BEHAVIOUR OF FLEXIBLY CONNECTED STEEL FRAMES

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MASTER OF SCIENCE

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ABSTRACT

The moment - rotation behaviour of the beam-to-column connections in steel frames affects frame displacements and internal forces. Because connection behaviour is nonlinear, it produces non-linear behaviour of the structure. Given the difficulty of modelling and accounting for the nonlinear connection behaviour, it is tempting to continue to model it only crudely. Recently, however, there has been increased interest in improved models which can be incorporated into structural analysis computer programs. This thesis describes a procedure and computer program for analyzing rectangular steel frames with nonlinear beam-to-column connections, carrying sequential, non-proportional load systems. The frame is modelled as an assemblage of linear column, beam and bracing elements with nonlinear connections. The analysis program uses repeated cycles of linear analysis and connection stiffness modification to compute connection moments and rotational deformations corresponding to points on their moment-rotation curves. A Richard-Abbott function is adapted to express five moment-rotation function which simulate observed connection behavior under several cycles of load reversal. The study includes an examination of how accurately connection behavior ought to be modeled, in order to determine whether the current models are adequate or whether more realistic ones are warranted. Examples are presented to demonstrate the sensitivity of the response of frames to variations in assumed beam-to-column connection behaviour.

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A	member cross-sectional area
a	distance of concentrated load from A end of beam AB
b	length of beam AB
C	connection tangent stiffness
C^*	effective stiffness
D_A	displacement vector at A for beam AB
D_{Lo}	out-of-plane displacement vector
D_{vi}	in-plane displacement vector at master joint
E	modulus of elasticity
G	shearing modulus
G_A	stiffness distribution factor at A end of column
I_1	torsion constant
I_2	second moment of area about principal axis 2 of cross-section
K	effective length of column
K_{AA}	stiffness matrix at A for beam AB
K_D	first order elastic stiffness matrix
K_G	geometric stiffness matrix
m_1	torsional moment about X_1 axis
m_2	bending moment about X_2 axis
M_o	connection reference moment

M_p	plastic moment
M_y	yield moment
n	parameter, which defines the sharpness of Richard-Abbott curve
p	intensity of concentrated load
p_1	force component in X_1 direction
P_A^F	fixed end force vector at A end for beam AB
p_{A2}	force component at A for beam AB
p_{B1}^F	fixed end force in X_1 direction at B end of beam AB
P_{Uo}	out-of-plane force vector
P_{vi}	in-plane force vector at master joint v
S_B	connection stiffness at B end
S_o	slope of initial asymptote
S_p	slope of second asymptote
w	intensity of uniformly distributed load for beam AB
δ_{A1}	displacement in X_1 direction at A end of beam AB
ϕ_1	connection rotation about X_1 axis
ϕ_o	reference rotation of connection

CHAPTER 1

INTRODUCTION

1.1 Introduction

The behaviour of steel building frames may be affected substantially by the force-deformation behaviour of the beam-to-column connections. Connection deformation contributes to the lateral drift, thus intensifying the P-Δ effect and contributing to instability of the frame (Ang and Morris 1984; Bijlaard and Zoetmeijar 1986; Lindsey 1987; Chen and Kishi 1989). Connection behaviour also affects the internal force distribution. Frye and Morris (1975) demonstrated differences in the column moments and axial forces in an 11-storey unbraced frame with deformable connections and then with rigid connections. Ang and Morris (1984) analyzed a 2-storey braced frame with double web angle connections which would normally be assumed to act as "pins". They found theoretical column bending stresses that were larger than the axial-load stresses.

Ackroyd (1987) showed that under gravity load the assumption of "pinned" connections may lead to over design of beams and under design of columns. Bijlaard et al. (1986; 1988) evaluated the effect of semi-rigid connections on unbraced frames and demonstrated that connection flexibility tends to decrease the elastic buckling load of the frame. Connection behaviour also affects the end restraint provided to columns (Jones et al. 1981; Lindsey 1987; Lui et al. 1987). As a frame is loaded, the connection stiffness, and thus the restraint provided to the columns, gradually decrease. Thus the column effective lengths increase. Several methods have been proposed to account for the loss of restraint (Driscoll 1976; 1982).

Connection deformation causes a frame to respond differently to proportional loading than to non-proportional loading. Moncarz and Gerstle (1981) analyzed an 11-storey flexibly connected frame under similar proportional and non-proportional loading systems and demonstrated that sway under proportional loading is larger than that under non-proportional loading. However, the larger the final lateral loads, the smaller the effect of the different load histories. Deierlein (1991) analyzed a 2-storey planar frame under both proportional and non-proportional loading . He found that upon subsequent cycling of the wind load the structure reached an elastic shake down state after the second cycle. Subsequent elastic wind drifts were 30 percent smaller than in the proportional loading case. He found also that for design purposes, proportional loading gives a conservative prediction of both strength and serviceability.

Although there is an increasing tendency among researchers and designers to recognize the significant effects of beam-to-column connection behaviour on the behaviour of steel frames, until recently that behaviour has not been recognized explicitly in design specifications. For example, the Canadian Standard CSA-S16.1 M89 Limit States

Design of Steel Structures (CSA, 1989) recognizes two types of construction and associated design assumptions for steel frames. Continuous construction may be assumed when the beam-to-column connections are perfectly rigid. If they are relatively flexible, simple construction may be assumed, in which the beam-to-column connections are considered to have no moment resistance. These highly idealized models are very simple and convenient to use for design and analysis purposes, but they may lead to significant inaccuracies when the beam-to-column connections are semi-rigid.

There are several factors which deter structural designers from employing more realistic beam-to-column connection models in design and analysis. It is common practice when accounting for connection behaviour, to consider only the relationship between the flexural moment M and the corresponding rotational deformation ϕ - usually referred to simply as the M-ϕ behaviour. The M-ϕ behaviour of beam-to-column connections is non-linear, often even at service loads, thus causing the structure to behave non-linearly throughout part of its loading range. Because the analysis of non-linear structures is relatively complex and cumbersome, realistic non-linear behaviour has not been an attractive option to designers. Another deterrent to the recognition of realistic connection behaviour in design is the lack of knowledge about the sensitivity of steel frames to connection behaviour. When accounting for non-linear connection behaviour designers must make interactive decisions to modify either the structural elements in the frame, the connections, or both. In the absence of information on how significantly connection behaviour influences structural behaviour, the tendency has been to model the connection stiffness either as "rigid" or "pinned".

Recently, however, there has been growing interest in more accurate modelling of connection behaviour and its effects on structural behaviour. Several analytical models of connections and of their M-ϕ behaviour have been developed and incorporated into non-linear structural analysis computer programs. Unfortunately, very little research has been done on the sensitivity of steel frames to connection behaviour. Moreover, most of the presently available non-linear analysis programs deal with proportional loading of the structure and monotonic loading of the connections only. Structures normally experience sequential, non-proportional loading systems, and the differences in response to sequential and non-proportional loading may be significant. To properly understand the sensitivity of a structure to connection behaviour, structural analysis under non-proportional loading is needed.

1.2 Scope of Study

The primary objective of the current study was to develop a procedure and a computer program to perform a nonlinear analysis of three-dimensional, rectangular, flexibly-connected steel frames under either proportional or non-proportional loading. The program incorporates functions for approximating the M-ϕ behaviour of the beam-to-column connections. It employs an iterative procedure for accounting for non-linear connection behaviour.

A secondary objective was to use the program to perform sensitivity studies on typical steel frames to determine the effects of connection behaviour on internal force distribution, on frame displacements, on end restraint for columns and on the amplification of P-Δ effects. The study also included an examination of how connection deformation affects structural response under proportional and non-proportional loading systems. The objective of the sensitivity studies was to examine how accurately beam-to-column connection behaviour should be modeled, and thus to determine whether the current models are adequate or whether more precise modelling is warranted.

The study included the following four distinct phases :

a) The M-ϕ behaviour of typical beam-to-column connections under cyclic loading was modeled using the Richard-Abbott (Richard-Abbott, 1965) function. The model represents the connection behaviour under initial loading, unloading and reloading.

b) A beam-to-column connection classification system (EUROCODE3, 1989) was examined. In that system connections are classified as "rigid", "semi-rigid" or "nominally pinned", depending upon their M-ϕ behavior, which is expressed in terms of the properties of the connected members. The boundaries between connection classifications were expressed in terms of the Richard-Abbott function (Richard-Abbott, 1965).

c) A set of formulae was derived for calculating effective length factors for columns in flexibly connected frames. The formulae relate the effective length factor K to member and connection properties, the latter expressed in terms ofthe Richard-Abbott function (Richard-Abbott, 1965).

d) A structural analysis micro computer program capable of analyzing three-dimensional rectangular frames with nonlinear beam-to-column connections was developed. The program is able to accommodate sequential, non-proportional loading systems. It also accounts for the effects of the column axial forces on their bending stiffness (the P-δ effects), and it uses either the approximate method described in CAN/CSA-S16.2 M89 (CSA, 1989) or the load control Newton-Raphson method (Chen and Lui, 1991) to account for the effects of axial column forces on frame displacements and forces (the P-Δ effects).

e) The analysis program was used to conduct sensitivity studies to determine the effects of connection deformations on frame displacements, internal forces, end restraint for columns and different load histories (proportional and non-proportional loading systems).

1.3 Relationship to Previous Studies

Research on semi-rigid connection behaviour began as early as 1917 with the work of Wilson and Moore (Wilson and Moore, 1917) at the University of Illinois. It has

evolved through three stages. The first of these was the measurement of connection behaviour. More than 1000 static load tests have been conducted on beam-to-column connection specimens (Nethercott, 1985). An electronic database which includes most of the available moment-rotation data for beam-to-column connections has been developed at Purdue University (Kishi and Chen, 1986).

The second stage of connection research has involved the modelling of connection M-ϕ behaviour. A single-stiffness linear model was proposed by Rathbun (1936), and Monforton and Wu (1963) and Lightfoot and LeMesurier (1974), among others, used linear M-ϕ models. A bilinear model (Tarpy and Cardinal, 1981) provided an improved representation of connection behaviour. Frye and Morris (1976) developed a polynomial model to predict the behaviour of several connection types. Jones et al. (1982) used the cubic B-spline method to curve-fit experimental M-ϕ data. A cubic B-spline curve was used to fit every subset of data with continuity of first and second order derivatives enforced at their intersections. Colson and Couvear (1983) introduced a power function based on a three-parameter exponential model. Ang and Morris (1984) used a standardized Ramberg-Osgood function to model beam-to-column connection behaviour.

The third stage of connection research has been the incorporation of connection behaviour into the analysis process for semi-rigidly connected frames. Batho and Rowan (1930) introduced a beam line method for the analysis of frames with semi-rigid connections. Slope-deflection and modified moment distribution methods were applied to the problem by J.F Baker (1931) and J.C.Rathbun (1935). Ackroyd (1976), Simitses and Giri (1982), Ang and Morris (1984), Gerstle (1985) and Cook (1983) discussed the influence of connection flexibility on frame response, considering monotonic, proportional loading only. Moncarz and Gerstle (1981), Poggi and Zandonini (1985), Nethercot (1985), Chen and Lui (1986), and Hsieh and Dierlein (1990) considered proportional loading of the structure, while accounting for unloading behaviour of the beam-to-column connections. However, while structures are almost always subjected to sequential, non-proportional loading systems, little software is available for the analysis of non-linear, semi-rigidly connected structures under non-proportional or cyclic loading.

Accordingly, the investigation described herein was conducted with the objective of developing and programming a procedure to account for the nonlinear moment-rotation behaviour of beam-to-column connections when analyzing rectangular steel frames under sequential, non-proportional loading systems. The program treats proportional loading as a special case.

1.4 Assumptions and Limitations

The assumptions employed and the limitations of the analysis program developed in this study are subdivided into three categories.

Those relating to the individual members :

a) All members are assumed to be prismatic and straight.

b) Possible buckling of individual members or portions of the structure is ignored.

c) All columns are assumed to be either rigid or pinned fixed at their bases.

d) All members are assumed to be linearly elastic.

e) Floors are assumed to act as rigid diaphragms for resisting in-plane forces.

f) In general, small deflection theory is employed. However, when computing column stiffness matrices, the effect of axial column force on member bending stiffness is accounted for.

g) The effects of possible column panel zone deformation are ignored.

Those relating to the connection :

a) Transverse dimensions of members are assumed to be negligible. Thus all beam-to-column connections are assumed to be located at the intersection of the beam and column centre lines.

b) Axial and direct shearing deformations in connections are ignored and connections are assumed to have infinite torsional stiffness. Thus, only the flexural moment-rotation behaviour of the connection is modeled as non-linear.

c) Connection dimensions are assumed to be negligible. Thus the rotational deformation of each connection is assumed to be concentrated at a point.

Those relating to the whole structure :

a) Only static loading, in the form of concentrated loads and uniformly distributed loads, can be accommodated.

b) Lateral loads can be applied at floor levels only. The lateral loading at each floor level must be treated as a single three-component concentrated load vector.

c) The load control Newton-Raphson second order analysis procedure for accounting for P-Δ effects is valid for proportional loading only. Alternatively, the approximate iterative procedure of CSA S16.1 M89 (CSA,1989) may be selected.

