.37	1.99	.15
8	.00	1.99	.15
9	.37	1.99	.15
10	.75	1.99	.15
11	1.12	1.99	.15
12	1.49	1.99	.15
13	1.87	1.99	.15
14	2.24	1.99	.15
15	2.61	1.99	.15
16	-2.61	1.59	.15
17	-2.24	1.59	.15
18	-1.87	1.59	.15
19	-1.49	1.59	.15
20	-1.12	1.59	.15
21	-.75	1.59	.15
22	-.37	1.59	.15
23	.00	1.59	.15
24	.37	1.59	.15
25	.75	1.59	.15
26	1.12	1.59	.15
27	1.49	1.59	.15
28	1.87	1.59	.15
29	2.24	1.59	.15
30	2.61	1.59	.15
31	-2.61	1.19	.15
32	-2.24	1.19	.15
33	-1.87	1.19	.15
34	-1.49	1.19	.15
35	-1.12	1.19	.15
36	-.75	1.19	.15
37	-.37	1.19	.15
38	.00	1.19	.15
39	.37	1.19	.15
40	.75	1.19	.15
41	1.12	1.19	.15
42	1.49	1.19	.15
43	1.87	1.19	.15
44	2.24	1.19	.15
45	2.61	1.19	.15
46	-2.61	.80	.15
47	-2.24	.80	.15
48	-1.87	.80	.15
49	-1.49	.80	.15
50	-1.12	.80	.15
51	-.75	.80	.15
52	-.37	.80	.15
53	.00	.80	.15
54	.37	.80	.15
55	.75	.80	.15
56	1.12	.80	.15
57	1.49	.80	.15
58	1.87	.80	.15
59	2.24	.80	.15
60	2.61	.80	.15
61	-2.61	.40	.15
62	-2.24	.40	.15
63	-1.87	.40	.15
64	-1.49	.40	.15
65	-1.12	.40	.15
66	-.75	.40	.15
67	-.37	.40	.15
68	.00	.40	.15
69	.37	.40	.15
70	.75	.40	.15
71	1.12	.40	.15
72	1.49	.40	.15
73	1.87	.40	.15
74	2.24	.40	.15
75	2.61	.40	.15
76	-2.61	-.00	.15
77	-2.24	-.00	.15
78	-1.87	-.00	.15
79	-1.49	-.00	.15
80	-1.12	-.00	.15
81	-.75	-.00	.15
82	-.37	-.00	.15
83	.00	-.00	.15
84	.37	-.00	.15
85	.75	-.00	.15
86	1.12	-.00	.15
87	1.49	-.00	.15
88	1.87	-.00	.15
89	2.24	-.00	.15
90	2.61	-.00	.15
91	-2.61	-.40	.15

92	-2.24	-.40	.15
93	-1.87	-.40	.15
94	-1.49	-.40	.15
95	-1.12	-.40	.15
96	-.75	-.40	.15
97	-.37	-.40	.15
98	.00	-.40	.15
99	.37	-.40	.15
100	.75	-.40	.15
101	1.12	-.40	.15
102	1.49	-.40	.15
103	1.87	-.40	.15
104	2.24	-.40	.15
105	2.61	-.40	.15
106	-2.61	-.80	.15
107	-2.24	-.80	.15
108	-1.87	-.80	.15
109	-1.49	-.80	.15
110	-1.12	-.80	.15
111	-.75	-.80	.15
112	-.37	-.80	.15
113	.00	-.80	.15
114	.37	-.80	.15
115	.75	-.80	.15
116	1.12	-.80	.15
117	1.49	-.80	.15
118	1.87	-.80	.15
119	2.24	-.80	.15
120	2.61	-.80	.15
121	-2.61	-1.19	.15
122	-2.24	-1.19	.15
123	-1.87	-1.19	.15
124	-1.49	-1.19	.15
125	-1.12	-1.19	.15
126	-.75	-1.19	.15
127	-.37	-1.19	.15
128	.00	-1.19	.15
129	.37	-1.19	.15
130	.75	-1.19	.15
131	1.12	-1.19	.15
132	1.49	-1.19	.15
133	1.87	-1.19	.15
134	2.24	-1.19	.15
135	2.61	-1.19	.15
136	-2.61	-1.59	.15
137	-2.24	-1.59	.15
138	-1.87	-1.59	.15
139	-1.49	-1.59	.15
140	-1.12	-1.59	.15
141	-.75	-1.59	.15
142	-.37	-1.59	.15
143	.00	-1.59	.15
144	.37	-1.59	.15
145	.75	-1.59	.15
146	1.12	-1.59	.15
147	1.49	-1.59	.15
148	1.87	-1.59	.15
149	2.24	-1.59	.15
150	2.61	-1.59	.15
151	-2.61	-1.99	.15
152	-2.24	-1.99	.15
153	-1.87	-1.99	.15
154	-1.49	-1.99	.15
155	-1.12	-1.99	.15
156	-.75	-1.99	.15
157	-.37	-1.99	.15
158	.00	-1.99	.15
159	.37	-1.99	.15
160	.75	-1.99	.15
161	1.12	-1.99	.15
162	1.49	-1.99	.15
163	1.87	-1.99	.15
164	2.24	-1.99	.15
165	2.61	-1.99	.15

MOMENT OF INERTIA ABOUT Z-AXIS= .309E+02
MOMENT OF INERTIA ABOUT X-AXIS= .638E+02

NUMBER OF READ IN STEEL FIBERS= 4

STEEL FIBER PROPERTIES

NO.	Z	X	AREA
1	2.13	-1.86	.03
2	2.13	1.86	.03
3	-2.13	-1.86	.10
4	-2.13	1.86	.10

NUMBER OF LOAD CASES= 3

*** LOAD CASE 1 ***

NUMBER OF LOAD INCREMENTS = 30
NUMBER OF MEMBERS LOADED = 2
PRINTING INDEX = 30

LOADED NO. 4

JOINT NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
3	0.	-.340E+02	0.	0.	0.	0.
5	0.	-.340E+02	0.	0.	0.	0.
8	0.	-.340E+02	0.	0.	0.	0.
10	0.	-.340E+02	0.	0.	0.	0.

LOADED NO. 5

JOINT NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
3	0.	-.340E+02	0.	0.	0.	0.
5	0.	-.340E+02	0.	0.	0.	0.
8	0.	-.340E+02	0.	0.	0.	0.
10	0.	-.340E+02	0.	0.	0.	0.

NUMBER OF FRAME JOINTS LOADED = 2

JOINT NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
8	0.	-.892E+02	0.	0.	0.	.242E+03
6	0.	-.892E+02	0.	0.	0.	-.242E+03

MEMBER 1
****

LOAD INCREMENT 1
****

*** DISPLACEMENTS ***

JOINT NO.	DISPL.-X	DISPL.-Y	DISPL.-Z	ROTATION-X	ROTATION-Y	ROTATION-Z
1	0.	0.	0.	0.	0.	0.
2	-.985E-06	-.073E-05	-.173E-20	-.135E-20	-.118E-20	.080E-06
3	-.290E-05	-.047E-05	-.049E-20	-.200E-20	-.230E-20	.142E-05
4	-.590E-05	-.140E-04	-.163E-19	-.359E-20	-.373E-20	.126E-05
5	-.924E-05	-.271E-04	-.262E-19	-.815E-20	-.518E-20	.120E-05
6	-.123E-04	-.255E-04	-.350E-19	-.455E-20	-.600E-20	.122E-05
7	-.148E-04	-.374E-04	-.077E-19	-.605E-20	-.703E-20	.542E-06
8	-.149E-04	-.361E-04	-.590E-19	-.600E-20	-.919E-20	-.175E-06
9	-.133E-04	-.415E-04	-.707E-19	-.307E-20	-.180E-19	-.113E-05
10	-.882E-05	-.808E-04	-.881E-19	-.319E-20	-.119E-19	-.233E-05
11	-.107E-05	-.521E-04	-.871E-19	-.211E-20	-.133E-19	-.370E-05

*** FORCES ***

SEG.NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
1						

END-A	.205E+01	.125E+03	.328E-14	.889E-13	.315E-13	-.234E+02
END-B	-.205E+01	-.125E+03	-.328E-14	-.889E-13	-.315E-13	.169E+02

SEG.NO. 2	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.205E+01	.125E+03	.328E-14	.889E-13	.315E-13	-.169E+02
END-B	-.205E+01	-.125E+03	-.328E-14	-.328E-13	-.315E-13	.184E+02

SEG.NO. 3	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.205E+01	.125E+03	.328E-14	.328E-13	.315E-13	-.184E+02
END-B	-.205E+01	-.125E+03	-.328E-14	-.283E-13	-.315E-13	.348E+01

SEG.NO. 4	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.205E+01	.125E+03	.328E-14	.283E-13	.315E-13	-.348E+01
END-B	-.205E+01	-.125E+03	-.328E-14	-.198E-13	-.315E-13	-.301E+01

SEG.NO. 5	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.205E+01	.125E+03	.328E-14	.198E-13	.315E-13	.301E+01
END-B	-.205E+01	-.125E+03	-.328E-14	-.720E-14	-.315E-13	-.103E+02

SEG.NO. 6	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.205E+01	.125E+03	.328E-14	.720E-14	.315E-13	.103E+02
END-B	-.205E+01	-.125E+03	-.328E-14	.124E-14	-.315E-13	-.172E+02

SEG.NO. 7	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.205E+01	.125E+03	.328E-14	-.124E-14	.315E-13	.172E+02
END-B	-.205E+01	-.125E+03	-.328E-14	.970E-14	-.315E-13	-.201E+02

SEG.NO. 8	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.205E+01	.125E+03	.328E-14	-.970E-14	.315E-13	.201E+02
END-B	-.205E+01	-.125E+03	-.328E-14	.103E-13	-.315E-13	-.318E+02

SEG.NO. 9	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.205E+01	.125E+03	.328E-14	-.103E-13	.315E-13	.318E+02
END-B	-.205E+01	-.125E+03	-.328E-14	.268E-13	-.315E-13	-.379E+02

SEG.NO. 10	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.205E+01	.125E+03	.328E-14	-.268E-13	.315E-13	.379E+02
END-B	-.205E+01	-.125E+03	-.328E-14	.393E-13	-.315E-13	-.448E+02

MEMBER 2

LOAD INCREMENT 1

*** DISPLACEMENTS ***

JOINT NO.	DISPL.-X	DISPL.-Y	DISPL.-Z	ROTATION-X	ROTATION-Y	ROTATION-Z
12	0.	0.	0.	0.	0.	0.
13	.302E-07	-.571E-09	.392E-20	.274E-22	-.260E-22	-.274E-07
14	.176E-06	-.114E-08	.132E-19	.409E-20	-.929E-22	-.464E-07
15	.234E-06	-.170E-08	.200E-19	.701E-20	-.834E-22	-.303E-07
16	.314E-06	-.243E-08	.402E-19	.842E-20	-.110E-21	-.106E-07
17	.319E-06	-.307E-08	.724E-19	.920E-20	-.144E-21	.108E-07
18	.244E-06	-.372E-08	.949E-19	.944E-20	-.175E-21	.734E-07
19	-.762E-07	-.430E-08	.171E-18	.900E-20	-.206E-21	.146E-06
20	-.509E-06	-.500E-08	.164E-18	.810E-20	-.234E-21	.237E-06
21	-.132E-05	-.505E-08	.165E-18	.653E-20	-.267E-21	.345E-06
22	-.237E-05	-.620E-08	.177E-18	.439E-20	-.297E-21	.479E-06

*** FORCES ***

SEG.NO.		FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
1	END-A	-.195E+00	.151E+03	-.663E-14	-.903E-13	.703E-15	.115E+01
	END-B	.195E+00	-.151E+03	.663E-14	.830E-13	-.703E-15	-.673E+00
2	END-A	-.195E+00	.151E+03	-.663E-14	-.830E-13	.703E-15	.673E+00
	END-B	.195E+00	-.151E+03	.663E-14	.667E-13	-.703E-15	-.192E+00
3	END-A	-.195E+00	.151E+03	-.663E-14	-.667E-13	.703E-15	.192E+00
	END-B	.195E+00	-.151E+03	.663E-14	.404E-13	-.703E-15	.315E+00
4	END-A	-.195E+00	.151E+03	-.663E-14	-.404E-13	.703E-15	-.315E+00
	END-B	.195E+00	-.151E+03	.663E-14	.322E-13	-.703E-15	.823E+00
5	END-A	-.195E+00	.151E+03	-.663E-14	-.322E-13	.703E-15	-.823E+00
	END-B	.195E+00	-.151E+03	.663E-14	.149E-13	-.703E-15	.133E+01
6	END-A	-.195E+00	.151E+03	-.663E-14	-.149E-13	.703E-15	-.133E+01
	END-B	.195E+00	-.151E+03	.663E-14	-.231E-14	-.703E-15	.184E+01
7	END-A	-.195E+00	.151E+03	-.663E-14	.231E-14	.703E-15	-.184E+01
	END-B	.195E+00	-.151E+03	.663E-14	-.166E-13	-.703E-15	.235E+01
8	END-A	-.195E+00	.151E+03	-.663E-14	.166E-13	.703E-15	-.235E+01
	END-B	.195E+00	-.151E+03	.663E-14	-.368E-13	-.703E-15	.285E+01