CHAPTER 2

CONNECTION BEHAVIOUR

2.1 Connection Force-Deformation Behaviour

A beam-to-column connection in a steel building frame experiences six force components. As illustrated in the Fig 2.1, they are the beam axial force, p_1, vertical and horizontal shear force components, p_2 and p_3, torsional moment, m_1, and bending moments m_2 and m_3. The corresponding deformation components, also shown in the figure, including three linear deformations, δ_1, δ_2 and δ_3, and three rotational deformations, ϕ_1, ϕ_2 and ϕ_3.

Because the building floors act approximately as rigid in-plane diaphragms to which the beams are bonded or welded, the deformations δ_1, δ_3 and ϕ_2 are negligibly small, as are force components p_1, p_3 and m_2. Since the torsional stiffness of I-shaped sections is normally small compared to that of their connections, the torsional moment, m_1, and thus the torsional deformation, ϕ_1, are small also, and can be ignored. Finally, it is commonly assumed that the shearing deformation, δ_2, in the connections has negligible effect on the behaviour of the frame. Consequently, the shearing force and deformation are customarily ignored also. Thus it is common practice when accounting for connection behaviour to consider only the relationship between the flexural moment, m_3, and the corresponding rotational deformation, ϕ_3 - usually referred to simply as the M-ϕ relationship.

2.2 Experimental Measurement of Moment-Rotation Behaviour

2.2.1 Bare Steel Connections Under Monotonic Loading

When designing steel frames it is customary to assume that beam-to-column connections are either pinned or rigid. However, the notions of either pinned or rigid connections are simply extreme cases of true connection behaviour. Experimental investigations have shown that connections exhibit moment-rotation characteristics which vary over a wide spectrum between those extremes. Most connections normally regarded as pinned possess some rotational stiffness, while connections which are regarded as rigid always display some degree of flexibility. Accordingly, it is logical to classify all beam-to-column connections as semi-rigid, while recognizing "simple" and "continuous" connections as extreme cases. Fig 2.2 illustrates the moment-rotation behaviour of several commonly used beam-to-column connection types. The single web angle connection is a relatively flexible type, while the top and seat angle connection is a rather rigid type.

The M - ϕ behaviour of any real beam-to-column connection is intermediate between the two extremes, rigid and pinned, and every connection is capable of developing at least some moment resistance. For example, Lewitt et al. (1969) found that double web angle connections, which are usually treated as pinned connections, developed as much as 20% of the beam fixed-end moment at the specified load level. Batho and Rowan (1934) tested a variety of connection types and concluded that savings in beam weights of as much as 20% could be achieved by taking account of the end restraint provided by flexible connections.

The many beam-to-column connection tests that have been conducted during the past seventy five years have demonstrated that for most connection types connection moment-rotation behaviour is nonlinear, even at moderate load levels. The non-linearity of connection behaviour is due to a number of factors (Chen & Lui,1986; Barakat,1989). Some of the important ones are the following.

1. In bolted connections, particularly those in which the bolts are merely snug tightened, bolt slip may occur at moderate load levels.
2. Because of the geometries of connecting elements such as plates and angles, local yielding of flexural elements often occurs.
3. Large bearing stresses between bolts and connected elements often result in local yielding.
4. Local buckling of beam and or column elements in the vicinity of the connection sometimes occurs.
5. Deformation of connection elements may significantly alter the initial geometry of the connection.

Nethercot (1985) presented a summary of tests conducted on bare steel beam-to-column steel connections under monotonic loading. His summary, which incorporates all available data for tests conducted between 1934 and 1984, is described with reference to Table 2.1. Some of Nethercot's findings for the various connection types are summarized below.

Single Web Angle

All known tests come from three series, conducted by Lipson (1968, 1977, 1980). Sample M-ϕ curves show that for a single line of beam web bolts both connection stiffness and moment capacity increase as the number of bolts increases. With fewer than 6 bolts the connection functions virtually as a pin support; increasing the number of bolts to 8 or more produces initial stiffnesses which are largely independent of the exact number, with erratic variations in moment capacity. Other findings were that performance is quite sensitive to the details of the attachment of the angle to the column, (the welding arrangement, or the use of round or slotted holes, if bolted) and that the stiffness achieved in the first load application could not be matched in the second and subsequent cycles.

Web Side Plate

Early studies, performed as a backup to the Australian standardized connection initiative, provide an indication of rotation stiffness characteristics. Detailed M-ϕ data were never published. However McCormick (1984) suggested that the M-ϕ curves for such connections are approximately bilinear. Finite element and experimental work by Richard (1980,1982,1983), aimed at the development of a method for predicting M-ϕ behaviour, highlighted the importance of the fastening to the beam web. However, from the available results it appears that finger tight bolts produce connections with virtually no rotational stiffness or moment capacity, whilst fully torquing the bolts produces sufficient friction between the surfaces for there to be no difference in the performance of connections with round or slotted holes. For one test arrangement with 7 x 7/8 in. A325 bolts in the web of a W24 x 55 beam, Richard (1980) quotes a connection moment as high as 37 per cent of the elastic fixed end value.

Double Web Angles

Sixty-eight sets of M-ϕ curves have been identified. The typical M-ϕ curve shows the three regions of initial comparatively stiff behaviour, softening due to angle deformation etc, and enhanced stiffness due to direct bearing of the toe of the beam against the column face. Of these, elimination of connection slip, especially for joints employing few web bolts, appears to be particularly important. Moment capacities generally lie between 5 and 15 percent of the beam strength.

Flange Angles

This connection type may be regarded as genuinely semi-rigid as it possesses a smooth, nonlinear M-ϕ curve. Although the restraint provided can amount to 50 percent of the moment capacity of the beam, it should be noted that in two of the main test series (Hotz, 1983, Maxwell et al., 1981) fully torqued bolts were used to fasten each angle to the beam flanges. The possible reduction in performance that might result from increasing the distances between the lines of fastening if welding were used for either component appear not to have been studied, nor has the effect of using untorqued bolts on the beam flanges. Thus these results may only be representative of the stiffest versions of this connection type.

Header Plate

This form is defined as an end plate of less than the full beam depth welded to the beam web, either centrally or offset toward the top flange. Fastening to the column is by bolting. Torqued bolts were used in the two main sets of tests (Kennedy and Hafez, 1985, Sommer, 1969) and no stiffening was employed. The available data suggest that the stiffness and moment capacity are broadly comparable to those of similarly proportioned double web angle connections. Both quantities are chiefly dependent on plate thickness, plate depth and the horizontal distance between lines of bolts. The test by Kennedy and Hafez (1985), and those by Sommer (1965) produced similar M-ϕ

characteristics.

Flush End Plate

The typical M-ϕ curve is initially almost linear, contains a round "knee" at a moment of about 60 percent of its capacity and then becomes virtually linear again at a slope of approximately 1/40th of its initial value. The extent of the knee seems to be dependent upon joint details such as the end plate thickness, the column stiffening etc. Connection moment capacity is influenced chiefly by end plate thickness and column web stiffening. Extending the end plate slightly below the beam flange or using short backing plates behind the column flange are possible alternatives to the use of conventional web stiffeners. In certain cases moment as large as M_y or even M_p may be attained, where M_y and M_p represent the beam yield moment and plastic moment, respectively.

Extended End Plates

This connection type has received the widest study; 106 separate M-ϕ curves from 17 different investigations are available. These cover extensions on one or both sides of the beam, although most of the data are for an extension on the tension side only. The typical M-ϕ curve is similar to that for the flush end plate, but with greater initial stiffness and moment capacity, often up to the beam M_p, and sustained through a larger range of rotation.

Combined Web and Flange Angles

Only 2 test series have been identified; in one case (Zoetemeijer, 1974) four M-ϕ curves were obtained from the four connections in a frame test. Both sets of results gave M-ϕ curves which are nonlinear over the whole range of rotations, with a form rather similar to that for the flush end plate. Both initial stiffness and moment capacity were strongly influenced by the details of the two types of angles. Factors which improve performance are: increased angle thickness, reduced gage for flange angles (web angle gage was not a variable) and increased length of flange angle, with web angle length having only a comparatively small effect.

Tee Stubs

Once again only 2 sets of M-ϕ data are available and these exhibit one rather noticeable difference. For the tests by Zoetemejer (1974) significant slip occurred at about 30-40 percent of the moment capacity of the joint, leading to a plateau on the M-ϕ curve followed by an increase in stiffness as the bolts came into bearing. Such behaviour is absent from the results of Bannister's (1966) tests, which show a remarkable degree of consistency in terms of a virtually linear initial stiffness. Changes in the number of bolts used to secure the stems of the tees to the column flanges affected only the moment capacity.

Top Plate and Seat Angle

Although several series of test have been performed very little M-ϕ data are available. The M-ϕ curves of Van Dalen's (1982) tests show that initial connection stiffness is maintained up to about 50 percent of the connection moment capacity. Although this may be extended by direct welding of the top flange of the beam to the column face, this tends to drastically reduce ductility, leading to a very brittle connection.

Tee Stubs and Web Angles

Only one study (Zoeitemeijer, 1974) using bolted joints has been identified, with only one M-ϕ curve provided. Direct comparison with flush end plate tests from the same series suggests that the tee-stub plus web angles is slightly stiffer and has a larger moment capacity.

2.2.2 Bare Steel Connections Under Cyclic Loading

Under wind or seismic loading, beam-to-column connections may experience moment fluctuations or even reversals. Beam-to-column connection tests have demonstrated that the unloading behaviour of a connection may be quite different than its behaviour under initial loading. The moment-rotation curves shown in Fig. 2.2 represent initial-loading connection behaviour only. Under cyclic loading, the behaviour illustrated in Fig. 2.3

is typical. The unloading curve is approximately a double mirror image of the loading curve, the unloading stiffness being approximately equal to the initial-loading stiffness. Because of this behaviour, identical connections at the two ends of a beam may behave differently at a given stage of loading of the structure, and the hysteretic behaviour of the connections may have a significant effect on both the static and the dynamic response of the structure.

Popov and Pinkney (1969) investigated five different connection specimens, three of them involving connections to the column flange, the other two involving connections indirectly to the column web. Bertero et al. (1972) and Popov et al. (1973) studied the energy absorbtion and dissipation capacities of the connection. Astaneh et al. (1989) studied the behaviour of double angle connections under cyclic loading. The behaviour of end-plate and extended end-plate connections under cyclic loading was investigated by Tsai et al. (1990), and Ghobarah et al. (1992). Some of the conclusions reached in the above-mentioned investigations are the following.

1 The moment-rotation hysteresis loops for cyclically loaded steel connections are highly reproducible.

2. The ability to withstand severe repeated and reversed loading seems to be assured for properly designed and fabricated steel connections.

3. The energy absorbtion and distribution capacity of a connection is an important indicator of connection reliability.

4. The energy dissipation per cycle of moment reversal is approximately

proportional to the residual deformation of the connection.

5. In the absence of slip, the mathematical representation of the hysteresis curve using a standard function such as the Ramberg-Osgood function has been found to be highly satisfactory for structural analysis purposes.

2.2.3 Connections of Composite Beams to Columns

Interest in the behaviour of semi-rigid connections between composite steel-concrete beams and steel columns has developed only recently. The use of continuous reinforcement over column lines to provide for composite action at the beam-to-column connections was first proposed by Barnard (1970). Since then several investigators have conducted tests to determine the characteristics of composite semi-rigid connections. Johnson and Hope-Gill (1970) tested several short double-cantilever composite beams at the University of Cambridge. Echeta and Owens (1980) tested two semi-rigid composite connections to a concrete-filled rectangular tube section column. Van Dalen and Godoy (1981) tested a series of similar composite and bare steel connections. Experimental studies of full scale composite beam-to-column connections have been conducted by Leon et al. (1987). Bernuzzi et al. (1991) investigated the behaviour of header plate, flush end plate, extended end plate and "rigid" fully welded composite connections under monotonic and cyclic loading. It was found that composite action greatly increased the strengths of the connections and that semi-rigid composite connections can sustain large rotational deformations.

Experimental studies of composite connections have revealed the following special characteristics.

1) Because behaviour is influenced by several material and geometric parameters, the determination of a moment-rotation curve for a particular type of connection requires a combined experimental-analytical investigation.

2) Even a careful experimental study may not clarify all relevant variables. For example, while most tests are conducted with a fixed value of the moment-to-shear ratio at the connection, for a weak connection the interaction between the moment and shear should be taken into account when developing a yield surface.

3) For composite semi-rigid connections the moment-rotation curves are not symmetrical about the zero-moment axis.

4) If cyclic loading is anticipated, the degeneration of the moment resistances of the connections should be accounted for.

5) For frames which include composite beams, the section properties for the beams are different at mid-span than at the columns.

6) Serviceability criteria must account for both time-dependent effects such as creep and shrinkage of the concrete, and the effects of end restraint.

Because of the complexity of composite connection behaviour, comprehensive mathematical models of their moment-rotation behaviour are not yet available. Research is needed on the behaviour of semi-rigid composite connections and frames which include them.