MEMBER 3

LOAD INCREMENT 1

>>> DISPLACEMENTS <<<

JOINT NO.	DISPL.-X	DISPL.-Y	DISPL.-Z	ROTATION-X	ROTATION-Y	ROTATION-Z
23	0.	0.	0.	0.	0.	0.
24	.706E-08	-.922E-05	-.179E-20	-.139E-20	.111E-20	-.529E-06
25	.249E-05	-.140E-04	-.669E-20	-.293E-20	.222E-20	-.876E-06
26	.988E-05	-.163E-04	-.147E-19	-.356E-20	.391E-20	-.108E-05
27	.796E-05	-.272E-04	-.250E-19	-.820E-20	.879E-20	-.102E-05
28	.103E-04	-.281E-04	-.367E-19	-.849E-20	.688E-20	-.766E-06
29	.117E-04	-.349E-04	-.492E-19	-.491E-20	.736E-20	-.287E-06
30	.116E-04	-.399E-04	-.616E-19	-.462E-20	.966E-20	.415E-06
31	.900E-05	-.459E-04	-.730E-19	-.413E-20	.993E-20	.134E-05
32	.451E-05	-.517E-04	-.822E-19	-.333E-20	.112E-19	.240E-05
33	-.365E-05	-.579E-04	-.962E-19	-.220E-20	.125E-19	.385E-05

>>> FORCES <<<

SEG.NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
1						

SEG.NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
9						
END-A	-.195E+00	.151E+03	-.663E-14	.360E-13	.703E-15	-.285E+01
END-B	.195E+00	-.151E+03	.663E-14	-.561E-13	-.703E-15	.336E+01

SEG.NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
12						
END-A	-.195E+00	.151E+03	-.663E-14	.561E-13	.703E-15	-.336E+01
END-B	.195E+00	-.151E+03	.663E-14	-.713E-13	-.703E-15	.387E+01

END-A	-.246E+01	.138E+03	.336E-14	.504E-13	-.296E-13	.244E+02
END-B	.246E+01	-.138E+03	-.336E-14	-.821E-13	.296E-13	-.146E+02

SEG.NO. 2	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.246E+01	.138E+03	.336E-14	.821E-13	-.296E-13	.146E+02
END-B	.246E+01	-.138E+03	-.336E-14	-.339E-13	.296E-13	-.853E+01

SEG.NO. 3	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.246E+01	.138E+03	.336E-14	.339E-13	-.296E-13	.853E+01
END-B	.246E+01	-.138E+03	-.336E-14	-.291E-13	.296E-13	-.214E+01

SEG.NO. 4	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.246E+01	.138E+03	.336E-14	.291E-13	-.296E-13	.214E+01
END-B	.246E+01	-.138E+03	-.336E-14	-.164E-13	.296E-13	.424E+01

SEG.NO. 5	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.246E+01	.138E+03	.336E-14	.164E-13	-.296E-13	-.424E+01
END-B	.246E+01	-.138E+03	-.336E-14	-.767E-14	.296E-13	.106E+02

SEG.NO. 6	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.246E+01	.138E+03	.336E-14	.767E-14	-.296E-13	-.106E+02
END-B	.246E+01	-.138E+03	-.336E-14	.100E-14	.296E-13	.170E+02

SEG.NO. 7	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.246E+01	.138E+03	.336E-14	-.100E-14	-.296E-13	-.170E+02
END-B	.246E+01	-.138E+03	-.336E-14	.980E-14	.296E-13	.234E+02

SEG.NO. 8	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.246E+01	.138E+03	.336E-14	-.980E-14	-.296E-13	-.234E+02
END-B	.246E+01	-.138E+03	-.336E-14	.105E-13	.296E-13	.298E+02

SEG.NO. 9	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.246E+01	.138E+03	.336E-14	-.105E-13	-.296E-13	-.298E+02
END-B	.246E+01	-.138E+03	-.336E-14	.273E-13	.296E-13	.362E+02

SEG.NO. 10	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.246E+01	.138E+03	.336E-14	-.273E-13	-.296E-13	-.362E+02
END-B	.246E+01	-.138E+03	-.336E-14	.540E-13	.296E-13	.426E+02

MEMBER 8

LOAD INCREMENT 1

>>> DISPLACEMENTS <<<

JOINT NO.	DISPL.-X	DISPL.-Y	DISPL.-Z	ROTATION-X	ROTATION-Y	ROTATION-Z
34	-.107E-05	-.521E-04	-.871E-19	-.211E-20	-.133E-19	-.376E-05
35	-.125E-05	-.707E-04	-.807E-19	-.121E-20	-.900E-20	-.650E-05
36	-.132E-05	-.948E-04	-.102E-19	-.661E-21	-.833E-20	-.618E-05
37	-.130E-05	-.112E-03	.519E-20	-.822E-22	-.809E-20	-.472E-05
38	-.153E-05	-.123E-03	.303E-19	.709E-21	-.793E-20	-.129E-05
39	-.168E-05	-.124E-03	.531E-19	.117E-20	-.703E-20	.370E-06
40	-.168E-05	-.121E-03	.706E-19	.168E-20	-.776E-20	.229E-05
41	-.178E-05	-.112E-03	.930E-19	.227E-20	-.788E-20	.445E-05
42	-.186E-05	-.828E-04	.102E-18	.327E-20	-.951E-20	.531E-05
43	-.198E-05	-.732E-04	.168E-18	.369E-20	-.794E-20	.445E-05
44	-.237E-05	-.629E-04	.177E-18	.835E-20	-.297E-21	.478E-06

>>> FORCES <<<

SEG.NO.		FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
1	END-A	.265E+01	.502E+02	.328E-14	-.393E-13	.319E-13	.807E+03
	END-B	-.265E+01	-.502E+02	-.328E-14	.393E-13	-.458E-13	-.819E+02
2	END-A	.265E+01	.502E+02	.328E-14	-.393E-13	.459E-13	.819E+02
	END-B	-.265E+01	-.502E+02	-.328E-14	.393E-13	-.544E-13	.107E+03
3	END-A	.265E+01	.216E+02	.328E-14	-.393E-13	.544E-13	-.107E+03
	END-B	-.265E+01	-.216E+02	-.328E-14	.393E-13	-.636E-13	.167E+03
4	END-A	.265E+01	.216E+02	.328E-14	-.393E-13	.636E-13	-.167E+03
	END-B	-.265E+01	-.216E+02	-.328E-14	.393E-13	-.773E-13	.250E+03
5	END-A	.265E+01	-.130E+02	.328E-14	-.393E-13	.773E-13	-.250E+03
	END-B	-.265E+01	.130E+02	-.328E-14	.393E-13	-.831E-13	.235E+03
6	END-A	.265E+01	-.130E+02	.328E-14	-.393E-13	.831E-13	-.235E+03
	END-B	-.265E+01	.130E+02	-.328E-14	.393E-13	-.908E-13	.208E+03
7	END-A	.265E+01	-.130E+02	.328E-14	-.393E-13	.908E-13	-.208E+03
	END-B	-.265E+01	.130E+02	-.328E-14	.393E-13	-.100E-12	.167E+03
8	END-A	.265E+01	-.476E+02	.328E-14	-.393E-13	.100E-12	-.167E+03
	END-B	-.265E+01	.476E+02	-.328E-14	.393E-13	-.117E-12	-.732E+02

SEG.NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
9						
END-A	.205E+01	-.476E+02	.320E-14	-.353E-13	.117E-12	.732E+02
END-B	-.205E+01	.476E+02	-.320E-14	.353E-13	-.123E-12	-.166E+03

SEG.NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
10						
END-A	.205E+01	-.822E+02	.320E-14	-.353E-13	.123E-12	.166E+03
END-B	-.205E+01	.822E+02	-.320E-14	.353E-13	-.135E-12	-.454E+03

MEMBER 5
****

LOAD INCREMENT 1
****

*** DISPLACEMENTS ***

JOINT NO.	DISPL.-X	DISPL.-Y	DISPL.-Z	ROTATION-X	ROTATION-Y	ROTATION-Z
45	-.237E-05	-.629E-04	.177E-18	.635E-20	-.297E-21	.678E-06
46	-.278E-05	-.772E-04	.162E-18	.364E-20	.768E-20	-.375E-05
47	-.298E-05	-.784E-04	.144E-18	.324E-20	.846E-22	-.476E-05
48	-.301E-05	-.104E-03	.956E-19	.223E-20	.858E-20	-.449E-05
49	-.318E-05	-.115E-03	.701E-19	.159E-20	.845E-20	-.271E-05
50	-.318E-05	-.119E-03	.512E-19	.111E-20	.842E-20	-.927E-06
51	-.326E-05	-.128E-03	.364E-19	.731E-21	.848E-20	.462E-06
52	-.334E-05	-.111E-03	.229E-21	-.167E-21	.887E-20	.379E-05
53	-.303E-05	-.975E-04	-.238E-19	-.757E-21	.844E-20	.546E-05
54	-.349E-05	-.419E-04	-.468E-19	-.131E-20	.803E-20	.488E-05
55	-.365E-05	-.575E-04	-.902E-19	-.229E-20	.129E-19	.305E-05

*** FORCES ***

SEG.NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
1						

END-A	.200E+01	.692E+02	-.336E-10	.360E-13	.139E-12	.450E+03
END-B	-.200E+01	-.692E+02	.336E-10	-.360E-13	-.124E-12	-.200E+03

SEG.NO. 2	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.200E+01	.692E+02	-.336E-10	.360E-13	.124E-12	.200E+03
END-B	-.200E+01	-.692E+02	.336E-10	-.360E-13	-.117E-12	-.727E+02

SEG.NO. 3	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.200E+01	.340E+02	-.336E-10	.360E-13	.117E-12	.727E+02
END-B	-.200E+01	-.340E+02	.336E-10	-.360E-13	-.100E-12	.102E+03

SEG.NO. 4	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.200E+01	.340E+02	-.336E-10	.360E-13	.100E-12	-.102E+03
END-B	-.200E+01	-.340E+02	.336E-10	-.360E-13	-.908E-13	.200E+03

SEG.NO. 5	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.200E+01	-.100E-01	-.336E-10	.360E-13	.908E-13	-.200E+03
END-B	-.200E+01	.100E-01	.336E-10	-.360E-13	-.825E-13	.200E+03

SEG.NO. 6	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.200E+01	-.100E-01	-.336E-10	.360E-13	.825E-13	-.200E+03
END-B	-.200E+01	.100E-01	.336E-10	-.360E-13	-.760E-13	.200E+03

SEG.NO. 7	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.200E+01	-.100E-01	-.336E-10	.360E-13	.760E-13	-.200E+03
END-B	-.200E+01	.100E-01	.336E-10	-.360E-13	-.625E-13	.200E+03

SEG.NO. 8	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.200E+01	-.340E+02	-.336E-10	.360E-13	.625E-13	-.200E+03
END-B	-.200E+01	.340E+02	.336E-10	-.360E-13	-.531E-13	.100E+03

SEG.NO. 9	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.200E+01	-.340E+02	-.336E-10	.360E-13	.531E-13	-.100E+03
END-B	-.200E+01	.340E+02	.336E-10	-.360E-13	-.442E-13	.109E+02

SEG.NO. 10	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.200E+01	-.692E+02	-.336E-10	.360E-13	.442E-13	-.109E+02
END-B	-.200E+01	.692E+02	.336E-10	-.360E-13	-.290E-13	-.205E+03

*** LOAD CASE 2 ***

NUMBER OF LOAD INCREMENTS= 46
NUMBER OF MEMBERS LOADED = 1
PRINTING INDEX = 12

LOADED NO. 4

JOINT NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
3	0.	-.175E+02	0.	0.	0.	0.
5	0.	-.175E+02	0.	0.	0.	0.
8	0.	-.175E+02	0.	0.	0.	0.
10	0.	-.175E+02	0.	0.	0.	0.

NUMBER OF FRAME JOINTS LOADED= 1

JOINT NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
6	0.	-.225E+02	0.	0.	0.	-.700E+02

MEMBER 1
****

LOAD INCREMENT 1
****

>>> DISPLACEMENTS <<<

JOINT NO.	DISPL.-X	DISPL.-Y	DISPL.-Z	ROTATION-X	ROTATION-Y	ROTATION-Z
1	0.	0.	0.	0.	0.	0.
2	-.867E-04	-.146E-03	-.288E-18	-.161E-18	-.114E-18	.664E-04
3	-.317E-03	-.292E-03	-.779E-18	-.207E-18	-.227E-18	.117E-03
4	-.677E-03	-.497E-03	-.173E-17	-.422E-18	-.357E-18	.155E-03
5	-.111E-02	-.622E-03	-.294E-17	-.916E-18	-.487E-18	.174E-03
6	-.157E-02	-.787E-03	-.439E-17	-.574E-18	-.617E-18	.173E-03
7	-.208E-02	-.951E-03	-.593E-17	-.685E-18	-.747E-18	.151E-03
8	-.230E-02	-.112E-02	-.751E-17	-.608E-18	-.877E-18	.110E-03
9	-.256E-02	-.128E-02	-.903E-17	-.564E-18	-.101E-17	.487E-04
10	-.258E-02	-.145E-02	-.106E-16	-.408E-18	-.114E-17	-.320E-04
11	-.237E-02	-.161E-02	-.116E-16	-.308E-18	-.129E-17	-.137E-03

>>> FORCES <<<

SEQ.NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
1						

SEG.NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.219E+03	.302E+04	.390E-12	.973E-11	.296E-11	-.242E+04
END-B	-.219E+03	-.302E+04	-.390E-12	-.487E-11	-.296E-11	.190E+04

SEG.NO. 2	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.219E+03	.302E+04	.390E-12	.487E-11	.296E-11	-.190E+04
END-B	-.219E+03	-.302E+04	-.390E-12	-.400E-11	-.296E-11	.137E+04

SEG.NO. 3	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.219E+03	.302E+04	.390E-12	.400E-11	.296E-11	-.137E+04
END-B	-.219E+03	-.302E+04	-.390E-12	-.309E-11	-.296E-11	.405E+03

SEG.NO. 4	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.219E+03	.302E+04	.390E-12	.309E-11	.296E-11	-.405E+03
END-B	-.219E+03	-.302E+04	-.390E-12	-.217E-11	-.296E-11	.244E+03

SEG.NO. 5	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.219E+03	.302E+04	.390E-12	.217E-11	.296E-11	-.244E+03
END-B	-.219E+03	-.302E+04	-.390E-12	-.126E-11	-.296E-11	-.317E+03

SEG.NO. 6	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.219E+03	.302E+04	.390E-12	.126E-11	.296E-11	.317E+03
END-B	-.219E+03	-.302E+04	-.390E-12	-.343E-12	-.296E-11	-.870E+03

SEG.NO. 7	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.219E+03	.302E+04	.390E-12	.343E-12	.296E-11	.870E+03
END-B	-.219E+03	-.302E+04	-.390E-12	.572E-12	-.296E-11	-.144E+04

SEG.NO. 8	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.219E+03	.302E+04	.390E-12	-.572E-12	.296E-11	.144E+04
END-B	-.219E+03	-.302E+04	-.390E-12	.149E-11	-.296E-11	-.200E+04

SEG.NO. 9	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.219E+03	.302E+04	.390E-12	-.149E-11	.296E-11	.200E+04
END-B	-.219E+03	-.302E+04	-.390E-12	.200E-11	-.296E-11	-.256E+04

SEG.NO. 10	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.219E+03	.302E+04	.390E-12	-.200E-11	.296E-11	.256E+04
END-B	-.219E+03	-.302E+04	-.390E-12	.332E-11	-.296E-11	-.312E+04

MEMBER 2

LOAD INCREMENT 1

>>> DISPLACEMENTS <<<

JOINT NO.	DISPL.-X	DISPL.-Y	DISPL.-Z	ROTATION-X	ROTATION-Y	ROTATION-Z
12	0.	0.	0.	0.	0.	0.
13	.577E-05	-.173E-03	.410E-18	.320E-18	.932E-20	-.426E-05
14	.149E-04	-.347E-03	.157E-17	.608E-18	.106E-19	-.608E-05
15	.394E-04	-.543E-03	.348E-17	.651E-18	.294E-19	-.769E-05
16	.585E-04	-.739E-03	.594E-17	.109E-17	.402E-19	-.646E-05
17	.717E-04	-.935E-03	.844E-17	.116E-17	.510E-19	-.312E-05
18	.734E-04	-.113E-02	.129E-16	.122E-17	.618E-19	.234E-05
19	.581E-04	-.133E-02	.151E-16	.121E-17	.726E-19	.991E-05
20	.205E-04	-.152E-02	.183E-16	.119E-17	.834E-19	.196E-04
21	-.451E-04	-.172E-02	.216E-16	.108E-17	.942E-19	.310E-04
22	-.144E-03	-.191E-02	.234E-16	.881E-18	.105E-18	.453E-04

>>> FORCES <<<

SEG.NO.		FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
1	END-A	-.226E+02	.453E+04	-.698E-12	-.114E-10	-.244E-12	.108E+03
	END-B	.226E+02	-.453E+04	.698E-12	.972E-11	.244E-12	-.110E+03
2	END-A	-.226E+02	.453E+04	-.698E-12	-.972E-11	-.244E-12	.110E+03
	END-B	.226E+02	-.453E+04	.698E-12	.808E-11	.244E-12	-.542E+02
3	END-A	-.226E+02	.453E+04	-.698E-12	-.808E-11	-.244E-12	.542E+02
	END-B	.226E+02	-.453E+04	.698E-12	.617E-11	.244E-12	.472E+01
4	END-A	-.226E+02	.453E+04	-.698E-12	-.617E-11	-.244E-12	-.472E+01
	END-B	.226E+02	-.453E+04	.698E-12	.435E-11	.244E-12	.637E+02
5	END-A	-.226E+02	.453E+04	-.698E-12	-.435E-11	-.244E-12	-.637E+02
	END-B	.226E+02	-.453E+04	.698E-12	.252E-11	.244E-12	.123E+03
6	END-A	-.226E+02	.453E+04	-.698E-12	-.252E-11	-.244E-12	-.123E+03
	END-B	.226E+02	-.453E+04	.698E-12	.698E-12	.244E-12	.181E+03
7	END-A	-.226E+02	.453E+04	-.698E-12	-.698E-12	-.244E-12	-.181E+03
	END-B	.226E+02	-.453E+04	.698E-12	-.113E-11	.244E-12	.240E+03
8	END-A	-.226E+02	.453E+04	-.698E-12	.113E-11	-.244E-12	-.240E+03
	END-B	.226E+02	-.453E+04	.698E-12	-.296E-11	.244E-12	.299E+03

SEG.NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
9						
END-A	-.226E+02	.453E+04	-.698E-12	.290E-11	-.244E-12	-.299E+03
END-B	.226E+02	-.453E+04	.698E-12	-.478E-11	.244E-12	.350E+03

SEG.NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
10						
END-A	-.226E+02	.453E+04	-.698E-12	.478E-11	-.244E-12	-.350E+03
END-B	.226E+02	-.453E+04	.698E-12	-.661E-11	.244E-12	.410E+03

MEMBER 3
****

LOAD INCREMENT 1
****

\>\>\> DISPLACEMENTS <<<

JOINT NO.	DISPL.-X	DISPL.-Y	DISPL.-Z	ROTATION-X	ROTATION-Y	ROTATION-Z
23	0.	0.	0.	0.	0.	0.
24	.752E-04	-.161E-03	-.295E-18	-.161E-18	.106E-18	-.575E-04
25	.274E-03	-.322E-03	-.773E-18	-.205E-18	.213E-18	-.100E-03
26	.503E-03	-.504E-03	-.171E-17	-.419E-18	.336E-18	-.133E-03
27	.952E-03	-.606E-03	-.203E-17	-.511E-18	.459E-18	-.107E-03
28	.134E-02	-.807E-03	-.435E-17	-.572E-18	.502E-18	-.143E-03
29	.169E-02	-.139E-02	-.580E-17	-.600E-18	.705E-18	-.122E-03
30	.196E-02	-.123E-02	-.745E-17	-.946E-18	.828E-18	-.822E-04
31	.210E-02	-.141E-02	-.806E-17	-.560E-18	.951E-18	-.207E-04
32	.227E-02	-.160E-02	-.183E-16	-.402E-18	.180E-17	.580E-04
33	.183E-02	-.178E-02	-.115E-16	-.349E-18	.121E-17	.140E-03

\>\>\> FORCES <<<

SEG.NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
1						

SEG. NO.		FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
	END-A	-.192E+03	.420E+04	.347E-12	.549E-11	-.279E-11	.218E+04
	END-B	.192E+03	-.420E+04	-.347E-12	-.484E-11	.279E-11	-.163E+04
2	END-A	-.192E+03	.420E+04	.347E-12	.484E-11	-.279E-11	.163E+04
	END-B	.192E+03	-.420E+04	-.347E-12	-.308E-11	.279E-11	-.115E+04
3	END-A	-.192E+03	.420E+04	.347E-12	.308E-11	-.279E-11	.115E+04
	END-B	.192E+03	-.420E+04	-.347E-12	-.307E-11	.279E-11	-.651E+03
4	END-A	-.192E+03	.420E+04	.347E-12	.307E-11	-.279E-11	.651E+03
	END-B	.192E+03	-.420E+04	-.347E-12	-.216E-11	.279E-11	-.149E+03
5	END-A	-.192E+03	.420E+04	.347E-12	.216E-11	-.279E-11	.149E+03
	END-B	.192E+03	-.420E+04	-.347E-12	-.129E-11	.279E-11	.353E+03
6	END-A	-.192E+03	.420E+04	.347E-12	.129E-11	-.279E-11	-.353E+03
	END-B	.192E+03	-.420E+04	-.347E-12	-.309E-12	.279E-11	.854E+03
7	END-A	-.192E+03	.420E+04	.347E-12	.309E-12	-.279E-11	-.854E+03
	END-B	.192E+03	-.420E+04	-.347E-12	.569E-12	.279E-11	.136E+04
8	END-A	-.192E+03	.420E+04	.347E-12	-.569E-12	-.279E-11	-.136E+04
	END-B	.192E+03	-.420E+04	-.347E-12	.147E-11	.279E-11	.186E+04
9	END-A	-.192E+03	.420E+04	.347E-12	-.147E-11	-.279E-11	-.186E+04
	END-B	.192E+03	-.420E+04	-.347E-12	.238E-11	.279E-11	.236E+04
10	END-A	-.192E+03	.420E+04	.347E-12	-.238E-11	-.279E-11	-.236E+04
	END-B	.192E+03	-.420E+04	-.347E-12	.329E-11	.279E-11	.286E+04

MEMBER 8
****

LOAD INCREMENT 1
****

*** DISPLACEMENTS ***

JOINT NO.	DISPL.-X	DISPL.-Y	DISPL.-Z	ROTATION-X	ROTATION-Y	ROTATION-Z
34	-.237E-02	-.101E-02	-.116E-16	-.309E-18	-.129E-17	-.137E-03
35	-.191E-02	-.323E-02	-.512E-17	-.214E-18	-.159E-17	-.494E-03
36	-.145E-02	-.460E-02	-.476E-18	-.113E-18	-.174E-17	-.503E-03
37	-.170E-02	-.569E-02	.429E-17	.297E-20	-.109E-17	-.393E-03
38	-.131E-02	-.781E-02	.114E-16	.170E-18	-.186E-17	-.103E-03
39	-.111E-02	-.790E-02	.120E-16	.252E-18	-.907E-18	.469E-04
40	-.800E-03	-.676E-02	.187E-16	.340E-18	-.723E-18	.212E-03
41	-.450E-03	-.593E-02	.160E-16	.471E-18	-.792E-18	.309E-03
42	-.511E-03	-.349E-02	.203E-16	.610E-18	-.774E-18	.426E-03
43	-.435E-03	-.272E-02	.217E-16	.689E-18	-.692E-18	.348E-03
44	-.144E-03	-.191E-02	.254E-16	.801E-18	.105E-18	.493E-04

*** FORCES ***

SEG.NO.		FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
1	END-A	.219E+03	.174E+04	.359E-12	-.332E-11	.296E-11	.104E+05
	END-B	-.219E+03	-.174E+04	-.359E-12	.332E-11	-.488E-11	-.200E+04
2	END-A	.219E+03	.174E+04	.359E-12	-.332E-11	.488E-11	.200E+04
	END-B	-.219E+03	-.174E+04	-.359E-12	.332E-11	-.541E-11	.100E+04
3	END-A	.219E+03	.609E+03	.359E-12	-.332E-11	.541E-11	-.100E+04
	END-B	-.219E+03	-.609E+03	-.359E-12	.332E-11	-.639E-11	.373E+04
4	END-A	.219E+03	.609E+03	.359E-12	-.332E-11	.639E-11	-.373E+04
	END-B	-.219E+03	-.609E+03	-.359E-12	.332E-11	-.786E-11	.660E+04
5	END-A	.219E+03	-.379E+03	.359E-12	-.332E-11	.786E-11	-.660E+04
	END-B	-.219E+03	.379E+03	-.359E-12	.332E-11	-.847E-11	.595E+04
6	END-A	.219E+03	-.379E+03	.359E-12	-.332E-11	.847E-11	-.595E+04
	END-B	-.219E+03	.379E+03	-.359E-12	.332E-11	-.926E-11	.512E+04
7	END-A	.219E+03	-.379E+03	.359E-12	-.332E-11	.926E-11	-.512E+04
	END-B	-.219E+03	.379E+03	-.359E-12	.332E-11	-.103E-10	.400E+04
8	END-A	.219E+03	-.103E+04	.359E-12	-.332E-11	.103E-10	-.400E+04
	END-B	-.219E+03	.103E+04	-.359E-12	.332E-11	-.121E-10	.320E+04

BEG.NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
8						
END-A	.215E+03	-.143E+04	.390E-12	-.332E-11	.121E-10	.320E+04
END-B	-.215E+03	.143E+04	-.390E-12	.332E-11	-.120E-10	-.590E+04

BEG.NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
12						
END-A	.215E+03	-.240E+04	.390E-12	-.332E-11	.120E-10	.900E+00
END-B	-.215E+03	.240E+04	-.359E-12	.332E-11	-.140E-10	-.107E+05