2.3 Connection Data Bases

Several researchers have assembled data bases of experimentally measured moment-rotation data for the commonly used beam-to-column connection types. The data bases include not only the connection behaviour, but also the material and geometric parameters which affect that behaviour.

Goverdhan (1983) complied connection data from experimental tests conducted subsequent to 1950. His summary covered double web angle, single web angle, single wing plate, header plate, end plate, and top and seat angle connections. Goverdhan evaluated the various analytical functions which have been proposed to model connection moment-rotation behaviour and indicated which are the most accurate. For double web angle connections, he recommended the use of the bi-linear function developed by Lewitt, Chesson and Munse (1969). The Richard-Abbott function (Richard et al., 1975) was recommended for single plate connections. Sommer's (1969) polynomial function was recommended for header plate connections. Goverdhan believed that none of the available prediction equations was suitable for the end plate connection and that additional research was needed to develop a suitable one. A polynomial prediction function developed by Altman et al. (1982) was recommended for top and seat angle connections.

Nethercot (1985) reviewed data from more than 70 separate experimental studies on steel beam-to-column connections. Of the more than 700 test specimens considered, Nethercot found that approximately 300 provided currently useful moment-rotation data. The study covered the following ten types of connections.

1) Single web angle
2) Single web plate
3) Double web angle
4) Flange angle
5) Header plate
6) Flash/extend end plate
7) Combined web and flange angle
8) T-stubs
9) Top and seat angle
10) T-stubs and web angles

Kishi and Chen (1986) constructed a comprehensive computerized database for monotonically loaded bare steel beam-to-column connections. The database encompassed all available experimental data for riveted, bolted and welded connections tested between 1936 and 1986. For each set of test data, the following information was included also:

1) connection type and fastening mode;
2) name of author(s), test identification and name of country where test was conducted;

3) beam and column material properties, and size of fasteners;

4) material properties for connecting elements; and

5) size parameters for all connection components.

The study which led to the development of the database included a review of analytical functions for modelling moment-rotation behaviour. Considering accuracy, simplicity and versatility, the Frye-Morris polynomial (1975), the Kishi-Chen power function (1988), the Ramberg-Osgood function proposed by Ang and Morris (1984) and the Wu-Chen exponential model were recommended.

2.4 Effect of Connection Behaviour on Column Strength

2.4.1 Background

The analysis of an elastic, perfectly straight, pin-ended column was first performed by Euler in 1759. Subsequent analyses have incorporated the effects of initial crookedness, load eccentricity, residual stresses, inelastic behaviour and continuity with other members in a frame. Several recent studies have examined the effects of beam-to-column connection behaviour on the strength of columns in steel frames.

Sugimoto and Chen (1982) used an "approximate deflection method" in an examination of the effects of connection behaviour on column end restraint. Vinnakota (1982), Razzaq and Chang (1981) and Razzaq (1983) employed a finite difference scheme to develop column load deflection curves. Jones, Kirby and Nethercot (1980) employed a one-dimensional finite element model and incremental loading scheme with iteration at each load step to account for both nonlinear column behaviour and nonlinear connections to beams at the column ends. Bergquist (1977) used test data to obtain a load-deflection curve for a column restrained by nonlinear connections to beams at its ends.

Notwithstanding the studies cited above, the effects of the beam-to-column connections on the effective length factors, and thus the strengths, of columns in flexibly connected steel frames are not yet fully understood.

2.4.2 Effective Length Factors for Columns in Flexibly Connected Frames

In the discussion that follows, the braced frame shown in Fig. 2.4 (a) was used. In this discussion it was assumed that elastic buckling of all columns occurs simultaneously and that the buckled configuration is as illustrated by the dashed lines in the figure. That is, the deflected shapes of all columns are similar, with a rotation of θ_U at one end and θ_L at the other, and the deflection curves of all beams are symmetrical about their mid points.

As column U-L, whose buckled configuration is shown in Fig 2.4 (a), and the columns above and below it buckle, the ends of the beams framing into joints U and L experience rotations θ_U and θ_L respectively. If the beam-to-column connections are rigid, the total restraining moment provided by the beams at joint U is

$$M=\left(\sum\left(\frac{2EI}{L}\right)_b\right)\theta_b \tag{2.1}$$

Where I_b and L_b are the second moment of area and the length of the typical beam.

Rearranging Eq. (2.1)

$$\theta_b=\frac{M}{\sum\left(\frac{2EI}{L}\right)_b}=\theta_U \tag{2.2}$$

where

θ_b is the end rotation for the typical beam, and

θ_U is the rotation of joint U.

Since the restraining moment M is apportioned to the columns immediately above and bellow joint U in proportion to their EI/L value, the moment applied to column UL is

$$M_U=\frac{\left(\frac{EI}{L}\right)_U}{\sum\left(\frac{EI}{L}\right)_c}\times M=\frac{\left(\frac{EI}{L}\right)_U\left(\sum\left(\frac{2EI}{L}\right)_b\right)}{\sum\left(\frac{EI}{L}\right)_c}\times\theta_U \tag{2.3}$$

Where

$(EI/L)_U$ refers to column UL, and summation of $(EI/L)_c$ refers to the two columns meeting at joint U.

Defining a stiffness distribution factor at joint U

$$G_U = \frac{\sum (\frac{EI}{L})_c}{\sum (\frac{EI}{L})_b} \qquad (2.4)$$

Eq. (2.3) can be written

$$M_U = \frac{2(\frac{EI}{I})_U \theta_U}{G_U} \qquad (2.5)$$

A similar stiffness distribution factor, G_L, at joint L can be defined, at the moment, M_L, applied to column UL computed as

$$M_L = \frac{2(\frac{EI}{L})_L \theta_L}{G_L} \qquad (2.6)$$

The differential equation describing the equilibrium of buckled column UL can be expressed in terms of G_U and G_L. The solution to that equation has been presented in terms of a nomograph (Julian and Lawrence, 1959) which, for given value of G_U and G_L, yields the effective length factor, K, for column UL.

If the beam-to-column connections in the frame shown in Fig. 2.4 (a) are flexible, the typical beam with deforms as shown in Fig 2.4 (b). The beam end rotation being θ_b and the rotational deformation of the connection denoted as θ_c. The rotation at joint U can be expressed as

$$\theta_U = \frac{M}{\sum\left(\frac{1}{\left(\frac{2EI}{L}\right)_b} + \frac{1}{C}\right)} \tag{2.7}$$

where C is the tangent stiffness of the connection.

Defining an effective stiffness

$$C^* = \frac{1}{\sum\left(\frac{1}{\left(\frac{2EI}{L}\right)_b} + \frac{1}{C}\right)} \tag{2.8}$$

The rotation at joint U can be written

$$\theta_U = \frac{M}{C^*} \tag{2.9}$$

Then the moment applied to column UL at U is

$$M_u = \frac{\left(\frac{EI}{L}\right)_U}{\sum\left(\frac{EI}{L}\right)_c} \times M = \frac{\left(\frac{EI}{L}\right)_U C^* \theta_U}{\sum\left(\frac{EI}{L}\right)_c} \tag{2.10}$$

Comparing equations (2.4) and (2.10) a stiffness distribution factor G_U^*, at joint U in the flexibly connected frame can be defined as

$$G_U^* = \frac{\sum\left(\frac{EI}{L}\right)_c}{2C^*} \tag{2.11}$$

A stiffness distribution factor G_L^*, at joint L can be defined similarly, and the Julian and Lawrence nomograph can be used to determine the effective length factor for column UL in the flexibly connected frame.