MEMBER 5
****

LOAD INCREMENT 1
****

>>> DISPLACEMENTS <<<

JOINT NO.	DISPL.-X	DISPL.-Y	DISPL.-Z	ROTATION-X	ROTATION-Y	ROTATION-Z
45	-.144E-03	-.191E-02	.230E-16	.081E-18	.105E-10	.053E-04
46	.162E-03	-.241E-02	.200E-16	.605E-18	.111E-17	-.270E-03
47	.245E-03	-.303E-02	.180E-16	.019E-10	.123E-17	-.355E-03
48	.345E-03	-.404E-02	.120E-16	.404E-14	.915E-10	-.304E-03
49	.533E-03	-.406E-02	.104E-16	.320E-10	.032E-10	-.222E-03
50	.721E-03	-.630E-02	.059E-17	.227E-10	.006E-10	-.705E-04
51	.470E-03	-.634E-02	.097E-17	.159E-10	.102E-17	.304E-04
52	.172E-02	-.507E-02	.210E-17	-.207E-19	.131E-17	.301E-03
53	.141E-02	-.455E-02	-.180E-17	-.130E-10	.145E-17	.033E-03
54	.100E-02	-.334E-02	-.590E-17	-.210E-18	.139E-17	.400E-03
55	.103E-02	-.170E-02	-.115E-10	-.309E-10	.121E-17	.140E-03

>>> FORCES <<<

BEG.NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
1						

	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.192E+03	.245E+00	-.347E-12	.329E-11	.137E-10	.142E+05
END-B	-.192E+03	-.205E+00	.347E-12	-.329E-11	-.125E-10	-.788E+04

SEG.NO. 2	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.192E+03	.205E+00	-.347E-12	.329E-11	.125E-10	.788E+04
END-B	-.192E+03	-.205E+00	.347E-12	-.329E-11	-.110E-10	-.309E+04

SEG.NO. 3	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.192E+03	.131E+00	-.347E-12	.329E-11	.110E-10	.309E+04
END-B	-.192E+03	-.101E+00	.347E-12	-.329E-11	-.101E-10	.281E+04

SEG.NO. 4	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.192E+03	.101E+00	-.347E-12	.329E-11	.101E-10	-.281E+04
END-B	-.192E+03	-.101E+00	.347E-12	-.329E-11	-.905E-11	.503E+04

SEG.NO. 5	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.192E+03	-.293E+02	-.347E-12	.329E-11	.905E-11	-.503E+04
END-B	-.192E+03	.293E+02	.347E-12	-.329E-11	-.827E-11	.697E+04

SEG.NO. 6	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.192E+03	-.293E+02	-.347E-12	.329E-11	.827E-11	-.697E+04
END-B	-.192E+03	.293E+02	.347E-12	-.329E-11	-.766E-11	.891E+04

SEG.NO. 7	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.192E+03	-.293E+02	-.347E-12	.329E-11	.766E-11	-.891E+04
END-B	-.192E+03	.293E+02	.347E-12	-.329E-11	-.682E-11	.879E+04

SEG.NO. 8	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.192E+03	-.187E+00	-.347E-12	.329E-11	.682E-11	-.879E+04
END-B	-.192E+03	.187E+00	.347E-12	-.329E-11	-.523E-11	.179E+04

SEG.NO. 9	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.192E+03	-.187E+00	-.347E-12	.329E-11	.523E-11	-.179E+04
END-B	-.192E+03	.187E+00	.347E-12	-.329E-11	-.831E-11	-.102E+04

SEG.NO. 10	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	.192E+03	-.211E+00	-.347E-12	.329E-11	.431E-11	.102E+04
END-B	-.192E+03	.211E+00	.347E-12	-.329E-11	-.270E-11	-.182E+05

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THE PIER-BENT HAD A STEEL TENSION FAILURE
*****************************************

   IN MEMBER      4
   IN SECTION     3
   OF SEGMENT     4


   IN LOAD INCREMENT    42
   OF LOAD CASE     2


       JUST BEFORE FAILURE
       *******************

MAX. COMPRESSIVE CONCRETE STRAIN= -.957E-03
OCCURED IN LOAD CASE             =     2
OCCURED IN INCREMENT             =    42
OCCURED IN MEMBER                =     4
OCCURED IN SEGMENT               =     4
OCCURED IN SECTION               =     3