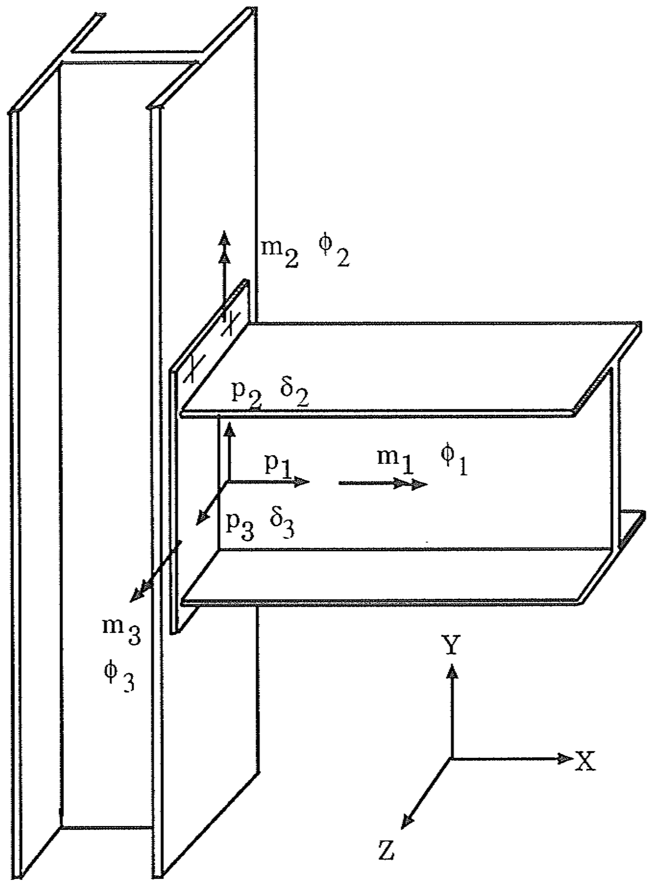


Fig. 2.1 Force and Dofomation Components of Connection

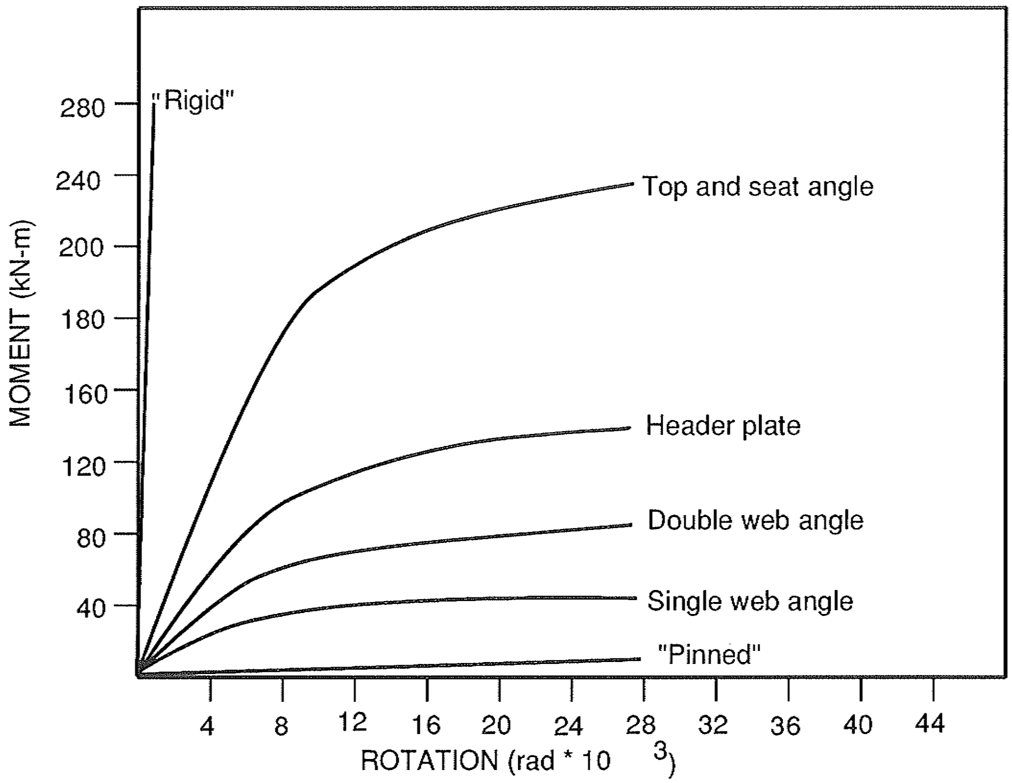


Fig. 2.2 Typical Moment-Rotation Curves for Common Connections

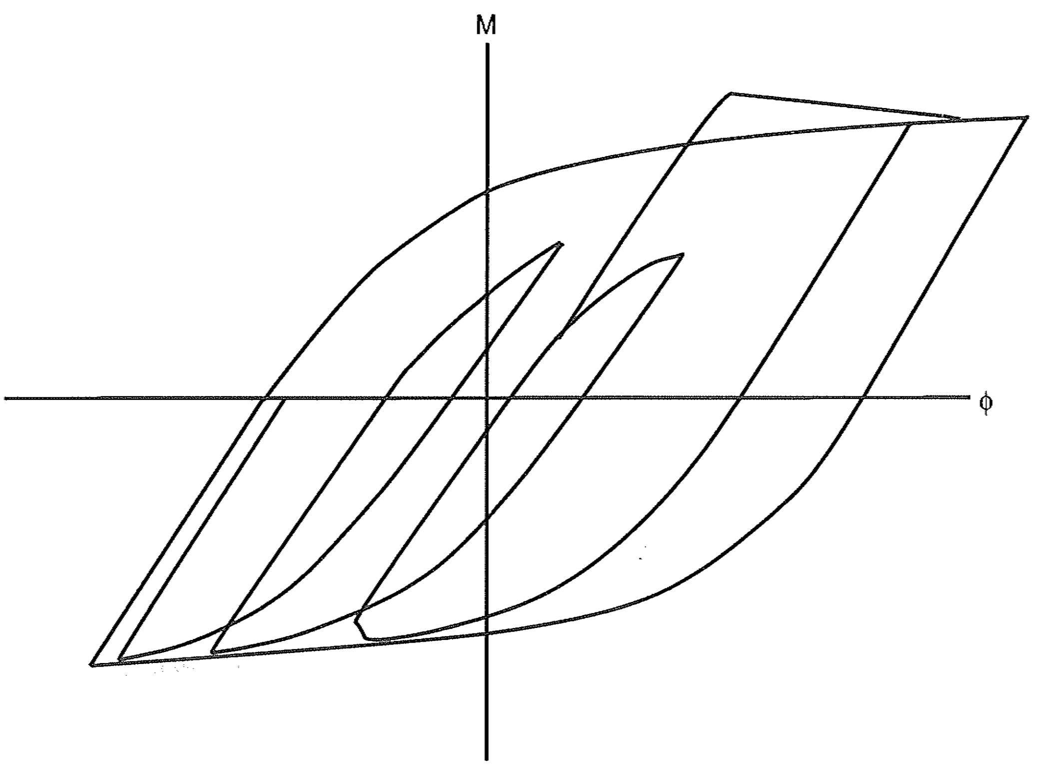


Fig. 2.3 Experimental Moment-Rotation Diagram

Type of connection	Reference for experimental data	Data	Country of origin	No. of test	Comments on M-ϕ curves
1. Single web angles	Lipson	1969	Canada	138	- Provided for 17 bolt/bolt tests and 4 weld/bolt test
	Lipson	1977	Canada	43	- Provided as bolt M vs ϕ for 13 weld/bolt tests
	Lipson & Antonio	1980	Canada	33	- Provided as bolt M vs ϕ for 11 weld/bolt tests
2. Double web angle	Bath & Rowan	1934	U.K.	3	
	Rathbun	1936	U.S.A.	7	
	Hechtmann & Johnston	1947	U.S.A.	4	
	Munse et al.	1959	U.S.A.	4	Provided for each test
	Lewitt et al.	1969	U.S.A.	7	Provided for each test
	Sommer	1969	Canada	1	Provided for each test
	Bergquist	1977	U.S.A.	3	Also provides unloading stiffness
	Bennetts et al.	1978	Australia	2	Provided for each test
	Mansell & Pham	1981	Australia	8	Some indication from shear and rotation results
	Bose	1981	U.K.	1	Provided
	Birkemoe & Gilmour	1978	Canada	2	Not provided
	Ricles & Yura	1982	U.S.A.	8	Not provided
	Wing & harris	1983	Canada	12	Not provided
	Bjorhovde	1984	U.S.A.	10	Provided for each test
	Onuah et al.	1988	Canada	20	Provided for each test
3. Web side plate	Pham & Mansell	1982	Australia	5	Some indication from shear rotation results
4. Flange angle	Batho & Rowan	1936	U.K.	10	
	Maxwell et al.	1981	U.K.	12	Provision of full curves
	Bose	1981	U.K.	1	Provision of full curve
	Van Dalen & Godoy	1982	Canada	1	Full curve for this test and for 2 similar composite specimens
	Hotz	1983	1983	37	Provision of full curves
	Rathbun	1936	1936	3	
	Hechtmann & Johnston	1947	1947	19	
5. Bottom flange angle & web angle					

Table 2.1 Summary of Bare Steel Connection Tests

Type of connection	Reference for experimental data	data	Country of origin	No. of test	Comments on M-ϕ curves
6. Header plate	Sommer	1969	Canada	20	- 8 M-ϕ curves given
	Kennedy & Hafez	1984	Canada	8	- Provision of full curves
	Bennetts et al.	1978	Australia	2	- Provision of full curves including unloading and, in one case, reloading
	Pham & Mansell	1982	Australia	7	- Some indication from shear-rotation results
7. Flush end plate	Zoetmeijer & Kolstein	1975	Netherlands	12	Provide for each test
	Bose	1981	U.K.	1	Provided
	Morris & Newsome	1981	U.K.	4	Provided for each test
	Mann & Morris		U.K.	6	M versus deflection curves provided
	Zoetemeijer	1981	Netherlands	23	Provided for 16 tests only, other 7 supplied privately
	Phillips & Packer	1981	Canada	5	Provided for each test
	Jenkins et al.	1984	U.K.	3	Provided for each test
	Zoetemeijer	1974	Netherlands	6	Provided for all tests four of which were for joints in frames
8. Extended end plate	Sherbourne	1961	U.K.	5	
	Johnson et al.	1960	U.K.	1	
	Bailey	1970	U.K.	18	
	Surtees & Mann	1970	U.K	5	
	Zoetemeijer	1974	Netherlands	8	Provided for 4 joints in frame test only
	Grundy	1980	Australia	2	Load-deflection curves only provided
	Packer & Morris	1977	Canada	5	Provided for 3 tests only
	Tarpy & Cardinal	1981	U.S.A.	16	Provided for 2 tests only
	Bahia et al.	1981	U.K.	20	Provided for 1 tests only
	Jenkins et al.	1984	U.K.	2	Provided for each test
	Moor & Sims	1983	U.K.	4	Provided for 2 test only
	Zoetemeijer & Munter	1984	Netherlands	4	Provided for each test
	Zoetemeijer	1981	Netherlands	10	Provided for each test
	Zoetemeijer	1981	Netherlands	5	Provided for each test

Table 2.1 Summary of Bare Steel Connection Tests (continued)

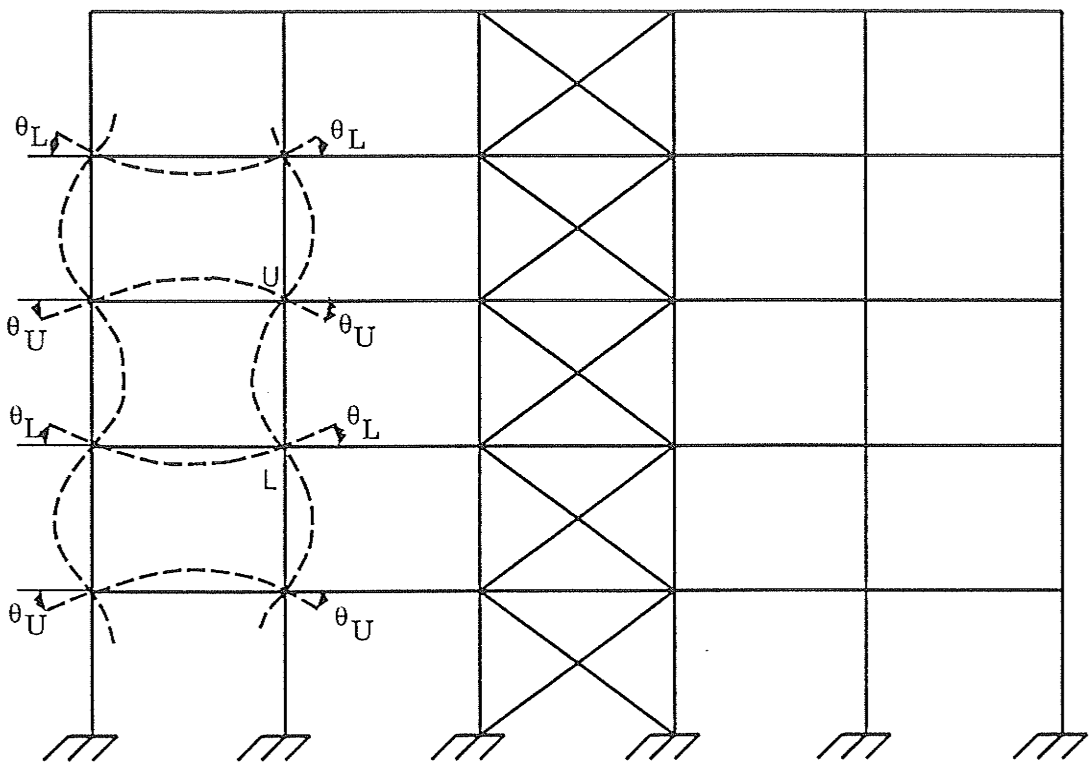


Fig. 2.4(a) Buckling of Flexibly-Connected Frame

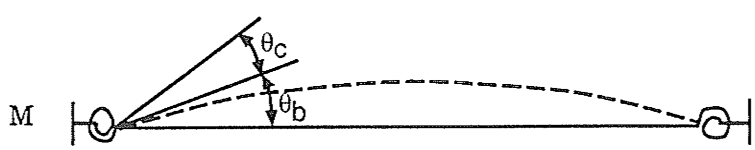


Fig. 2.4 (b) Beam With Flexible End Connections

CHAPTER 3

MODELLING OF CONNECTION BEHAVIOUR

3.1 Modelling of Beam-to-Column Connections

Physical testing of connection specimens is the primary means by which the behaviour of connections is established. However, because physical testing is time consuming and expensive, numerous attempts have been made to compute connection behaviour through the use of analytical connection models. Yield line analysis, and more recently finite element analysis, have been used.

Sherbourne and Murthy (1978) used finite element techniques to investigate and develop simple equations for the design of both welded and bolted beam-to-column connections, taking into account elasto-plastic material behaviour. Krishnamurthy and Graddy (1976) studied the behaviour of end plate connections using 2-D and 3-D finite element techniques. Lipson and Muhammad (1978) modelled elastic-plastic behaviour of single web angle connections subjected to cyclic loading involving end shear and moment. Ricles et al. (1982) used an elastic finite element technique to study and model a modified block shear failure of double-row bolted-web connections. However, their study was confined to the strength and stability of only some of the connection components. Tarpy and Cardinal (1981) used a finite element model to perform a parametric study of 97 end plate connections. They obtained a prediction equation for the M-ϕ relationship using a multiple linear regression analysis. Krishnamurthy et al. (1979) developed a two-dimensional finite element model for end plate connections. The results of their parametric study were statistically processed by a multiple linear regression analysis package to obtain moment-rotation equations. Richard et al. (1980) studied single plate connections using the finite element method. They proposed a powerful prediction function, the Richard-Abbott function, for modelling the M-ϕ behaviour of the connection. Few finite element studies of connection behaviour have been conducted during the past ten years.

Yield line analysis was used by Abolitz and Warner (1965) in a study of brackets welded to column webs. Stockwell (1974) analyzed column webs with welded beam connections. From his analysis Stockwell computed the ultimate moment which could be transferred through the connection to the column web. Packer and Morris (1977) developed the analysis of the column flange failure mechanism by allowing for curved yield lines which accurately predict the column flange flexural yield loads in both unstiffened and normally-stiffened connections. By application of the yield line theory to the end-plate, they developed a convenient design approach.

3.2 Modelling of Moment-Rotation Behaviour

The most commonly used approach to describe the moment-rotation behaviour of connections has been the curve-fitting of experimental data to simple expressions. Alternatively, simple analytical procedures have been used to predict connection behaviour in those cases where no test data were available for the specific connection details. Numerous connection tests have been performed. They have yielded a rather large body of M-ϕ data, which have been used in the development of variety of M-ϕ models.

1. Linear models

Rathbun (1936), Monforton and Wu (1963) and Lightfoot and LeMessurier (1974) used the single stiffness linear model. With that model the initial stiffness is used to represent the connection behaviour over its entire loading range. The model is effective only for moments within the serviceability limit of the connection.

A bilinear model was proposed by Tarpy and Cardinal (1981). In this model the initial slope of the moment-rotation line is followed, at a particular transition moment, by a second line with a smaller slope. The bilinear model provides a better representation of the connection behaviour than the single linear stiffness model does.

2. Polynomial models

Frye and Morris (1976) developed a polynomial model to predict the behaviour of several common connection types. They first used statistical regression approaches to curve fit experimental data. In this model, the M-ϕ relationship is represented by an odd power polynomial of the form

$$\phi = C_1 (KM)^1 + C_2 (KM)^3 + C_3 (KM)^5 \qquad (3.1)$$

where K is a standardization parameter dependent upon the connection type and its geometric parameters, and C_1, C_2 and C_3 are curve fitting constants. This model is able to represent the M-ϕ relationship reasonably well. However, due to its inherent oscillatory nature, it may yield erratic approximations to the connection stiffness at various rotation values (Jones et. al. 1980).

3. Exponential model

Yee and Melchers (1986) proposed the following four-parameter exponential model to represent the nonlinear M-ϕ behaviour of bolted connections.

$$M = M_p \left(1 - \exp\left(\frac{-(R_i - R_p + C\phi)\phi}{M_p}\right)\right) \qquad (3.2)$$

where M_p is the plastic moment capacity of the connection, R_i is the initial elastic connection stiffness, R_p is the strain hardening connection stiffness, and C is a constant that controls the slope of the curve.