MAX. TENSILE STEEL STRAIN=  .961E-02
OCCURED IN LOAD CASE       =     2
OCCURED IN INCREMENT       =    41
OCCURED IN MEMBER          =     4
OCCURED IN SEGMENT         =     4
OCCURED IN SECTION         =     3
```

** FORCES AND DISPLACEMENTS PREVIOUS TO FAILURE **

MEMBER 1

LOAD INCREMENT 91

>>> DISPLACEMENTS <<<

JOINT NO.	DISPL.-X	DISPL.-Y	DISPL.-Z	ROTATION-X	ROTATION-Y	ROTATION-Z
1	0.	0.	0.	0.	0.	0.
2	-.972E-03	-.144E-03	.470E-16	.350E-18	-.175E-17	.426E-03
3	-.290E-02	-.352E-03	.167E-17	.503E-18	-.330E-17	.703E-03
4	-.411E-02	-.620E-03	.388E-17	.740E-18	-.488E-17	.869E-03
5	-.650E-02	-.894E-03	.574E-17	.949E-18	-.637E-17	.970E-03
6	-.912E-02	-.117E-02	.830E-17	.108E-17	-.786E-17	.958E-03
7	-.115[illegible]	-.144E-02	.113E-16	.118E-17	-.948E-17	.854E-03
8	-.135E-01	-.170E-02	.145E-16	.127E-17	-.112E-16	.633E-03
9	-.147E-01	-.189E-02	.179E-16	.131E-17	-.134E-16	.225E-03
10	-.145E-01	-.191E-02	.213E-16	.127E-17	-.160E-16	-.452E-03
11	-.121E-01	-.187E-02	.245E-16	.111E-17	-.189E-16	-.148E-02

>>> FORCES <<<

SEG.NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
1						

SEG.NO.		FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
	END-A	.974E+03	.624E+04	-.276E-12	-.671E-11	.329E-10	-.149E+05
	END-B	-.974E+03	-.624E+04	.276E-12	.684E-11	-.329E-10	.446E+04

SEG.NO.		FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
2	END-A	.974E+03	.624E+04	-.276E-12	-.684E-11	.329E-10	-.446E+04
	END-B	-.974E+03	-.624E+04	.276E-12	.537E-11	-.329E-10	.685E+04

SEG.NO.		FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
3	END-A	.974E+03	.624E+04	-.276E-12	-.537E-11	.329E-10	-.685E+04
	END-B	-.974E+03	-.624E+04	.276E-12	.464E-11	-.329E-10	.358E+04

SEG.NO.		FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
4	END-A	.974E+03	.624E+04	-.276E-12	-.464E-11	.329E-10	-.358E+04
	END-B	-.974E+03	-.624E+04	.276E-12	.391E-11	-.329E-10	.955E+03

SEG.NO.		FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
5	END-A	.974E+03	.624E+04	-.276E-12	-.391E-11	.329E-10	-.955E+03
	END-B	-.974E+03	-.624E+04	.276E-12	.318E-11	-.329E-10	-.159E+04

SEG.NO.		FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
6	END-A	.974E+03	.624E+04	-.276E-12	-.318E-11	.329E-10	.159E+04
	END-B	-.974E+03	-.624E+04	.276E-12	.244E-11	-.329E-10	-.414E+04

SEG.NO.		FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
7	END-A	.974E+03	.624E+04	-.276E-12	-.244E-11	.329E-10	.414E+04
	END-B	-.974E+03	-.624E+04	.276E-12	.170E-11	-.329E-10	-.669E+04

SEG.NO.		FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
8	END-A	.974E+03	.624E+04	-.276E-12	-.170E-11	.329E-1[illegible]	.669E+04
	END-B	-.974E+03	-.624E+04	.276E-12	.942E-12	-.329E-10	-.923E+04

SEG.NO.		FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
9	END-A	.974E+03	.624E+04	-.276E-12	-.942E-12	.329E-10	.923E+04
	END-B	-.974E+03	-.624E+04	.276E-12	.181E-12	-.329E-10	-.118E+05

SEG.NO.		FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
10	END-A	.974E+03	.624E+04	-.276E-12	-.181E-12	.329E-10	.118E+05
	END-B	-.974E+03	-.624E+04	.276E-12	-.547E-12	-.329E-10	-.143E+05

MEMBER 2

LOAD INCREMENT 91

>>> DISPLACEMENTS <<<

JOINT NO.	DISPL.-X	DISPL.-Y	DISPL.-Z	ROTATION-X	ROTATION-Y	ROTATION-Z
12	0.	0.	0.	0.	0.	0.
13	.443E-03	-.312E-03	.184E-18	.155E-18	-.520E-20	-.329E-03
14	.155E-02	-.607E-03	.771E-18	.317E-18	.650E-20	-.548E-03
15	.321E-02	-.103E-02	.181E-17	.471E-18	.297E-19	-.704E-03
16	.515E-02	-.141E-02	.319E-17	.581E-18	.536E-19	-.768E-03
17	.714E-02	-.179E-02	.480E-17	40E-18	.772E-19	-.780E-03
18	.904E-02	-.217E-02	.653E-17	.669E-18	.101E-18	-.620E-03
19	.103E-01	-.255E-02	.825E-17	.608E-18	.114E-18	-.403E-03
20	.109E-01	-.291E-02	.988E-17	.508E-18	.101E-18	-.494E-04
21	.104E-01	-.319E-02	.114E-16	.551E-18	.685E-19	.403E-03
22	.922E-02	-.334E-02	.128E-16	.555E-18	.143E-19	.129E-02

>>> FORCES <<<

SEG.NO.		FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
1	END-A	-.947E+03	.862E+00	-.802E-12	-.749E-11	-.604E-12	.101E+05
	END-B	.947E+03	-.862E+00	.802E-12	.631E-11	.603E-12	-.778E+03
2	END-A	-.947E+03	.862E+00	-.802E-12	-.631E-11	-.603E-12	.778E+03
	END-B	.947E+03	-.862E+00	.802E-12	.512E-11	.601E-12	-.508E+03
3	END-A	-.947E+03	.862E+00	-.802E-12	-.512E-11	-.601E-12	.508E+03
	END-B	.947E+03	-.862E+00	.802E-12	.386E-11	.597E-12	-.297E+03
4	END-A	-.947E+03	.862E+00	-.802E-12	-.386E-11	-.597E-12	.297E+03
	END-B	.947E+03	-.862E+00	.802E-12	.259E-11	.592E-12	-.086E+03
5	END-A	-.947E+03	.862E+00	-.802E-12	-.259E-11	-.592E-12	.086E+03
	END-B	.947E+03	-.862E+00	.802E-12	.133E-11	.587E-12	.199E+00
6	END-A	-.947E+03	.862E+00	-.802E-12	-.133E-11	-.587E-12	-.199E+00
	END-B	.947E+03	-.862E+00	.802E-12	.596E-13	.581E-12	.407E+00
7	END-A	-.947E+03	.862E+00	-.802E-12	-.596E-13	-.581E-12	-.407E+00
	END-B	.947E+03	-.862E+00	.802E-12	-.121E-11	.576E-12	.694E+00
8	END-A	-.947E+03	.862E+00	-.802E-12	.121E-11	-.576E-12	-.694E+00
	END-B	.947E+03	-.862E+00	.802E-12	-.207E-11	.571E-12	.941E+00

SEG.NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
9						
END-A	-.947E+03	.802E+04	-.882E-12	.247E-11	-.571E-12	-.941E+04
END-B	.947E+03	-.802E+04	.882E-12	-.373E-11	.507E-12	.119E+05

SEG.NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
10						
END-A	-.947E+03	.802E+04	-.882E-12	.373E-11	-.507E-12	-.119E+05
END-B	.947E+03	-.802E+04	.882E-12	-.588E-11	.565E-12	.143E+05

MEMBER 3
****

LOAD INCREMENT 1
****

>>> DISPLACEMENTS <<<

JOINT NO.	DISPL.-X	DISPL.-Y	DISPL.-Z	ROTATION-X	ROTATION-Y	ROTATION-Z
23	0.	0.	0.	0.	0.	0.
24	.181E-03	-.232E-03	-.520E-18	-.429E-18	-.984E-20	-.817E-04
25	.481E-03	-.464E-03	-.147E-17	-.758E-18	-.108E-19	-.161E-03
26	.938E-03	-.727E-03	-.440E-17	-.109E-17	-.291E-19	-.251E-03
27	.171E-02	-.990E-03	-.761E-17	-.136E-17	-.481E-19	-.339E-03
28	.274E-02	-.125E-02	-.114E-16	-.155E-17	-.512E-19	-.423E-03
29	.394E-02	-.152E-02	-.196E-16	-.167E-17	-.622E-19	-.505E-03
30	.512E-02	-.178E-02	-.201E-16	-.172E-17	-.734E-19	-.584E-03
31	.694E-02	-.204E-02	-.245E-16	-.170E-17	-.846E-19	-.660E-03
32	.875E-02	-.230E-02	-.280E-16	-.161E-17	-.958E-19	-.733E-03
33	.108E-01	-.257E-02	-.329E-16	-.148E-17	-.975E-19	-.804E-03

>>> FORCES <<<

SEG.NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
1						

END-A	-.272E+02	.681E+04	.757E-12	.141E-10	.108E-12	.265E+04
END-B	.272E+02	-.681E+04	-.757E-12	-.122E-10	-.108E-12	-.258E+04

SEG.NO. 2	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.272E+02	.681E+04	.757E-12	.122E-10	.108E-12	.258E+04
END-B	.272E+02	-.681E+04	-.757E-12	-.103E-10	-.108E-12	-.251E+04

SEG.NO. 3	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.272E+02	.681E+04	.757E-12	.103E-10	.108E-12	.251E+04
END-B	.272E+02	-.681E+04	-.757E-12	-.830E-11	-.109E-12	-.244E+04

SEG.NO. 4	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.272E+02	.681E+04	.757E-12	.830E-11	.109E-12	.244E+04
END-B	.272E+02	-.681E+04	-.757E-12	-.638E-11	-.109E-12	-.236E+04

SEG.NO. 5	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.272E+02	.681E+04	.757E-12	.638E-11	.109E-12	.236E+04
END-B	.272E+02	-.681E+04	-.757E-12	-.439E-11	-.170E-12	-.229E+04

SEG.NO. 6	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.272E+02	.681E+04	.757E-12	.439E-11	.170E-12	.229E+04
END-B	.272E+02	-.681E+04	-.757E-12	-.239E-11	-.170E-12	-.221E+04

SEG.NO. 7	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.272E+02	.681E+04	.757E-12	.239E-11	.170E-12	.221E+04
END-B	.272E+02	-.681E+04	-.757E-12	-.397E-12	-.171E-12	-.213E+04

SEG.NO. 8	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.272E+02	.681E+04	.757E-12	.397E-12	.171E-12	.213E+04
END-B	.272E+02	-.681E+04	-.757E-12	.160E-11	-.171E-12	-.205E+04

SEG.NO. 9	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.272E+02	.681E+04	.757E-12	-.160E-11	.171E-12	.205E+04
END-B	.272E+02	-.681E+04	-.757E-12	.359E-11	-.172E-12	-.197E+04

SEG.NO. 10	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
END-A	-.272E+02	.681E+04	.757E-12	-.359E-11	.172E-12	.197E+04
END-B	.272E+02	-.681E+04	-.757E-12	.559E-11	-.172E-12	-.189E+04

MEMBER 4

LOAD INCREMENT 41

>>> DISPLACEMENTS <<<

JOINT NO.	DISPL.-X	DISPL.-Y	DISPL.-Z	ROTATION-X	ROTATION-Y	ROTATION-Z
34	-.121E-01	-.167E-02	.265E-16	.111E-17	-.109E-16	-.140E-02
35	-.990E-02	-.179E-01	-.226E-16	.104E-17	.986E-17	-.370E-02
36	-.741E-02	-.273E-01	-.476E-16	.102E-17	.905E-17	-.373E-02
37	-.609E-02	-.373E-01	-.722E-16	.988E-18	.878E-17	-.334E-02
38	-.443E-03	-.495E-01	-.106E-15	.859E-19	.135E-17	-.149E-03
39	.378E-02	-.472E-01	-.986E-16	.761E-18	-.822E-17	.105E-02
40	.456E-02	-.423E-01	-.807E-16	.722E-18	-.776E-17	.240E-02
41	.530E-02	-.342E-01	-.581E-16	.673E-18	-.750E-17	.295E-02
42	.564E-02	-.182E-01	-.223E-16	.664E-18	-.733E-17	.309E-02
43	.478E-02	-.124E-01	-.812E-17	.645E-18	-.785E-17	.291E-02
44	.822E-02	-.334E-02	.128E-16	.595E-18	.143E-19	.129E-02

>>> FORCES <<<

SEG.NO.		FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
1	END-A	.974E+03	.416E+04	-.276E-12	.507E-12	.329E-10	.215E+05
	END-B	-.974E+03	-.416E+04	.276E-12	-.507E-12	-.317E-10	-.340E+04
2	END-A	.974E+03	.416E+04	-.276E-12	.507E-12	.317E-10	.340E+04
	END-B	-.974E+03	-.416E+04	.276E-12	-.507E-12	-.310E-10	.759E+04
3	END-A	.974E+03	.153E+04	-.276E-12	.507E-12	.310E-10	-.759E+04
	END-B	-.974E+03	-.153E+04	.276E-12	-.507E-12	-.302E-10	.119E+05
4	END-A	.974E+03	.153E+04	-.276E-12	.507E-12	.302E-10	-.119E+05
	END-B	-.974E+03	-.153E+04	.276E-12	-.507E-12	-.291E-10	.103E+05
5	END-A	.974E+03	-.110E+04	-.276E-12	.507E-12	.291E-10	-.103E+05
	END-B	-.974E+03	.110E+04	.276E-12	-.507E-12	-.286E-10	.164E+05
6	END-A	.974E+03	-.110E+04	-.276E-12	.507E-12	.286E-10	-.164E+05
	END-B	-.974E+03	.110E+04	.276E-12	-.507E-12	-.280E-10	.139E+05
7	END-A	.974E+03	-.110E+04	-.276E-12	.507E-12	.280E-10	-.139E+05
	END-B	-.974E+03	.110E+04	.276E-12	-.507E-12	-.271E-10	.186E+05
8	END-A	.974E+03	-.373E+04	-.276E-12	.507E-12	.271E-10	-.186E+05
	END-B	-.974E+03	.373E+04	.276E-12	-.507E-12	-.257E-10	-.821E+04

SEG.NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
9						
END-A	.974E+03	-.373E+04	-.276E-12	.587E-12	.257E-10	.821E+04
END-B	-.974E+03	.373E+04	.276E-12	-.587E-12	-.252E-10	-.155E+05

SEG.NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
10						
END-A	.974E+03	-.636E+04	-.276E-12	.587E-12	.252E-10	.155E+05
END-B	-.974E+03	.636E+04	.276E-12	-.587E-12	-.242E-10	-.377E+05

MEMBER 5
****

LOAD INCREMENT 91
****

>>> DISPLACEMENTS <<<

JOINT NO.	DISPL.-X	DISPL.-Y	DISPL.-Z	ROTATION-X	ROTATION-Y	ROTATION-Z
45	.822E-02	-.334E-02	.124E-16	.555E-18	.143E-19	.124E-02
46	.892E-02	-.603E-04	.813E-17	.352E-18	.230E-17	.650E-03
47	.916E-02	.951E-03	.291E-17	.263E-18	.293E-17	.418E-03
48	.953E-02	.200E-02	-.135E-16	-.804E-19	.343E-17	.650E-04
49	.979E-02	.168E-02	-.214E-16	-.235E-18	.205E-17	-.977E-04
50	.984E-02	.161E-02	-.257E-16	-.386E-18	.165E-17	-.140E-03
51	.986E-02	.134E-02	-.282E-16	-.510E-18	.127E-17	-.156E-03
52	.984E-02	.660E-03	-.316E-16	-.814E-18	.472E-18	-.159E-03
53	.991E-02	.200E-03	-.324E-16	-.100E-17	.110E-18	-.197E-03
54	.101E-01	-.448E-03	-.326E-16	-.115E-17	.815E-19	-.309E-03
55	.100E-01	-.257E-02	-.329E-16	-.164E-17	-.975E-19	-.804E-03

>>> FORCES <<<

SEG.NO.	FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
1						

SEG.NO.		FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
	END-A	.272E+02	.226E+04	-.797E-12	.550E-11	.237E-10	.234E+05
	END-B	-.272E+02	-.226E+04	.797E-12	-.550E-11	-.210E-10	-.155E+05
2	END-A	.272E+02	.226E+04	-.797E-12	.550E-11	.210E-10	.155E+05
	END-B	-.272E+02	-.226E+04	.797E-12	-.550E-11	-.196E-10	-.111E+05
3	END-A	.272E+02	.123E+04	-.797E-12	.550E-11	.196E-10	.111E+05
	END-B	-.272E+02	-.123E+04	.797E-12	-.550E-11	-.157E-10	-.409E+04
4	END-A	.272E+02	.123E+04	-.797E-12	.550E-11	.157E-10	.409E+04
	END-B	-.272E+02	-.123E+04	.797E-12	-.550E-11	-.135E-10	-.122E+04
5	END-A	.272E+02	.108E+03	-.797E-12	.550E-11	.135E-10	.122E+04
	END-B	-.272E+02	-.108E+03	.797E-12	-.550E-11	-.110E-10	-.796E+03
6	END-A	.272E+02	.108E+03	-.797E-12	.550E-11	.110E-10	.796E+03
	END-B	-.272E+02	-.108E+03	.797E-12	-.550E-11	-.108E-10	-.468E+03
7	END-A	.272E+02	.108E+03	-.797E-12	.550E-11	.104E-10	.468E+03
	END-B	-.272E+02	-.108E+03	.797E-12	-.550E-11	-.726E-11	.310E+03
8	END-A	.272E+02	-.850E+03	-.797E-12	.550E-11	.726E-11	-.310E+03
	END-B	-.272E+02	.850E+03	.797E-12	-.550E-11	-.513E-11	-.207E+04

SEG.NO.		FORCE-X	FORCE-Y	FORCE-Z	MOMENT-X	MOMENT-Y	MOMENT-Z
9	END-A	.272E+02	-.850E+03	-.797E-12	.550E-11	.513E-11	.207E+04
	END-B	-.272E+02	.850E+03	.797E-12	-.550E-11	-.313E-11	-.432E+04
10	END-A	.272E+02	-.109E+04	-.797E-12	.550E-11	.313E-11	.432E+04
	END-B	-.272E+02	.109E+04	.797E-12	-.550E-11	.172E-12	-.125E+0[illegible]

A P P E N D I X   D

QUESTIONNAIRE CONCERNING SLENDER CONCRETE BRIDGE PIERS

Introduction

The Civil Engineering Structures Research Laboratory at The University of Texas at Austin is currently engaged in a project entitled "The Design of Tall Nonprismatic or Hollow Concrete Bridge Piers," under the joint sponsorship of the Federal Highway Administration and the Texas State Department of Highways and Public Transportation.

This project's objective is to develop design criteria suitable for incorporation into future editions of the AASHTO Specification for concrete bridge piers falling into one of the classifications given in a subsequent section of the introduction.

An initial phase of the study is the gathering of data concerning usage and design considerations for the pier types covered along with input on dimensions, detailing, and construction practices. This questionnaire seeks to document several areas in the state-of-the-art for concrete bridge piers and your view of possible trends over the next decade.

Pier Types to Be Considered--The scope of interest for this project includes any concrete bridge pier that falls into any one of the following classifications:

(a) Its height-to-depth ratio (L/D) is greater than 10.
(b) Its height-to-width ratio (L/W) is greater than 10.
(c) Its ratio of height to radius of gyration about either axis (L/r) is greater than 20.

Furthermore, the pier may have any of the following characteristics:

(a) The pier may have constant or variable cross-sectional dimensions along its height.
(b) The pier cross section may be solid or it may be hollow; i.e., contain one or more voids.

Both prestressed and reinforced concrete bridge piers meeting the above criteria and supporting any type of timber, steel, or concrete superstructure are within the scope of this questionnaire.

Questionnaire Format--To simplify reponses, most questions are asked in terms of a percentage of total usage for given subject areas. Note that in some cases this percentage of usage is broken down into two time periods, one extending from 1960 to 1979, and the other covering an estimate of future trends during the next ten years. Extreme precision is not required in your percentage estimates for these questions, as their sole purpose is to obtain a general survey of recent past, present, and future trends in the design of tall concrete bridge piers.

Those questions that deal with detailing practices and recommendations, however, do require accurate responses to be of maximum use to the study.

Some of the questions asked may not be applicable to your organization's usage of concrete bridge piers, and wherever this is the case, please indicate by writing "N.A." through that question.

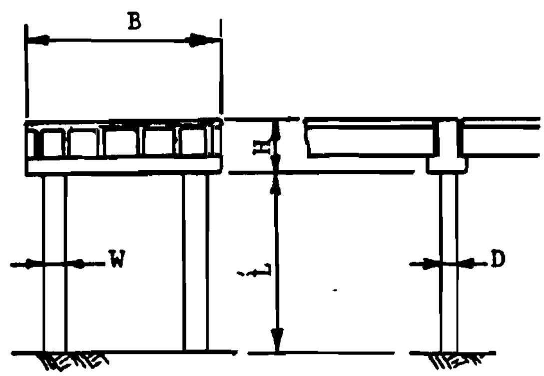
In several questions, specific sections of the AASHTO Standard Specifications for Highway Bridges are referred to. Note that both the current 1977 AASHTO Specification section number and the number of the equivalent section in the 1973 AASHO Specification are given in these questions.

Prior to general distribution of this questionnaire, it was given to several engineers of the Texas State Department of Highways and Public Transportation who spent one to two hours to complete it. Based upon comments from these engineers, it is suggested that prior to answering any questions in detail, a few minutes be taken to read through the entire questionnaire.

If upon completion of the questionnaire you consider that some areas of importance have not been addressed, please include any additional comments.

Finally, any published reports derived from the results of this questionnaire will not attribute any responses given to a specific individual or organization.

Nomenclature



B - width of pier bent

D - depth of pier, taken parallel to superstructure longitudinal axis

H - depth of bent cap

ℓ_u - unsupported wall length in hollow pier

L - length of pier from top of footing to underside of superstructure, also referred to as pier height

r - radius of gyration of pier cross section

$= \sqrt{I/A}$ I = moment of inertia, A = area

= .3D or .3W for solid rectangular piers

= .25D = .25W for solid circular piers

t - wall thickness in a hollow pier

W - width of pier, taken perpendicular to superstructure longitudinal axis

Data Concerning Respondent to Questionnaire

Your name:

Name of Your Organization:

Which of the following best describes the activities of your organization in bridge structures?

Design

Design/Construction

Construction

Other (please specify)

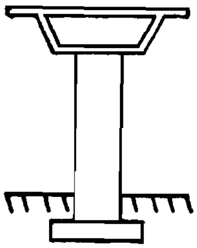
How many years have you been involved in civil engineering?

How many years have you been involved in the design and/or construction of bridge structures?

Have you had field experience involving bridge structures?

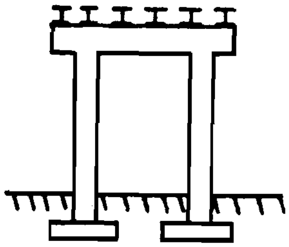
1. Bent/Foundation Types

Considering each of the pier height ranges separately estimate the degree of usage by your organization of the bent type/foundation classification shown. Present your estimate of usage as a percentage of the total number of bents used within each height range.

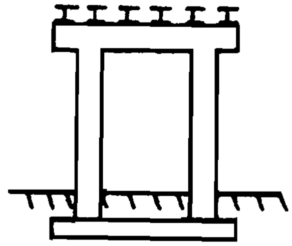


Single Pier Bent

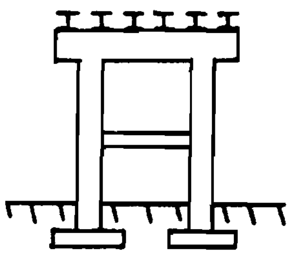
TYPE A



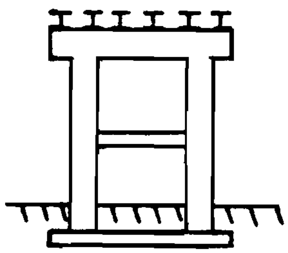
Multi Pier Bent
Individual Foundations
TYPE B



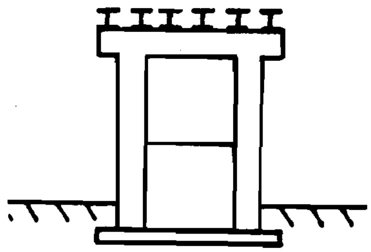
Multi Pier Bent
Common Foundation
TYPE C



Multi Pier Bent
Individual Foundations
Cross Bracing
TYPE D



Multi Pier Bent
Common Foundation
Cross Bracing
TYPE E



Multi Pier Bent
Common Foundation
Cross Wall
TYPE F

Note: A multi pier bent consists of two or more individual piers.

Bent Type		Pier Length, L: Less than 30': Usage from 1960 to 1979	Less than 30': Anticipated Usage Over Next Decade	30' to 50': Usage from 1960 to 1979	30' to 50': Anticipated Usage Over Next Decade	50' to 100': Usage from 1960 to 1979	50' to 100': Anticipated Usage Over Next Decade	Greater than 100': Usage from 1960 to 1979	Greater than 100': Anticipated Usage Over Next Decade
A	On Spread Footing or Raft								
	On Piles or Piers								
B	On Spread Footing or Raft								
	On Piles or Piers								
C	On Spread Footing or Raft								
	On Piles or Piers								
D	On Spread Footing or Raft								
	On Piles or Piers								
E	On Spread Footing or Raft								
	On Piles or Piers								
F	On Spread Footing or Raft								
	On Piles or Piers								
Other Bent Type (Please Describe)									
Total Percentage		100%	100%	100%	100%	100%	100%	100%	100%

Questions 2 to 9 relate to single pier bents only. If your organization has not used single pier bents in the past and presently does not intend to use them during the next decade, skip these questions and continue with No. 10.

2. Number of Single Pier Bents

Estimate the number of single pier bents satisfying the criteria given in the Introduction which fall into the height ranges specified below designed and/or constructed by your organization from 1960 to 1979. Also, estimate your anticipated usage of these piers during the next decade.

	Pier Height, L									
	Less than 30'		30' to 50'		50' to 100'		100' to 200'		Greater than 200'	
	Usage from 1960 to 1979	Anticipated Usage Over Next Decade	Usage from 1960 to 1979	Anticipated Usage Over Next Decade	Usage from 1960 to 1979	Anticipated Usage Over Next Decade	Usage from 1960 to 1979	Anticipated Usage Over Next Decade	Usage from 1960 to 1979	Anticipated Usage Over Next Decade
Number of Single Pier Bents Designed and/or Constructed										

3. Pier L/r Parallel to Bridge Axis - Single Pier Bents

Estimate your organization's usage of bridge piers for each of the following ranges of pier height to radius of gyration ratios, (L/r), in a direction parallel to the longitudinal axis of the bridge. Present your estimates as a percentage of the total number of piers used.

L/r Ratio	Usage From 1960 to 1979		Anticipated Usage Over Next Decade	
	Solid Section	Hollow Section	Solid Section	Hollow Section
Less than 20				
20 to 30				
30 to 50				
50 to 70				
70 to 100				
In excess of 100				
Total Percentage	100%	100%	100%	100%

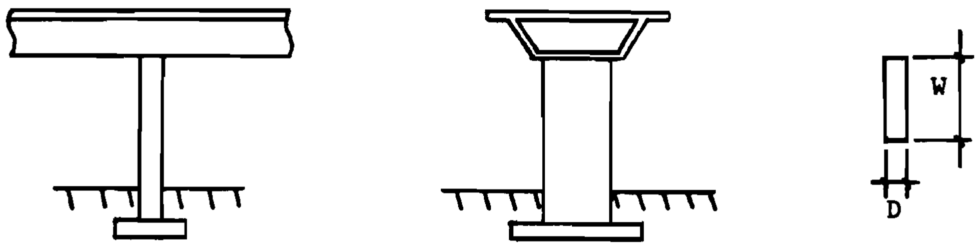
4. Pier L/r Perpendicular to Bridge Axis - Single Pier Bents

Estimate your organization's usage of bridge piers for each of the following ranges of pier height to radius of gyration radios, (L/r), in a direction transverse to the longitudinal axis of the bridge. Present your estimates as a percentage of the total number of piers used.

L/r Ratio	Usage from 1960 to 1979		Anticipated Usage Over Next Decade	
	Solid Section	Hollow Section	Solid Section	Hollow Section
Less than 20				
20 to 30				
30 to 50				
50 to 70				
70 to 100				
In excess of 100				
Total Percentage	100%	100%	100%	100%

5. Single Pier Bent Detailed as a Wall

At what ratio of pier width to pier depth (W/D ratio) would you detail the reinforcement of a concrete bridge pier as a wall instead of a column? i.e., delete confining reinforcement such as closed ties, hoops, or spirals.



Consider solid and hollow sections separately.

Solid Section W/D = ____________________

Hollow Section W/D = ____________________

6. Prestressing Single Pier Bents

Considering each of the pier type/height ranges listed below separately, estimate your organization's usage of unprestressed versus partially prestressed versus fully prestressed concrete bridge piers. Present your estimate as a percentage of the total number of piers used within each height range.

In the context of this question, use the following definitions to describe the degree of prestressing:

(a) Unprestressed - No prestressing steel in the pier
(b) Fully Prestressed - Prestressing steel used as vertical reinforcement; either no or very small tensile stresses (0 to 500 psi) allowed in concrete at service load
(c) Partially Prestressed - Prestressing steel used as vertical reinforcement; however, significant tensile stresses (700 to 1200 psi) allowed in concrete at service load

Degree of Prestressing	Pier Type/Height Range							
	Less than 30'		30' to 50'		50' to 100'		Greater than 100'	
	Solid Section	Hollow Section	Solid Section	Hollow Section	Solid Section	Hollow Section	Solid Section	Hollow Section
Unprestressed								
Partially Prestressed								
Fully Prestressed								