Lui and Chen (1986) proposed the multiparameter exponential model

$$M=\sum_{j=1}^{m} C_j\,(1-e^{-|\phi|/2j\alpha})+M_0+R_{kf}|\phi| \qquad (3.3)$$

where M_o is the starting value of the connection moment to which the curve is fitted, R_{kf} is the strain-hardening stiffness of the connection, α is a scaling factor, and C_j is a curve fitting constant. Kishi and Chen (1986) refined the Lui-Chen exponential model, expressing it in the following form.

$$M=\sum_{j=1}^{m} (1-e^{-|\phi|/2j\alpha})+M_o+\sum_{J=1}^{M} D_K(\phi-\phi_k)\,H[\phi-\phi_k] \qquad \mathbf{(3.4)}$$

where M_o and α are defined as in Eq. (3.3), D_k is a constant parameter for the linear portion of the curve, ϕ_k is the initial rotation for the linear component of the curve, and C_j and D_k are curve fitting constants obtained from a linear regression analysis, and $H[\phi]$ is Heaviside's step function (unity for $\phi > 0$, zero for $\phi < 0$).

Wu and Chen (1990) proposed a three-parameter exponential model to represent the moment-rotation behaviour of top and seat angle connections with or without double web angles. It has the following form.

$$\frac{M}{M_u}=n\,(\ln(1+\frac{\phi}{n\phi_o})) \qquad (3.5)$$

where M_u is the idealized elastic-plastic mechanism moment, ϕ is a reference rotation, and n is a shape parameter.

4. Ramberg-Osgood function

Ang and Morris (1984) used a standardized Ramberg-Osgood function (Ramberg and Osgood,1943) in the power form.

$$\frac{\phi}{\phi_o} = \frac{KM}{(KM)_o}\left(1 + \left(\frac{KM}{(KM)_o}\right)^{n-1}\right) \tag{3.6}$$

In this equation ϕ_o, $(KM)_o$ and n are constants to be evaluated from a Ramberg-Osgood curve fitting program.

5. Richard-Abbott function

An M-ϕ model using the Richard-Abbott (1975) function, expressed in the following form, was developed by Attiogbe and Morris (1991).

$$M = S_o\phi\left[\frac{1-\frac{S_p}{S_o}}{\left(1+\left|\left(\frac{1}{\phi_o}-\frac{S_p}{\phi_o S_o}\right)\phi\right|^{n}\right)^{\frac{1}{n}}}+\frac{S_p}{S_o}\right] \tag{3.7}$$

In Eq. (3.7) $S_o=M_o/\phi_o$ is the initial slope, M_o is a reference moment, ϕ_o is a reference rotation, S_p is the slope of the asymptote for large values of ϕ, and n is the parameter that defines the sharpness of the curve. The function, which requires only four parameters to define it, was shown to be more accurate and versatile than others that have been used for the purpose.

3.3 Standardized Moment - Rotation Function

In the preceding section, a number of connection models and their mathematical forms have been presented in general terms. In order to account for connection behaviour in a structural analysis program it is necessary for the program to have the connection M-ϕ functions available to it. Because it would be prohibitively expensive to store the M-ϕ relationships for all practical connection types and sizes, an attractive solution is to derive and store a single " standardized " function expressed in terms of the geometric parameters, for each connection type (double web angle, end plate,etc.). Then, when the structural analysis computer program is run the geometric parameters can be input and substituted into the standardized function to generate the specific moment-rotation functions for all connections in the structure.

The standardization procedure was first used by Sommer (1969) to describe the M-ϕ behaviour of welded header plate connections. The procedure was subsequently generalized by Frye and Morris (1976), Altman et al. (1982) and Ang and Morris (1984).

Frye and Morris (1976) developed M-ϕ equations for seven types of commonly used connection types. In their model, the M-ϕ relationship was represented by an odd power polynomial of the form:

$$\phi=C_1(KM)^1+C_2(KM)^3+C_3(KM)^5 \tag{3.8}$$

In Eq. (3.8), K was expressed as a product of the significant geometric parameters for the connection, each raised to an exponent evaluated in a curve fitting process. For example, for the double web angle connection, their standardized M-ϕ function was

$$\phi=3.66(KM)\times10^{-4}+1.15(KM)^3\times10^{-6}+4.57(KM)\times10^{-8} \tag{3.9}$$

where

$$K=d^{-2.4}t^{-0.23}g^{1.60} \tag{3.10}$$

In Eq. (3.10)

d = depth of beam

t = thickness of web angle, and

g = gage of bolts to column flange

Altman et al. (1982), following the procedure used by Frye and Morris, developed prediction equations to describe the M-ϕ behaviour for strap angle connections and top and seat angle connections with double web angles. Their equation had the following form.

$$\phi=0.223\times10^{-4}(KM)+0.185\times10^{-7}(KM)^3+0.319\times10^{-11}(KM)^5 \tag{3.11}$$

where

$$K=t_a^{-1.1281}d^{-1.2870}t_w^{-0.6941}g^{1.3499}l_t^{-0.6941} \qquad (3.12)$$

in which t_a is the top angle thickness, d is the beam depth, t_w is the web angle thickness, l_t is the length of the top angle, and g is the distance from the heel of the top angle to the lower edge of the fastener. All parameters in Eq. (3.10) are expressed in inches.

Ang and Morris (1984) used a standardized Ramberg-Osgood (1943) function of form:

$$\frac{\phi}{\phi_o}=\frac{KM}{(KM)_o}\left[1+\left(\frac{KM}{(KM)_o}\right)^{n-1}\right] \qquad (3.13)$$

where ϕ_o, $(KM)_o$, and n are constants that define the shape of the standardized function. Again Parameter K, which accounted for the effects of the connection size parameters, had the form:

$$K=\prod_{j=1}^{m} q_j^{a_j} \qquad (3.14)$$

where

q_j =numerical value of the jth connection geometric parameter

a_j =dimensionless exponent which indicated the effect of the jth geometric parameter

m =number of parameters

They derived standardized moment rotation prediction equations for single web angle, double web angle, header plate, top and seat angle and strap angle connections. Since the standardization process involved the averaging of all available experimental moment-rotation data, reasonably good accuracy was obtained. The differences between the predicted moments and the experimental data on which they were based were reported to be in the range of 3 to 18 percent.

Attiogbe and Morris (1991) adapted the Richard-Abbott function to express the M-ϕ relationship in terms of connection size parameters. The function was expressed in the following form.

$$\frac{M}{M_o}=\frac{\phi}{\phi_o}\left[\frac{1-S_p\frac{\phi_o}{M_o}}{\left[1+\left|\left(\frac{1}{\phi_o}-\frac{S_p}{M_o}\right)\phi\right|^n\right]^{\frac{1}{n}}}+\frac{S_p\phi_o}{M_o}\right] \qquad (3.15)$$

While earlier standardized functions used a single parameter K to account for the influence of all of the geometric parameters, Attiogbe and Morris used multiple linear regression analysis to express each of the four Richard-Abbott parameters in terms of the connection geometry. For example, for the double web angle connection the derived parameters were

$$\phi_o=(t^{0.595}g^{-2.817}l^{4.737}d^{-0.784}b^{-5.957})\times 10^{-3}$$

$$M_o=t^{1.136}g^{-1.515}l^{1.139}d^{0.258}b^{0.309}$$

$$n=t^{0.522}g^{1.564}l^{-1.073}d^{-0.737}b^{1.704} \qquad (3.16)$$

$$S_p=(t^{0.955}g^{2.044}l^{-4.445}d^{0.327}b^{7.555})\times 10^{3}$$

where,

t = angle thickness, mm

g = gauge of column flange bolts, mm

l = angle length, mm

d = beam depth, mm; and

b = number of bolts per angle leg on column flange.

The units for ϕ_o, M_o and S_p are radians, kN m and kN m/radian, respectively.

In their study standardized M-ϕ equations for single web angle, double web angle, header plate, top and seat angle, and strap angle connections were derived using a multiple regression analysis procedure. From the comparison it was concluded that the Richard-Abbott function provided better fits than did the Ramberg-Osgood function.

3.4 Modelling of Connection Behaviour Under Cyclic Loading

While most connection tests have involved monotonic loading only, connections in frames normally experience some form of cyclic loading. The M-ϕ data from cyclic loading tests suggest that the unloading branch of the M-ϕ curve approximates a double mirror image of the loading branch. Following Moncarz and Gerstle (1981) who used a bi-linear M-ϕ function, Huang and Morris (1991) employed five functions, all derived from the Richard-Abbott function, to model the M-ϕ behaviour of connections under cyclic loading.

3.5 Connections of Composite Beams to Steel Columns

Although construction involving steel beams composite with floor slabs is predominant in North America, though not in Europe, only a few researchers have studied the behaviour of connections between steel columns and composite beams.

Wald and Parik (1990) proposed the following power model to represent the M-ϕ curve for connections in composite construction. Their prediction equation was expressed as:

$$\phi = \frac{M}{C}\left(1 - \frac{M}{M_u}\right)^n \tag{3.17}$$

where C is the initial stiffness of the connection, M_u is its ultimate moment capacity and n is a variable accounting for the dissipated energy. Parameter n was evaluated using experimental data.

Ammerman and Leon (1990) employed a tri-linear model for the seat angle and web clip connection. Their tri-linear curve was obtained from an exponential moment curve developed by Lin (1987). Wald (1991) employed an exponential expression proposed by Frye and Morris (1976) for composite beam-to-column connections.

M-ϕ prediction equations for composite connections under cyclic loading have not yet been proposed. It would appear that the Richard-Abbott function can provide accurate fits to the M-ϕ behaviour of composite, as well as non composite, connections.

However, the lack of symmetry of the moment-rotation behaviour of the composite connection must be represented.

3.6 Treatment of Connection Behaviour in Design Specifications

Traditionally, connection deformation and its effect on structures have been dealt with only indirectly in steel design specifications. In recent years, however, there has been a trend toward the explicit recognition of connection behaviour. The treatment of beam-to-column connections in three important steel design specifications is described below.

1. Canadian Standards Association (CSA)

CSA Standard CAN3-S16.1M-89 (CSA,1989) is a limit states design specification for steel structures in Canada. It specifies that structures shall be designed assuming either "continuous construction" or "simple construction". In continuous construction the beams and girders are to be assumed either rigidly framed or continuous over supports. Connections are to be designed to resist bending moments and internal forces calculated on the assumption that the original angles between intersecting members remain unchanged as the structure is loaded. In simple construction the ends of beams and girders are to be assumed to be free to rotate under load, in the plane of loading. Resistance to lateral load is to be provided by bracing or shear walls or a part of the structure designed as continuous construction. Subject to certain conditions, a frame designed to resist gravity loads on the basis of simple construction may be proportioned to resist lateral loads through the moment resistance of certain specified connections. The specification does not explicitly recognize the non-linear M-ϕ behaviour exhibited by virtually all beam-to-column connections.

2. American Institute of Steel Construction (AISC)

The AISC Load and Resistance Factor Design (LRFD) specification (AISC,1986) permits two basic types of construction - Fully Restrained (FR) and Partially Restrained (PR). Fully Restrained construction assumes rigid connections. Partially Restrained construction describes those conditions where either the connections lack sufficient strength to transfer the beam end moments into the columns, or they lack sufficient stiffness to virtually eliminate rotational deformations between the beams and the columns. The specification requires that when connection restraint is relied upon in the design of frames, the capacity of the connection for the desired restraint must be established by analytical or empirical methods. It thus recognizes the effect of connection behaviour in general, though it provides no guidance on how to model it.

3. EUROCODE 3: Common Unified Rules for Steel Structures

EUROCODE 3 (1990) defines three framing types - simple, continuous and semi-continuous - and specifies the types of beam-to-column connections required for each.

The connection types are defined using a system which classifies them by rigidity as "nominally pinned", "semi-rigid" and "rigid", and by strength as "partial-strength" and "full-strength". The connection M-ϕ characteristics of the connections assumed in design must be based on theory supported by experimental evidence. They must define the initial stiffness, the ultimate moment resistance and the rotational deformation capacity. The specification thus requires that connection behaviour be carefully modeled and accounted for in the design of frames.

3.7 Classification of Connections

When a frame is designed the connection characteristics are not known in advance. At most, the designer may have decided to use relatively flexible connections, as in a braced frame, relatively rigid ones, as in a moment-resistant frame, or ones that are intermediate between the two extremes. Thus for preliminary design purposes it is not possible to model connection behaviour precisely. However, that behaviour may be approximated using a connection classification system which permits connection M-ϕ behaviour to be expressed in terms of the properties of the connected members. One such classification system has been incorporated into EUROCODE3 (1990) and at least two other similar ones have been proposed (Bjorhovde et. al. 1988; Bijlaard and Steenhuis 1991).

The connection classification system incorporated into EUROCODE3 defines connection types in terms of the strength and stiffness of the connected beam. In terms of stiffness, the system is illustrated in Fig. 3.1, for braced frames and in Fig 3.2, for unbraced frames. The connection is classified as either "nominally pinned", "semi-rigid" or "rigid", depending upon its M-ϕ behaviour. Nominally pinned connections sustain large rotational deformation while transmitting only small moment, rigid ones permit only small relative rotation between the beam and column while transferring relatively large moment, and semi-rigid ones are intermediate between rigid and nominally pinned. The boundary between the rigid and semi-rigid classifications is tri-linear, while that between the semi-rigid and nominally pinned ones is linear. The connection moment, M, is expressed as a fraction of the beam plastic moment, M_p, while its rotational deformation, ϕ_p, is expressed as a function of

$$\phi_p = M_p L_b / EI_b.$$

where,

E = modulus of elasticity

I_b= second moment of area of the beam

L_b= beam length.