Total Percentage	100%	100%	100%	100%	100%	100%	100%	100%

Do you anticipate changes in your organization's usage of prestressing in concrete bridge piers in single pier bents?

7. Pier Cross Section Shape - Single Pier Bents

Considering each of the pier height ranges listed below separately, estimate your organization's usage of the pier cross-sectional types given. Present your estimate as a percentage of the total number of piers used within each height range.

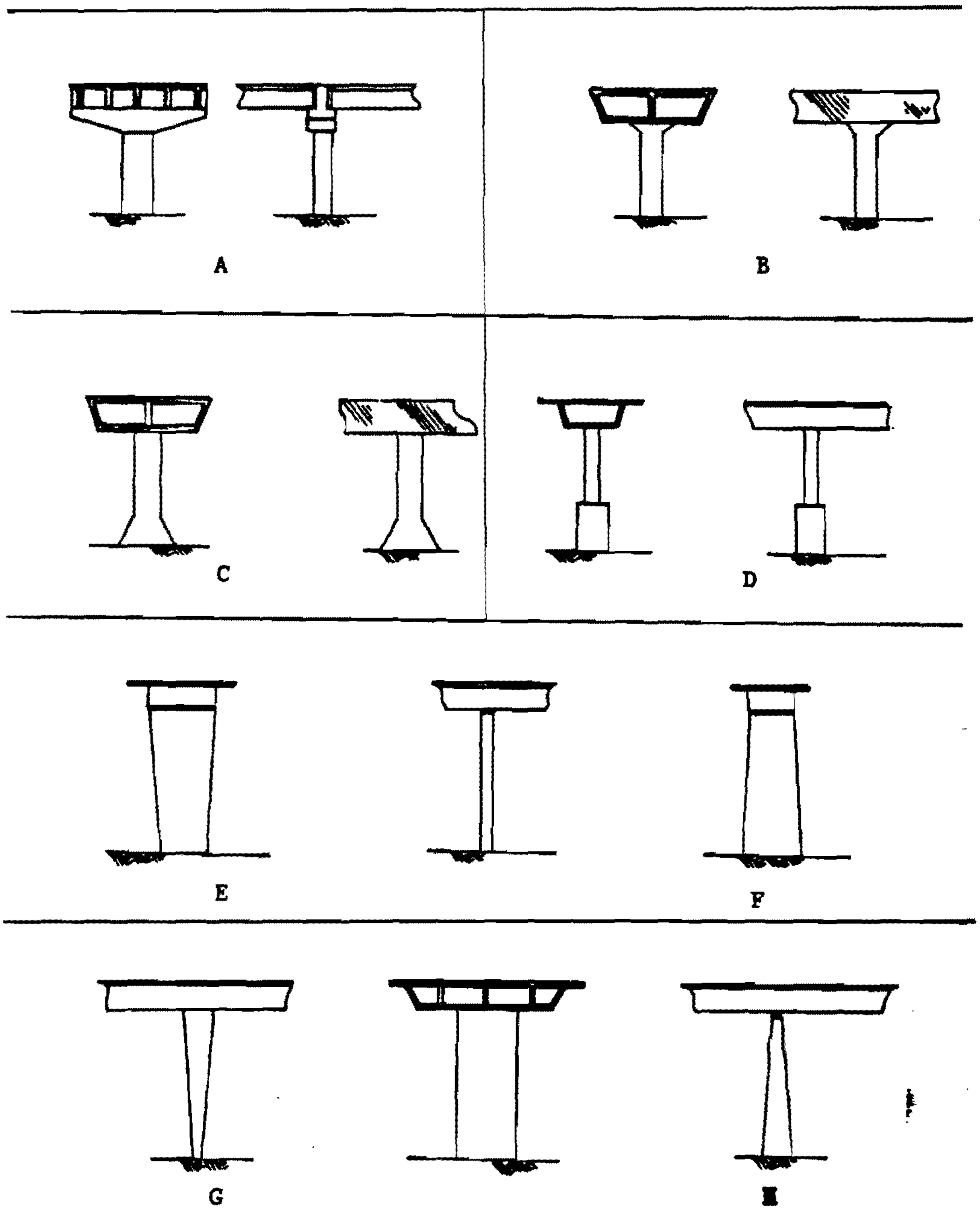
In the context of this question, a wall pier is defined as one fulfilling the criteria set by you in question No. 5.

Pier Cross Section Description	Pier Height, L: Less than 30'		30' to 50'		50' to 100'		Greater than 100'	
	Usage from 1960 to 1979	Anticipated Usage Over Next Decade	Usage from 1960 to 1979	Anticipated Usage Over Next Decade	Usage from 1960 to 1979	Anticipated Usage Over Next Decade	Usage from 1960 to 1979	Anticipated Usage Over Next Decade
Circular Solid								
Rectangular Solid								
Trapezoidal Solid								
Wall Type Solid								
Circular Hollow								
Rectangular, Single Cell Hollow								
Trapezoidal, Single Cell Hollow								
Wall Type Hollow								
Multi Cell Hollow								
Other (Please Describe)								
Total Percentage	100%	100%	100%	100%	100%	100%	100%	100%

8. Vertical Configuration of Piers - Single Pier Bents

Considering each of the pier height ranges listed separately, estimate your organization's usage of piers as described by the given categories. Present your estimate as a percentage of the total number of piers used within each height range.

Figures fitting the general description of each category are given below.



Pier Type	Pier Description	Cross Section	Pier Height, L: Less than 30': Usage from 1960 to 1979	Less than 30': Anticipated Usage Over Next Decade	30' to 50': Usage from 1960 to 1979	30' to 50': Anticipated Usage Over Next Decade	50' to 100': Usage from 1960 to 1979	50' to 100': Anticipated Usage Over Next Decade	Greater than 100': Usage from 1960 to 1979	Greater than 100': Anticipated Usage Over Next Decade
A	Constant Dimensions Along Height	Solid								
		Hollow								
B	Flare or Capital At Top	Solid								
		Hollow								
C	Flared at Bottom	Solid								
		Hollow								
D	Stepped	Solid								
		Hollow								
E	Linear Variation in Width, W, Larger at Top	Solid								
		Hollow								
F	Linear Variation in Width, W, Larger at Bottom	Solid								
		Hollow								
G	Linear Variation in Depth, D, Larger at Top	Solid								
		Hollow								
H	Linear Variation in Depth, D, Larger at Bottom	Solid								
		Hollow								
	Other (Please Describe)									
	Total Percentage		100%	100%	100%	100%	100%	100%	100%	100%

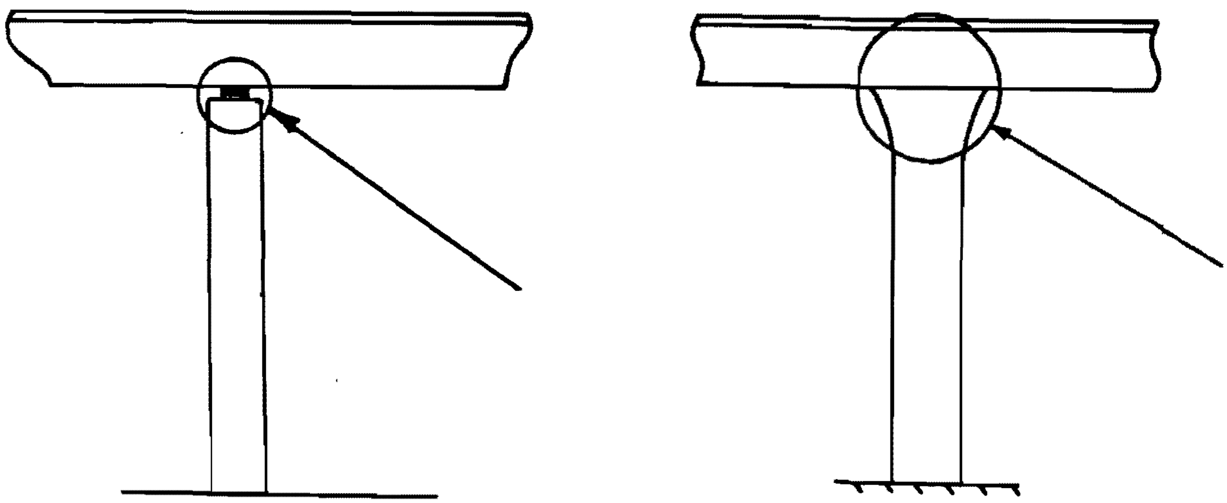
9a. Pier Superstructure Connections - Single Pier Bents

Considering each of the pier height/span length/span classification categories separately, estimate the number of pier superstructure connections which fall into the given connection classifications as used by your organization in the past 20 years. Present your estimate of use for each pier superstructure connection type as a percentage of the total connections used for each pier height/span length/span classification category.

If a pier height/span length/span classification category is not used by your organization, indicate by writing "N.A." in the appropriate category column.

The figures on page 13 show two bridge elevations with pier superstructure connection locations indicated.

Pier Superstructure Connection Type	Piers Less Than 50' Long				Piers Greater Than 50' Long			
	Span < 100'		Span > 100'		Span < 100'		Spans > 100'	
	Simply Supported Spans	Continuous Spans	Simply Supported Spans	Continuous Spans	Simply Supported Spans	Continuous Spans	Simply Supported Spans	Continuous Spans
Elastomeric Pad Bearings								
Monolithic Joint								
Pot Bearings								
Sliding Bearings								
Steel Shoes								
Fabrika								
Other (Please Specify)								
Total Percentage	100%	100%	100%	100%	100%	100%	100%	100%



Location of Pier Superstructure Connection

9b. Does your organization use different pier superstructure connections with solid piers than with hollow piers?

9c. If your answer is yes, please explain:

9d. Do you foresee a substantial shift in the types of pier superstructure connections used by your organization during the next decade?

9e. If your answer is yes, indicate in which pier height/span length/span classifications the hift will occur and the reasons for the shifts in connection type usage.

Questions 10 to 17 relate to multi pier bents only. If your organization has not used multi pier bents in the past and presently does not intend to use them during the next decade, skip these questions and continue with No. 18.

10. Number of Multi Pier Bents

Estimate the number of multi pier bents satisfying the criteria given in the Introduction which fall into the height ranges specified below designed and/or constructed by your organization from 1960 to 1979. Also, estimate your anticipated usage of these piers during the next decade.

	Pier Height, L									
	Less than 30'		30' to 50'		50' to 100'		100' to 200'		Greater than 200'	
	Usage from 1960 to 1979	Anticipated Usage Over Next Decade	Usage from 1960 to 1979	Anticipated Usage Over Next Decade	Usage from 1960 to 1979	Anticipated Usage Over Next Decade	Usage from 1960 to 1979	Anticipated Usage Over Next Decade	Usage from 1960 to 1979	Anticipated Usage Over Next Decade
Number of Multi Pier Bents Designed and/or Constructed										

11. Pier L/r Parallel to Bridge Axis - Multi Pier Bents

Estimate your organization's usage of bridge piers for each of the following ranges of pier height to radius of gyration ratios, (L/r), in a direction parallel to the longitudinal axis of the bridge. Present your estimates as a percentage of the total number of piers used.

L/r Ratio	Usage from 1960 to 1979		Anticipated Usage Over Next Decade	
	Solid Section	Hollow Section	Solid Section	Hollow Section
Less than 20				
20 to 30				
30 to 50				
50 to 70				
70 to 100				
In Excess of 100				
Total Percentage	100%	100%	100%	100%

12. Pier L/r Perpendicular to Bridge Axis - Multi Pier Bents

Estimate your organization's usage of bridge piers for each of the following ranges of pier height to radius of gyration ratios (L/r), in a direction transverse to the longitudinal axis of the bridge. Present your estimates as a percentage of the total number of piers used.

L/r Ratio	Usage from 1960 to 1979		Anticipated Usage Over Next Decade	
	Solid Section	Hollow Section	Solid Section	Hollow Section
Less than 20				
20 to 30				
30 to 50				
50 to 70				
70 to 100				
In Excess of 100				
Total Percentage	100%	100%	100%	100%

13. Number of Piers Within Multi Pier Bents

Estimate on a percentage basis the number of piers your organization uses in a multi pier bent for each bent width range.

Number of Piers Within Bent	Pier Height, L	Bent Width, B: Less than 15' One Lane — Usage from 1960 to 1979	Less than 15' One Lane — Anticipated Usage Over Next Decade	15' to 30' Two Lanes — Usage from 1960 to 1979	15' to 30' Two Lanes — Anticipated Usage Over Next Decade	30' to 60' Three, Four Lanes — Usage from 1960 to 1979	30' to 60' Three, Four Lanes — Anticipated Usage Over Next Decade	Greater than 60' More than Four Lanes — Usage from 1960 to 1979	Greater than 60' More than Four Lanes — Anticipated Usage Over Next Decade
2	<50'								
	>50'								
3	<50'								
	>50'								
4 or more	<50'								
	>50'								
Total Percent		100%	100%	100%	100%	100%	100%	100%	100%

14. Prestressing Multi Pier Bents

Considering each of the pier type/height ranges listed below separately, estimate your organization's usage of unprestressed versus partial prestressed versus fully prestressed concrete bridge piers. Present your estimate as a percentage of the total number of piers used within each height range. See question No. 6 for definitions of the various degrees of prestressing.

Degree of Prestressing	Pier Type/Height Range: Less than 30' — Solid Section	Less than 30' — Hollow Section	30' to 50' — Solid Section	30' to 50' — Hollow Section	50' to 100' — Solid Section	50' to 100' — Hollow Section	Greater than 100' — Solid Section	Greater than 100' — Hollow Section
Unprestressed								
Partially Prestressed								
Fully Prestressed								
Total Percentage	100%	100%	100%	100%	100%	100%	100%	100%

Do you anticipate changes in your organization's usage of prestressing in concrete bridge piers in multi pier bents?

15. Pier Cross Section Shape - Multi Pier Bents

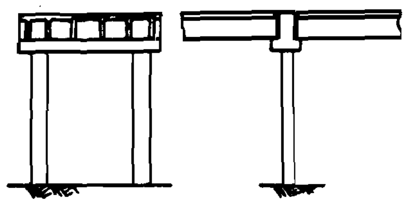
Considering each of the pier height ranges listed below separately, estimate your organization's usage of the pier cross-sectional types given. Present your estimate as a percentage of the total number of piers used within each height range.

Pier cross section description	Pier Height, L: Less than 30' — Usage from 1960 to 1979	Less than 30' — Anticipated Usage Over Next Decade	30' to 50' — Usage from 1960 to 1979	30' to 50' — Anticipated Usage Over Next Decade	50' to 100' — Usage from 1960 to 1979	50' to 100' — Anticipated Usage Over Next Decade	Greater than 100' — Usage from 1960 to 1979	Greater than 100' — Anticipated Usage Over Next Decade
Circular Solid								
Rectangular Solid								
Trapezoidal Solid								
Wall Type Solid								
Circular Hollow								
Rectangular, Single Cell Hollow								
Trapezoidal, Single Cell Hollow								
Wall Type Hollow								
Multi Cell Hollow								
Other (please describe)								
Total Percentage	100%	100%	100%	100%	100%	100%	100%	100%

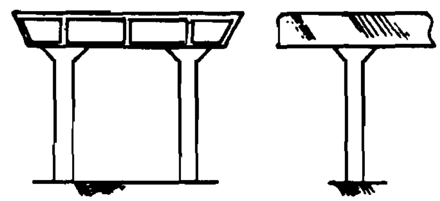
16. Vertical Configuration of Piers - Multi Pier Bents

Considering each of the pier height ranges listed separately, estimate your organization's usage of piers as described by the given categories. Present your estimate as a percentage of the total number of piers used within each height range.

Figures fitting the general description of each category are given below.



A



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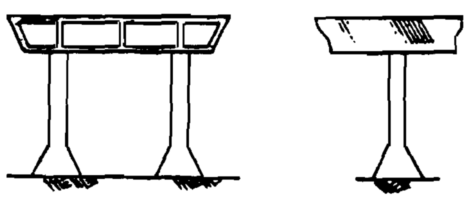

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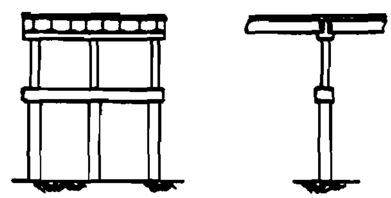
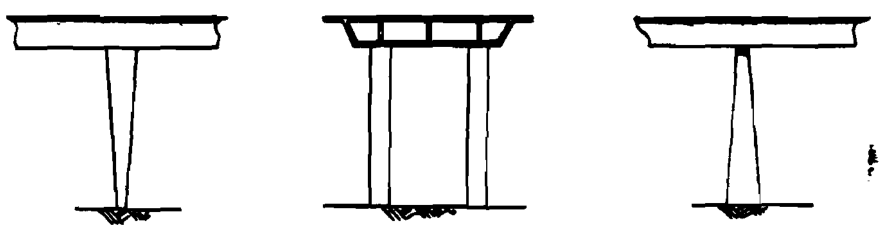


D

E F



G H

Pier Type	Pier Description	Cross Section	Pier Height, L: Less than 30' — Usage from 1960 to 1979	Less than 30' — Anticipated Usage Over Next Decade	30' to 50' — Usage from 1960 to 1979	30' to 50' — Anticipated Usage Over Next Decade	50' to 100' — Usage from 1960 to 1979	50' to 100' — Anticipated Usage Over Next Decade	Greater than 100' — Usage from 1960 to 1979	Greater than 100' — Anticipated Usage Over Next Decade
A	Constant Dimensions Along Height	Solid								
		Hollow								
B	Flare or Capital at Top	Solid								
		Hollow								
C	Flared at Bottom	Solid								
		Hollow								
D	Stepped	Solid								
		Hollow								
E	Linear Variation in Width, W, Larger at Top	Solid								
		Hollow								
F	Linear Variation in Width, W, Larger at Bottom	Solid								
		Hollow								
G	Linear Variation in Depth, D, Larger at Top	Solid								
		Hollow								
H	Linear Variation in Depth, D, Larger at Bottom	Solid								
		Hollow								
	Other (please specify)									
	Total Percentage		100%	100%	100%	100%	100%	100%	100%	100%

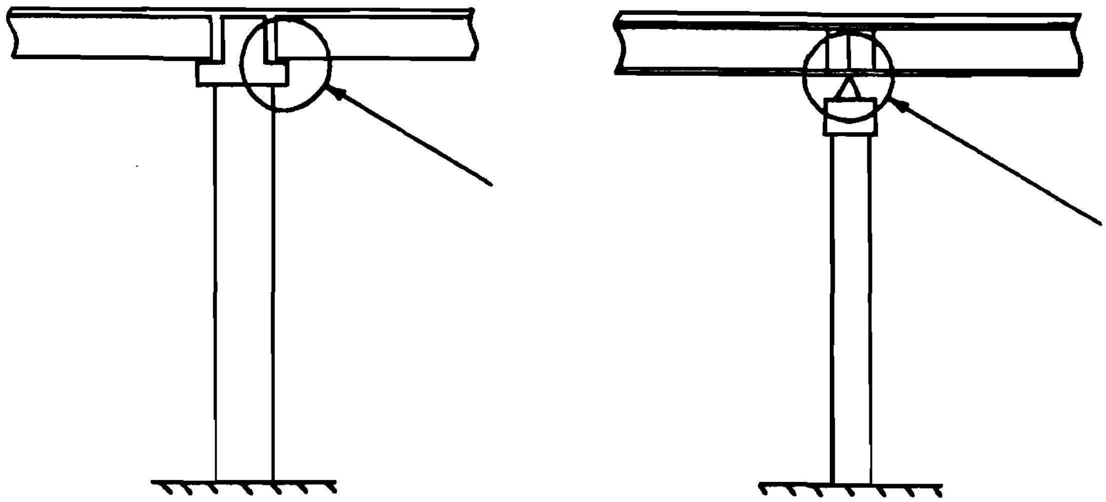
17a. Pier Superstructure Connections - Multi Pier Bents

Considering each of the pier height/span length/span classification categories separately, estimate the number of pier superstructure connections which fall into the given connection classifications as used by your organization in the past 20 years. Present your estimate of use for each pier superstructure connection type as a percentage of the total connections used for each pier height/span length/span classification category.

If a pier height/span length/span classification category is not used by your organization, indicate by writing "N.A." in the appropriate category column.

The figures on page 21 show two bridge elevations with pier superstructure connection locations indicated.

Pier Superstructure Connection Type	Piers Less Than 50' Long				Piers Greater Than 50' Long			
	Span ≤ 100'		Span > 100'		Span ≤ 100'		Span > 100'	
	Simply Supported Spans	Continuous Spans	Simply Supported Spans	Continuous Spans	Simply Supported Spans	Continuous Spans	Simply Supported Spans	Continuous Spans
Elastomeric Pad Bearings								
Monolithic Joint								
Pot Bearings								
Sliding Bearings								
Steel Shoes								
Other (please specify)								
Total Percentage	100%	100%	100%	100%	100%	100%	100%	100%



Location of Pier Superstructure Connections

17b. Does your organization use different pier superstructure connections with solid piers than with hollow piers?

17c. If your answer is yes, please explain:

17d. Do you foresee a substantial shift in the types of pier superstructure connections used by your organization during the next decade?

17e. If your answer is yes, indicate in which pier height/span classifications the shift will occur and the reasons for the shifts in connection type usage.

Except where otherwise noted, the remaining questions apply to both single and multi pier bents.

18. Load/Effects

For each of the following pier height and span length ranges, indicate the relative importance each load/effect category has upon the design of tall concrete bridge piers.

Use the numbers given below to rank the various effects.

Number	Meaning
0	Load/Effect never considered
2	Load/Effect considered, rarely important
5	Load/Effect considered, often important
7	Load/Effect considered, frequently critical
10	Load/Effect considered, always critical

Load/Effect	Pier Height, L			
	Less Than 50'		Greater Than 50'	
	Spans ≤ 100'	Spans ≥ 100'	Spans ≤ 100'	Spans > 100'
Structure Deadload				
Deadload from Horizontally Curved Superstructure				
Unbalanced Live Load				
Vehicle Braking Loads				
Centrifugal Forces Due to Live Load on Horizontally Curved Superstructure				
Dimensional Changes and Loads Due to Temperature Variations				
Dimensional Changes and Loads Due to Shrinkage and Creep				
Settlements and Lateral Movements of Foundations				
Rotational Stiffness of Foundations at Various Load Levels				
Pier Stiffness at Various Load Levels				

Load/Effect	Pier Length, L			
	Less Than 50'		Greater Than 50'	
	Spans $\leq$ 100'	Spans > 100'	Spans $\leq$ 100'	Spans > 100'
Serviceability Criteria (Deflection Limits)				
Wind Load Perpendicular to Superstructure				
Wind Load Parallel to Superstructure				
Wind on Liveload				
Ice/Snow Load on Bridge Structure				
Stream Ice/Current Loads				
Seismic Loads				
Out of Plumbness Due to Construction Tolerances				
Forces Occurring During Construction (specify)				
P-Δ Moment, Slenderness Effects				
Biaxial Bending Effects				
Other (specify)				

19. Load Combinations

Using AASHTO Load Factor Design Groups (1977 AASHTO Section 1.2.22, 1973 AASHO Section 1.5.17 B), indicate the relative importance each has upon the design of tall concrete bridge piers for the given pier height/span length ranges.

Use the following table to rank the load groups.

Number	Meaning
0	Never controls
2	Rarely controls
7	Frequently controls
10	Always controls

1977 AASHTO Load Groups	Value of β_d (1)	Pier Height, L							
		Less than 30'		30' to 50'		50' to 100'		Greater than 100'	
		Spans < 100'	Spans > 100'	Spans < 150'	Spans > 150'	Spans < 150'	Spans > 150'	Spans < 150'	Spans > 150'
I (2)	1.0								
	0.75								
IA (3)	1.0								
	0.75								
II	1.0								
	0.75								
III	1.0								
	0.75								
IV	1.0								
	0.75								
V	1.0								
	0.75								
VI (4)	1.0								
	0.75								
VII	1.0								
	0.75								
VIII	1.0								
	0.75								
IX	1.0								
	0.75								

(1) β_d deadload axial force parameter as defined in 1977 AASHTO Section 1.2.22.
(2) Equivalent to 1973 AASHO Load Group I, with β_d = 1.0.
(3) Equivalent to 1973 AASHO Load Group IA, with β_d = 1.0.
(4) Roughly equivalent to 1973 AASHO Load Group III, with β_d = 1.0.

20a. Are any load/effects or load groups more critical in the design of single pier bents than in the design of multi pier bents?
If your answer is yes, please explain:

20b. Are any load/effects or load groups more critical in the design of multi pier bents than in the design of single pier bents?
If your answer is yes, please explain:

21. Detailing

Make recommendations as to what you consider good engineering practice for tall concrete bridge piers in each of the areas listed below. Include both structural requirements and consideration of constructability in the formulation of your recommendations.

(a) Estimate the bent cap depth over bent width to pier width over length $(\frac{H/B}{W/L})$ ratios used by your organization in multi pier bents.

Number of Piers Within Bent	Pier Length, L	Bent Width, B			
		Less than 15'	15' to 30'	30' to 70'	Greater than 70'
2	< 50				
	> 50				
3	< 50				
	> 50				
4 or more	< 50				
	> 50				

(b) Minimum and maximum percentage of vertical reinforcement for solid and hollow cross sections.

Solid Section Minimum percentage ____________________

Maximum percentage ____________________

Hollow Section Minimum percentage ____________________

Maximum percentage ____________________

(c) Minimum and maximum spacing of vertical reinforcement for solid and hollow cross sections.

Solid Section Minimum spacing ____________________

Maximum spacing ____________________

Hollow Section Minimum spacing ____________________

Maximum spacing ____________________

(d) Spacing and layout of ties and stirrups in solid and hollow piers. Consider both vertical spacing and confining requirements. Drawings from some of your organization's projects showing typical reinforcing details for concrete bridge piers may be attached and would be most helpful to this study.

Solid Section

Hollow Section

(e) Minimum required concrete cover for the following environments:

(1) Exposed to air ____________________

(2) Exposed to water ____________________

(3) Exposed to salt water ________________

(4) Exposed to soil ____________________

(5) Exposed to deicing agents ____________

(f) Requirements for crack control (temperature and shrinkage) reinforcement in solid and hollow piers. Give details or sections of specifications used.

(g) (1) Range of concrete strengths used:

(2) Range of reinforcement strengths used:

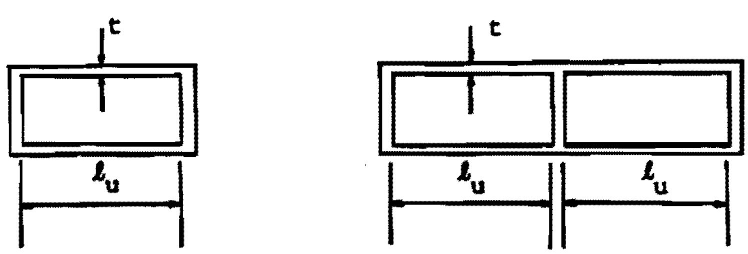
If either of the above are different for solid and hollow sections, indicate and give reasons for the difference.

(h) Requirement for the use of mechanical rebar splicing devices.

(i) Minimum wall thickness for hollow pier sections.

(j) Maximum unsupported length of wall you would allow in a hollow pier section.

Express your answer as a ratio of maximum unsupported wall length to wall thickness (ℓ_u/t ratio).



(k) Minimum cross-sectional dimensions for the pier of a single pier bent.

	W	D
Solid Section	________	________
Hollow Section	________	________

Minimum cross-sectional dimensions for a pier in a multi pier bent.

	W	D
Solid Section	________	________
Hollow Section	________	________

Do these minimum required dimensions change as different superstructure pier connection details are used? For example, would W and/or D vary if an elastomeric pad is used in place of a monolithic joint?

If your answer is yes, please explain:

(*l*) If possible, provide sketches or detail sheets showing your organization's typical structural details in the following areas:

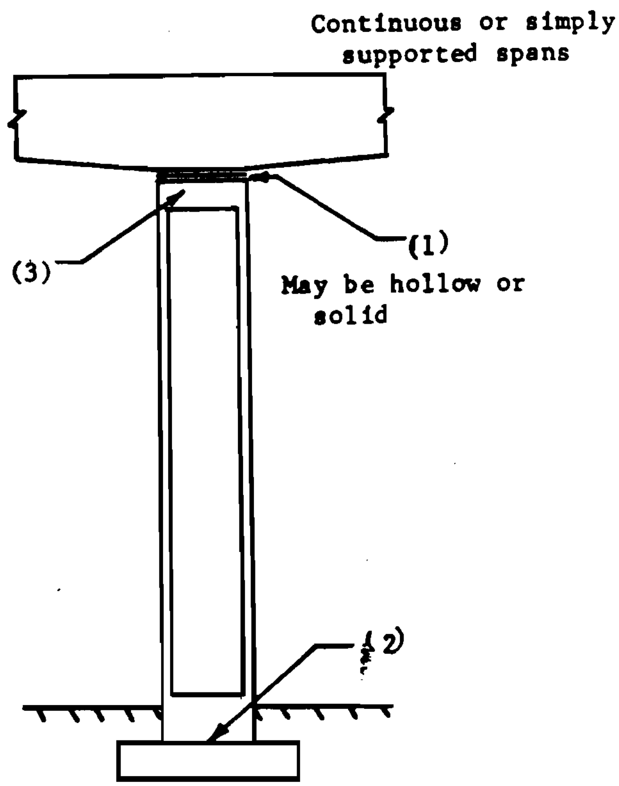
(1) Connection between pier and superstructure

(2) Connection between pier and foundation

(3) Cap beam details for hollow pier sections

(4) Prestressing anchorage details if used for piers

(5) Corner details for hollow piers



22. Specification Usage and Applicability

(a) Indicate your organization's current degree of usage of the following criteria in the design of tall concrete bridge piers:

(1) We use AASHTO Allowable Stress Design _______% of the time.

(2) We use AASHTO Load Factor Design _______% of the time.

(3) We use another specification _______% of the time.

If C is not zero, please indicate other criteria you use.

(b) Were the 1973 AASHO Specifications for the determination of allowable load on a tall concrete bridge pier possibly subjected to biaxial loading effects adequate for your design needs?________________
(1973 AASHO Sections 1.5.9 C-4 and 1.5.9 D-4)

(c) Is the present AASHTO Specification for slenderness effects in concrete piers adequate for your design needs?_______________
(1977 AASHTO Section 1.5.34; 1973 AASHO Section 1.5.22 C)

(d) Is the present AASHTO Specification for biaxial effects in concrete piers for load factor design adequate for your design needs?______
(1977 AASHTO Section 1.5.33 C; 1973 AASHO Section 1.5.22 B-5)

(e) Is 1977 AASHTO Section 1.5.34-7 (1973 AASHO Section 1.5.22 C-2) in conjunction with applicable preceding sections of the AASHTO Specification adequate for your design needs for tall concrete bridge piers?_______________

(f) If any of your answers to questions (c), (d), and/or (e) was No, please describe as specifically as possible the shortcomings you consider to be present in the current AASHTO Specification sections relevant to the design of tall concrete bridge piers.

23. Which of the following formats would be most useful to you in a specification for the design of tall concrete bridge piers? (More than one may be chosen.)

<u>Format A</u> - A set of equations similar in format to those presently in the AASHTO Specification (1977 Section 1.5.34-B; 1973 AASHO Section 1.5.22-C), and any required design aids or charts for a variety of tall pier configurations.

<u>Format B</u> - A set of general guidelines, criteria, and trouble areas in the use, design, and construction of tall concrete bridge piers, but no set calculation procedure would be specified.

<u>Format C</u> - A computer analysis program which would check the adequacy of design sections that you would input for specified load conditions.

<u>Format D</u> - A computer analysis-design program which would require input of only dimensional constraints and end conditions, and which would output an optimal bridge pier designed using optimization criteria contained in the program.

<u>Format E</u> - Another format type not listed above.

Format(s)____________ would be most helpful to me in the design of tall concrete bridge piers.

If you chose Format D as the most desirable output from this study, list in order of their relative importance the criteria that should be contained in the optimization portion of the computer program.

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