The boundary between nominally pinned and semi-rigid connections for both unbraced and braced frames has a slope $S = 0.5\ EI_b/L_b$. In terms of strength, the connection is classified as either "full-strength" or "partial-strength", depending whether its ultimate moment resistance is equal to or less than M_p for the beam.

While EUROCODE3 uses a tri-linear function for the boundary between rigid and semi-rigid connection classes, an attractive alternative for computer use would be a continuous function. For example, the Richard-Abbott function can be fitted to the rigid - semi-rigid boundary for full strength braced-frame connections as follows. Since the tri-linear boundary is flat-topped, Richard-Abbott parameter $S_p = 0$ and $M_o = M_p$ for the beam. Since the initial asymptote for the Richard-Abbott function, M_o/ϕ_o, equals the initial slope, $8EI_b/L_b$, of the tri-linear boundary, parameter $\phi_o = M_oL_b/8EI_b$. Finally, because curve fitting tests have demonstrated that the accuracy of the fit is insensitive to the value of Richard-Abbott parameter n, it is convenient to use n = 2.6, the ratio of the slopes of the initial and second segments of the tri-linear boundary. Similarly, it can be shown that the rigid - semi-rigid boundary for connections in braced frames can be represented by a Richard-Abbott function with the parameters:

$M_o = M_p$

$S_p = 0$

$\phi_o = M_pL_b/25EI_b$

$n = 7.$

For partial strength connections, M_o and ϕ_o are prorated appropriately. Fig 3.3 shows the EUROCODE3 full strength connection classification boundaries with the corresponding Richard-Abbott functions superimposed.

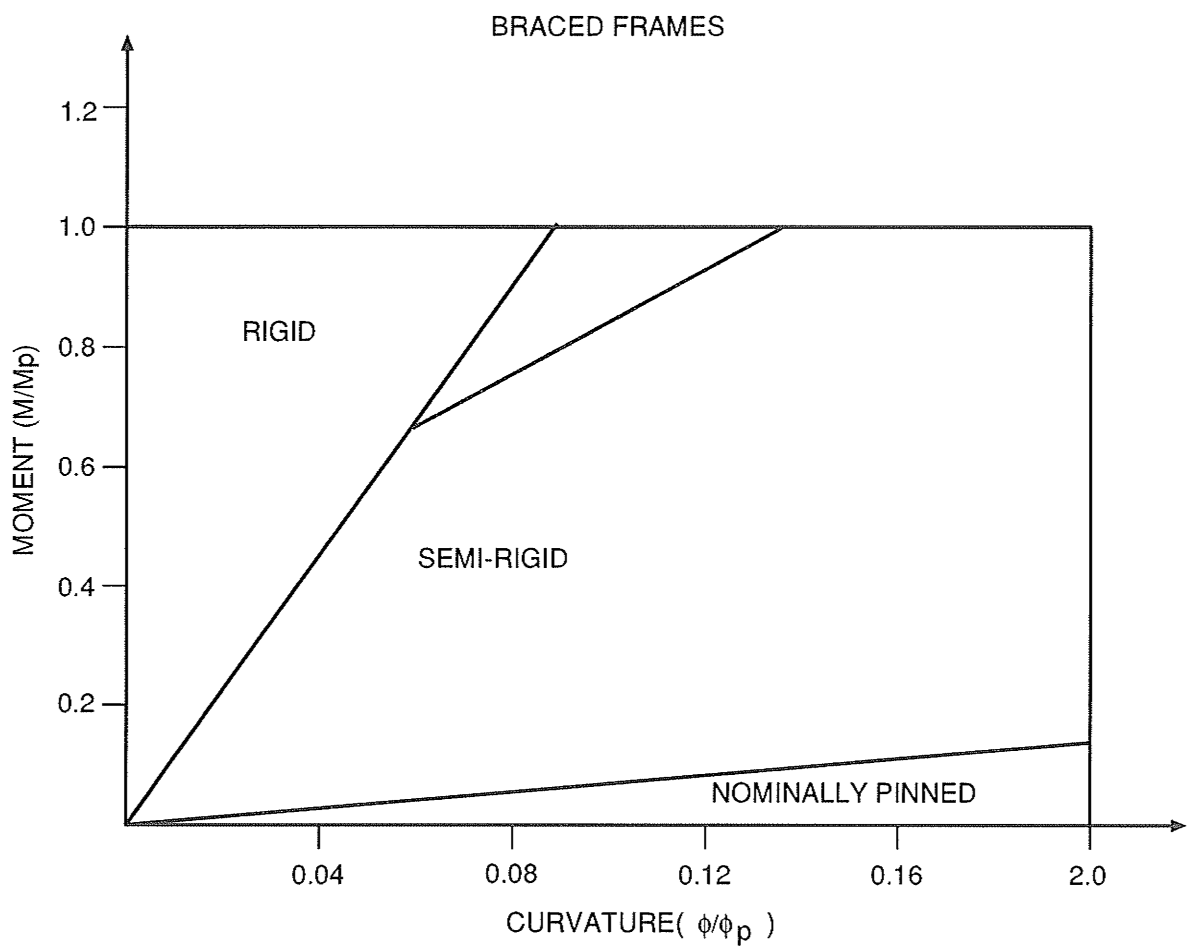


Fig. 3.1 EUROCODE3 Connection Classification for Braced Frame

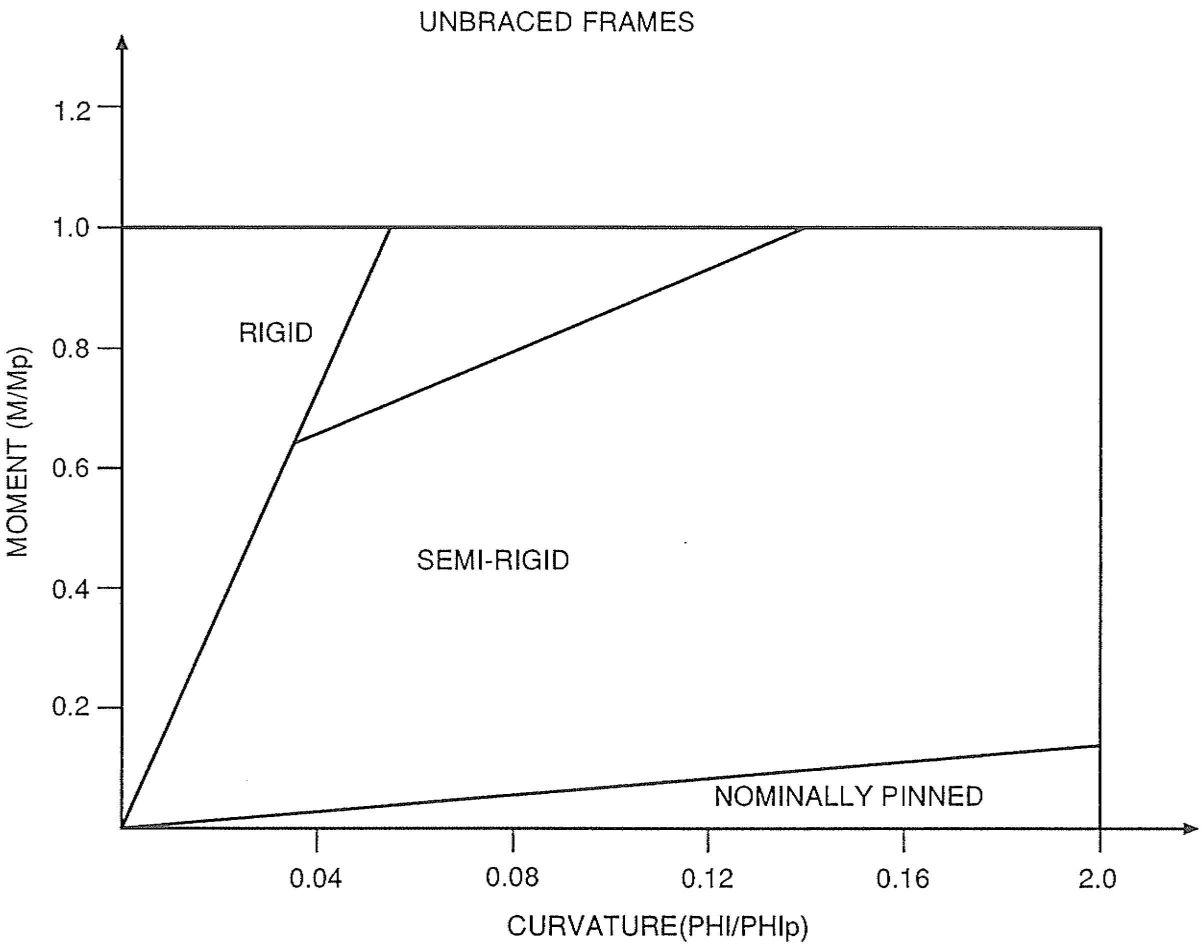


Fig. 3.2 EUROCODE3 Connection Classification for Unbraced Frame

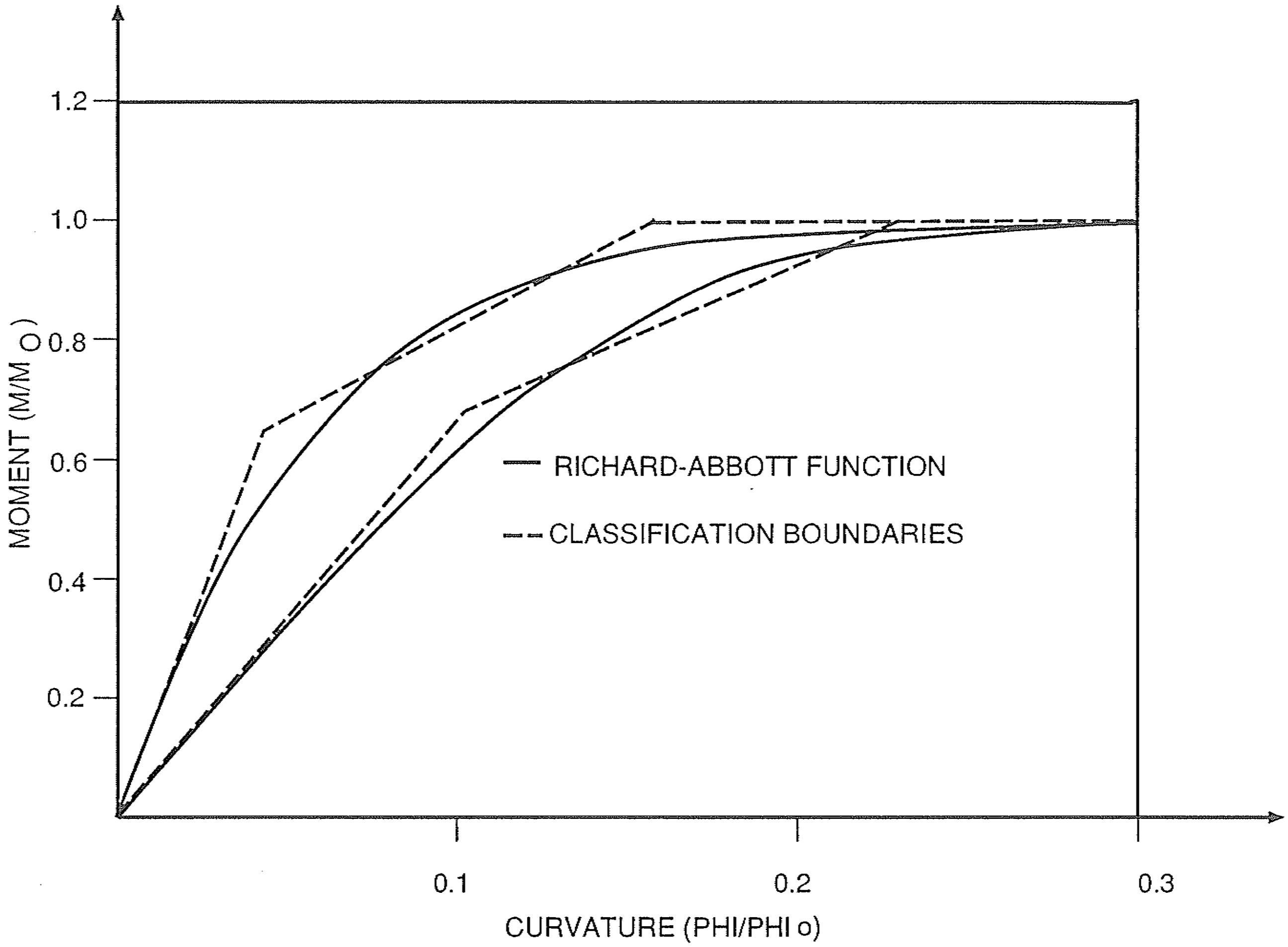


Fig. 3.3 EUROCODE Connection Classification by Richard-Abbott Function

CHAPTER 4

ANALYSIS OF FLEXIBLY CONNECTED FRAMES

4.1 Modelling of Structure and Loading

4.1.1 Structure Idealization

For the purpose of modelling the structure and loading, two types of right hand coordinate system are used. A single global system, with vertical and horizontal axes, is used to express all joint coordinates, all joint loads and the computed displacements of all joints. Any load or displacement vector expressed in the global coordinate system is identified by a prime. For example, D_J' is a displacement vector at joint J, expressed in the global system.

Each member is assumed to have a direction and a local coordinate system associated with it. For example if member AB has a direction from A to B, its local X_1-axis is coincident with the member axis and its X_2 and X_3 axes coincide with the principal axes of the cross-section. All member loadings and end forces are expressed in the appropriate local coordinate systems. Any force or displacement quantity expressed in a local system is identified by the absence of a prime. For example, P_B is a force vector at point B, expressed in a local system.

The structure is composed of a rectangular layout of beams and columns plus optional braces. All floors are assumed to act as rigid diaphragms in resisting in-plane forces, while the floor beams provide their normal stiffnesses in resisting out-of-plane forces. Composite action between the beams and the floor diaphragm is not considered.

The beam-to-column connections are associated with the beams and accounted for in the beam stiffness matrices and fixed-end-force vectors. The connections are assumed to have negligible size as compared to the lengths of members. Furthermore, they are assumed to permit no direct shearing deformation, axial deformation or torsional deformation. Since the floor diaphragms are assumed to be rigid in plane, the stiffnesses of the typical connection with respect to displacement components in the plane of the floor are assumed to be infinite. All joints are assumed to lie at floor level.

The entire structure is assumed to be linearly elastic except for the connections, which are assumed to behave non-linearly even at working load levels. Consequently an iterative procedure, in which the non-linear connection stiffnesses are linearized and modified in successive cycles of linear analysis, is employed.

4.1.2 Column Stiffness Matrix

The typical column is modeled as a line element with end force and displacement components as shown in Fig. 4.1. The stiffness matrix expressed in the local system can be resolved into two contributions. Thus

$$K=K_D+K_G \tag{4.1}$$

where

K_D =first order elastic stiffness matrix, and

K_G =geometric stiffness matrix, which accounts for the effect of the axial force P on the bending stiffnesses of the member.

Furthermore, the stiffness matrix can be expressed in partitioned form as

$$\begin{bmatrix} P_U \\ P_L \end{bmatrix} = \begin{bmatrix} K_{UU} & K_{UL} \\ K_{LU} & K_{LL} \end{bmatrix} \begin{bmatrix} D_U \\ D_L \end{bmatrix} \tag{4.2}$$

where subscripts U and L refer to the joints at the upper and lower ends of the column respectively. In Eq. 4.2,

$$P_U = \left[\, p_{U1} \;\; p_{U2} \;\; p_{U3} \;\; m_{U1} \;\; m_{U2} \;\; m_{U3} \,\right]^T \tag{4.3}$$

and

$$P_L = \left[\, p_{L1} \;\; p_{L2} \;\; p_{L3} \;\; m_{L1} \;\; m_{L2} \;\; m_{L3} \,\right]^T \tag{4.4}$$

are the force vectors at the upper and lower ends of the column, and

$$D_U = \left[\, \delta_{U1} \;\; \delta_{U2} \;\; \delta_{U3} \;\; \theta_{U1} \;\; \theta_{U2} \;\; \theta_{U3} \,\right]^T \tag{4.5}$$

$$D_L = \left[\, \delta_{L1} \;\; \delta_{L2} \;\; \delta_{L3} \;\; \theta_{L1} \;\; \theta_{L2} \;\; \theta_{L3} \,\right]^T \tag{4.6}$$

are the displacement vectors at the upper and lower ends. All of the above vectors are expressed in the local coordinate system for the column.

It can be shown (Ang & Morris, 1984, Chen & Lui, 1990) that in the partitioned stiffness matrix

$$K_{UU} = \begin{bmatrix} \frac{AE}{L} & 0 & 0 & 0 & 0 & 0 \\ 0 & \frac{12EI_3}{L^3} & 0 & 0 & 0 & \frac{-6EI_3}{L^2} \\ 0 & 0 & \frac{12EI_2}{L^3} & 0 & \frac{6EI_2}{L^2} & 0 \\ 0 & 0 & 0 & \frac{GI_1}{L} & 0 & 0 \\ 0 & 0 & \frac{6EI_2}{L^2} & 0 & \frac{4EI_2}{L} & 0 \\ 0 & \frac{-6EI_3}{L^2} & 0 & 0 & 0 & \frac{4EI_3}{l} \end{bmatrix} + \begin{bmatrix} 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & \frac{-6P_1}{5L} & 0 & 0 & 0 & \frac{P_1}{10} \\ 0 & 0 & \frac{-6P_1}{5L} & 0 & \frac{-P_1}{10} & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & \frac{-P_1}{10} & 0 & \frac{-2P_1L}{15} & 0 \\ 0 & \frac{P_1}{10} & 0 & 0 & 0 & \frac{-2P_1L}{15} \end{bmatrix} \qquad (4.7)$$

$$K_{UL} = \begin{bmatrix} \frac{-AE}{L} & 0 & 0 & 0 & 0 & 0 \\ 0 & \frac{-12EI_3}{L^3} & 0 & 0 & 0 & \frac{-6EI_3}{L_2} \\ 0 & 0 & \frac{-12EI_3}{L^2} & 0 & \frac{6EI_2}{l^2} & 0 \\ 0 & 0 & 0 & \frac{-GI_1}{L} & 0 & 0 \\ 0 & 0 & \frac{-6EI_2}{L^2} & 0 & \frac{2EI_2}{L} & 0 \\ 0 & \frac{6EI_3}{L^2} & 0 & 0 & 0 & \frac{2EI_3}{L} \end{bmatrix} + \begin{bmatrix} 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & \frac{6P_1}{5L} & 0 & 0 & 0 & \frac{P_1}{10} \\ 0 & 0 & \frac{6P_1}{5L} & 0 & \frac{-P_1}{10} & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & \frac{P_1}{10} & 0 & \frac{P_1L}{30} & 0 \\ 0 & \frac{-P_1}{10} & 0 & 0 & 0 & \frac{P_1L}{30} \end{bmatrix} \qquad (4.8)$$

$$K_{LU} = (K_{UL})^T \qquad (4.9)$$

and

$$
K_{LL} = \begin{bmatrix} \frac{AE}{L} & 0 & 0 & 0 & 0 & 0 \\ 0 & \frac{12EI_3}{L^3} & 0 & 0 & 0 & \frac{6EI_3}{L^2} \\ 0 & 0 & \frac{12EI_2}{L^3} & 0 & \frac{-6EI_2}{L^2} & 0 \\ 0 & 0 & 0 & \frac{GI_1}{L} & 0 & 0 \\ 0 & 0 & \frac{-6EI_2}{L^2} & 0 & \frac{4EI_2}{I} & 0 \\ 0 & \frac{6EI_3}{L^2} & 0 & 0 & 0 & \frac{4EI_3}{L} \end{bmatrix} + \begin{bmatrix} 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & \frac{-6P_1}{5L} & 0 & 0 & 0 & \frac{-P_1}{10} \\ 0 & 0 & \frac{-6P_1}{5L} & 0 & \frac{P_1}{10} & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & \frac{P_1}{10} & 0 & \frac{-2P_1L}{15} & 0 \\ 0 & 0 & \frac{-P_1}{10} & 0 & 0 & \frac{-2P_1L}{15} \end{bmatrix} \quad (4.10)
$$

where,

I_1 = torsion constant for the column

I_2 = second moment of area about cross-sectional axis X_2

I_3 = second moment of area about cross-sectional axis X_3

E = modulus of elasticity

G = shearing modulus, and

A = member cross-sectional area

L = member length

p_1 = member axial force

4.1.3 Beam Stiffness Matrix

Consider a typical beam with linear-stiffness, flexible connections at A and B, as shown in Fig. 4.2. Because the beam is assumed to be bonded to the floor diaphragm its axial and horizontal shearing forces and deformations and its bending moment and curvature about vertical axes are assumed to be negligible. Thus only 3 degrees of freedom need be considered at each end of the beam, and the stiffness matrix for the beam plus connections, expressed in the local coordinate system, is defined by the following equation.

$$\begin{bmatrix} P_B \\ P_A \end{bmatrix} = \begin{bmatrix} K_{BB} & K_{BA} \\ K_{AB} & K_{AA} \end{bmatrix} \begin{bmatrix} D_B \\ D_A \end{bmatrix} \tag{4.11}$$

where

$$P_B = [\ p_{B2}\ \ m_{B1}\ \ m_{B3}\ ]^T \tag{4.12}$$

and

$$P_A = [\ p_{A2} m_{A1} m_{A3}\ ]^T \tag{4.13}$$

are the member end force vectors at ends A and B respectively, and

$$D_B = [\ \delta_{B2}\ \ \theta_{B1}\ \ \theta_{B3}\ ]^T \tag{4.14}$$

and

$$D_A = [\ \delta_{A2}\ \ \theta_{A1}\ \ \theta_{A3}\ ]^T \tag{4.15}$$

are the member end displacement vectors.

It can be shown (Ang and Morris, 1984) that the submatrices of the partitioned stiffness matrix for the beam plus connections can be expressed as follows.

$$K_{BB} = \begin{bmatrix} S1 & 0 & -S2 \\ 0 & S3 & 0 \\ -S2 & 0 & S4 \end{bmatrix} \tag{4.16}$$

$$K_{BA}=\begin{bmatrix} -S1 & 0 & -S5 \\ 0 & -S3 & 0 \\ S2 & 0 & S7 \end{bmatrix} \tag{4.17}$$

$$K_{AB} = (K_{BA})^{T} \tag{4.18}$$

and

$$K_{AA} = \begin{bmatrix} S1 & 0 & S5 \\ 0 & S3 & 0 \\ S5 & 0 & S6 \end{bmatrix} \tag{4.19}$$

where

$$S1=\frac{12EI}{L^3}\beta_1$$

$$S2=\frac{6EI}{L^2}\beta_2$$

$$S3=\frac{GI_1}{L}$$

$$S4=\frac{4EI}{L}\beta_5$$

$$S5=\frac{6EI}{L^2}\beta_2$$

$$S6=\frac{4EI}{L}\beta_4$$

$$S7=\frac{2EI}{L}\beta$$

and

$$\beta=\frac{3}{4\left(1+\frac{3EI}{S_A L}\right)\left(1+\frac{3EI}{S_B L}\right)-1}$$

$$\beta_1=\beta\left(1+\frac{EI(S_A+S_B)}{S_A S_B L}\right)$$

$$\beta_2=\beta\left(1+\frac{2EI}{S_B L}\right)$$

$$\beta_3=\beta\left(1+\frac{2EI}{S_A L}\right)$$

$$\beta_4=\beta\left(1+\frac{3EI}{S_B L}\right)$$

$$\beta_5=\beta\left(1+\frac{3EI}{S_A L}\right)$$

In the above expressions, S_A and S_B are the linear stiffness coefficients for the connections at A and B respectively. While they vary with changes in load, in the iterative procedure used in this study for a given linear analysis the connections are assumed to have constant stiffness.

4.1.4 Assembly of Structure Stiffness Matrix

When formulating the structure stiffness matrix the columns and beams are considered in turn and their contributions are computed and incorporated. The member stiffness coefficients are evaluated, appropriately transformed to the global coordinate system and then added, in the appropriate locations, to the structure stiffness matrix. Because of the assumption that the floors act as rigid in-plane diaphragms, the treatment of columns is different from that of beams, and a master-slave joint procedure is used to minimize the number of degrees of freedom.

Consider a typical column element UL between two floor diaphragms, as illustrated in Fig.4.3. The end displacement and end force vectors can be partitioned into out-of-plane sub-vectors and in-plane sub-vectors. Since the floor at each level is assumed to be a rigid in-plane diaphragm in resisting the in-plane forces, the in-plane forces and displacements for each column can be translated from the column ends to a common "master joint" in the floor diaphragm. The joint from which the in-plane components are translated is referred to as a "slave joint". In Fig 4.3, the master joint in the floor at the upper end of the column is designated V and U is a typical slave joint at that floor level. As shown in the figure, master joints V and M lie on a common, arbitrarily located vertical axis in the floors at the upper and lower ends, respectively, of the column. The force and displacement vectors P' and D', acting at the upper and lower ends, can be partitioned into in-plane and out-of-plane components. Thus the force-displacement relationships for the column are

$$\begin{bmatrix} P'_{Uo} \\ P'_{Vi} \\ P'_{Lo} \\ P'_{Mi} \end{bmatrix} = \begin{bmatrix} K'_{UU} & K'_{UV} & K'_{UL} & K'_{UM} \\ K'_{VU} & K'_{VV} & K'_{VL} & K'_{VM} \\ K'_{LU} & K'_{LV} & K'_{LL} & K'_{LM} \\ K'_{MU} & K'_{MV} & K'_{ML} & K'_{MM} \end{bmatrix} \begin{bmatrix} D'_{Uo} \\ D'_{Vi} \\ D'_{Lo} \\ D'_{Mi} \end{bmatrix} \qquad (4.20)$$

In the Eq. 4.20, subscripts i and o refer to in-plane and out-of-plane components, respectively. The stiffness submatrices relating out-of-plane force and displacement components have the form

$$K'_{LL} = RK_{LL_{oo}}R^{T} \qquad (4.21)$$

where

R = rotation transformation from the column local system to the global system and

K_{LLoo} = local system column stiffness submatrix relating out-of-plane force and displacement components at L.

Similarly, these relating in-plane force and displacement components have the form

$$K'_{MM} = T_{ML}K_{LLii}T^{T}_{ML} \qquad (4.22)$$

Where

T_{ML} = combined rotation-translation transformation matrix from column local system at L to global system at M and

K_{LLii} = local system column stiffness submatrix relating in-plane force and displacement components at L.

Finally, the stiffness submatrices relating in-plane to out-of-plane force and displacement quantities have the form

$$K^{/}_{ML} = T_{ML} K_{LUio} R^{T} \tag{4.23}$$

where

K_{LUio} = local system column stiffness submatrix relating in-plane force components to out-of-plane displacement components at U.

4.1.5 Beam Fixed-End-Force Vectors

The structure can be analyzed for two different types of loading - gravity loading and horizontal loading. The gravity load may be applied as beam loadings, as joint loadings or both. Joint loads are expressed in the global coordinate system while beam loads, which may be either uniformly distributed or a series of concentrated loads, are expressed in the local coordinate system. Horizontal load is expressed in the global coordinate system, as a three components vector applied at the master joint at each floor level.

The beam loading types are illustrated in Fig. 4.4. Assuming the beams to be bonded to the rigid in-plane floor diaphragms, the fixed end force vectors for the typical beam with end connections are the following. Under uniformly distributed load

$$P_B^F = \begin{bmatrix} p_{B2}^F \\ m_{B1}^F \\ m_{B3}^F \end{bmatrix} = w \begin{bmatrix} S_1\left(\frac{L^4}{8EI}+\frac{L^3}{2S_A}\right)-S_2\left(\frac{L^3}{6EI}+\frac{L^2}{2S_A}\right) \\ 0 \\ -S_2\left(\frac{L^4}{8EI}+\frac{L^3}{2S_A}\right)+S_4\left(\frac{L^3}{6EI}+\frac{L^2}{2S_A}\right) \end{bmatrix} \tag{4.24}$$

and

$$P_A^F = \begin{bmatrix} -wL-p_{B1}^F \\ 0 \\ -\frac{wL^2}{2}-p_{B1}^F L-p_{B2}^F \end{bmatrix} \tag{4.25}$$

where S_1, S_2 and S_4 are as defined in Section 4.1.3 and w is the intensity of the uniformly distributed load.

Similarly under a single concentrated gravity load located as shown in Fig. 4.4,

$$P_B^F = \begin{bmatrix} S_2\left(\frac{P.a^2}{2EI_3}+\frac{P.a}{S_A}\right)-S_1\left(\frac{P.a^3}{3EI_3}+\frac{P.a^2(L-a)}{2EI_3}+\frac{P.L.a}{S_A}\right) \\ 0 \\ S_2\left(\frac{P.a^3}{3EI_3}+\frac{P.a^2(L-a)}{2EI_3}+\frac{P.L.a}{S_A}\right)-S_4\left(\frac{P.a^2}{2EI_3}+\frac{P.a}{S_A}\right) \end{bmatrix} \tag{4.26}$$

$$P_A^F = \begin{bmatrix} -p-p_{B1}^F \\ 0 \\ -p.a-b.p_{B1}^F-p_{B2}^F \end{bmatrix} \tag{4.27}$$

where p, a and L are as shown in the figure.

4.2 Modelling of Connection Behaviour

Tests of common beam-to-column connection types under several cycles of load reversal reveal moment-rotation behaviour similar to that illustrated in Fig. 2.3. That behaviour has been modelled by five separate functions, as illustrated in Fig. 4.5, all derived from the Richard-Abbott function.

The initial loading curve is modelled by the Richard - Abbott function (function 1), and expressed as.

$$M=S_o\phi\left[\frac{1-\frac{S_p}{S_o}}{\left(1+\left|\left(\frac{1}{\phi_o}-\frac{S_p}{\phi_o S_o}\right)\phi\right|^n\right)^{\frac{1}{n}}}+\frac{S_p}{S_o}\right] \tag{4.28}$$

In the event of a moment reversal at point "a" on the curve, the following linear relationship (function 2), with a slope of S_o, is invoked.

$$M=S_o\left(\phi-\phi_a+\frac{M_a}{S_o}\right) \tag{4.29}$$

Function 2 is assumed to be applicable until at point "b" it intersects a line through point "o" with a slope of S_p. The value of moment at point b is expressed as follows:

$$M_b=\frac{S_o\phi_a-M_a}{\frac{S_o}{S_p}-1} \tag{4.30}$$

If the moment reversal continues, a double mirror image of the Richard-Abbott function (function 3) is applicable. It is expressed as

$$M=S_o(\phi-\phi_b)\left[\frac{1-\frac{S_p}{S_o}}{\left(1+\left|\left(\frac{1}{\phi_o}-\frac{S_p}{\phi_o S_o}\right)(\phi-\phi_b)\right|^n\right)^{\frac{1}{n}}}+\frac{S_p}{S_o}\right]+M_b \tag{4.31}$$

It is used until the moment again reverses, say at point "c", after which the following linear function, (function 4) with a slope equal to the initial-loading slope, is applicable.

$$M=S_o\left(\phi-\phi_a+\phi_c+\frac{M_a}{S_o}+\frac{M_c}{S_o}\right) \tag{4.32}$$

The moment value at point "d" is expressed as follows :

$$M_d=\frac{S_o\phi_a-S_o\phi_c-M_a-M_c}{\frac{S_o}{S_p}-1} \tag{4.33}$$

Finally, if the moment continues to increase in the original direction, function 5, the Richard - Abbott function, displaced from initial point "o" to point "d", is applicable. It is expressed as

$$M=S_o(\phi-\phi_d)\left[\frac{1-\frac{S_p}{S_o}}{\left(1+\left|\left(\frac{1}{\phi_o}-\frac{S_p}{\phi_o S_o}\right)(\phi-\phi_d)\right|^n\right)^{\frac{1}{n}}}+\frac{S_p}{S_o}\right]+M_d \tag{4.34}$$

The connection moment-rotation behaviour can be generated in either of two ways. For bolted-bolted double web angle connections, functions based on test data and expressed in terms of the connection geometric parameters (angle thickness and depth, bolt gages, etc.) have been built in to the program (Attiogbe and Morris,1990). Using them the connection geometric parameters are input and the moment-rotation functions are generated automatically by the program. Similar functions for other connection types are yet to be developed.

Alternatively, the Richard-Abbott parameters, M_o, ϕ_o, S_p and n for any connection M-ϕ curve can be input directly.

4.3 Non-linear Analysis Procedure

4.3.1 Analysis for Single Loading System

The stiffness matrix for the typical column includes coefficients which vary with the column axial force. That for the typical beam, with its associated end connections, includes coefficients which depend upon the connection moments. As a result, the elements in the structure stiffness matrix, which is formed by superimposing the beam and column stiffness matrices, depend upon the internal forces in the structure. Accordingly, an iterative analysis procedure involving repeated cycle of linear analysis and stiffness modification, is required. In the iterative procedure, linear beam-plus-connection and column stiffnesses are evaluated which are consistent with the nonlinear force-deformation behaviour of those members. Thus for a given loading system the final linear analysis gives the correct displacements and internal forces for the nonlinear structure.

The procedure can be demonstrated with reference to the typical connection moment-rotation diagram shown in Fig. 4.6. The nonlinear moment-rotation function for the connection is expressed as

$$M=f(\phi) \tag{4.35}$$

where $f(\phi)$ is a nonlinear function of the rotational deformation of the connection. This function is initially replaced by a linear one of the form

$$M=S_1\phi \tag{4.36}$$

where S_1 is the slope of the initial tangent to the M-ϕ curve. The moment-rotation functions for all other connections in the structure are similarly linearized, the corresponding member force - displacement relationships are determined, and a linear analysis is performed. The moments at all connections are then computed. If the moment at the connection originally considered is M_1, the corresponding rotational deformation is

$$\phi_1=\frac{1}{S_1}M_1 \tag{4.37}$$

However, the rotation, ϕ_1', can be calculated from the correct nonlinear relationship

$$M=f(\phi) \tag{4.38}$$

A better approximation to the connection moment - rotation function is thus,

$$M=S_2\phi \tag{4.39}$$

where

$$S_2 = \frac{M_1}{\phi'_1} \tag{4.40}$$

The equation above and similar relationships for the other connections are then used to calculate the new member force - displacement relationships and a second linear analysis is performed. Because a smaller stiffness has been assumed for the connection considered, its moment is reduced to M_2. The corresponding rotational deformation is ϕ_2. However, the nonlinear M-ϕ function yeilds a rotational deformation of ϕ_2'. It is used to compute a new stiffness, the stiffnesses of all other connections are similarly modified, and a third linear analysis is performed. The procedure is repeated until the rotational deformation of each connection, calculated from the linear relationship for the current cycle, is sufficiently close to the nonlinear relationship of the exact nonlinear M -ϕ function. The structural forces and displacements calculated in the final linear analysis cycle, which correctly account for the nonlinear connection behaviour, are then printed.

4.3.2 Analysis for Sequential Loading Systems

The iterative analysis procedure is described with reference to the moment-rotation diagram for a typical connection, as shown in Fig. 4.7. For the first loading system, function 1, expressed by Eq. 4.28, is selected automatically for each connection. Repeated cycles of linear analysis and connection stiffness modification are performed until the moments and rotational deformations at all connections correspond to points on their moment-rotation curves. Assuming that for the connection considered that point is "a" in the figure, the connection secant stiffness is S_{L1}. The connection moment, M_a, and rotation, ϕ_a, are stored, along with those for all other connections.

For the second loading system, an initial linear analysis is performed using the connection secant stiffnesses from the previous loading system. The moment increments at all connections are then evaluated in order to determine which connections experience increased moment under the new loading and which experience decreased moment. If the moment increment at a particular connection has the same sign as M_a, function 1 is retained, otherwise function 2 is invoked. Repeated cycles of linear analysis under loading system 2 are performed in order to evaluate new secant stiffnesses corresponding to points on the M-ϕ diagrams. For the connection considered, assuming that function 1 is retained, the secant stiffness for the second loading system is S_{L2} and the cumulative connection moment and rotational deformation are M_b and ϕ_b. All connection moments and rotations are again recorded.

Assuming a moment reversal at the connection considered under loading system 3, the connection secant stiffness under subsequent loadings are S_{L3}, S_{L4}, S_{L5}, etc. The procedure is repeated until all loading systems have been applied. No matter which function is invoked, the secant stiffness for the current iteration cycle is evaluated in terms of the following equation.

where

$$S_{secant} = \frac{\Delta M}{\Delta \phi} \tag{4.41}$$

ΔM = moment increment under current loading system, and

$\Delta\phi$ = rotation increment under current loading system.

That is, a secant stiffness is always used.

4.3.3 Evaluation of P - Δ Effects

The P-Δ effect calculation can be performed in either of two ways. The effect can be calculated using the iterative procedure recommended by the Canadian Standards Association (CSA, 1989). Alternatively, the load control Newton-Raphson method can be used to account for both the P-Δ and the P-δ effects. In the latter approach, the loading is applied incrementally. The deformed configuration of the structure at the end of each cycle of calculation is used as the basis for the formulation of the equilibrium equations for the next cycle. In a given analysis cycle, the structure is assumed to behave linearly. The procedure is explained with reference to the load-displacement diagram in Fig. 4.8. The steps corresponding to the application of the typical load increment $\Delta Q^{1'}$, are the following.

i) Apply the typical loading increment $\Delta Q_i^{'0}$

ii) Perform a linear analysis to compute the incremental joint displacements,$\Delta D_i^{'1}$,using $\Delta Q_i^{'0} = K_{s\,i-1}\, \Delta D_i^{'1}$, where $K_{s\,i-1}$ is the structure stiffness matrixat the beginning of the loading increment.

iii) Compute the vector of joint displacements, $D'^{1}_{i} = D'_{i-1} + \Delta D'^{1}_{i}$

iv) Compute the member end forces, and using equilibrium equations at all joints, compute the residual joint loads, $\Delta Q'^{1}_{i}$, the portions of the joint loads $\Delta Q'^{0}_{i}$ that are not equilibrated by the internal forces.

v) Compute a new structure stiffness matrix, $K_{si}^{1,}$ based upon the displaced position for the joints.

vi) Apply the residual joint loads, $\Delta Q'^{1}_{i}$ and perform a new linear analysis tocompute new incremental joint displacements $\Delta D'^{2}_{i}$, and update the cumulative displacement vector.

vii) Repeat steps iv),v) and vi) until the residual joint loads are negligibly small. The process is repeated until convergence occurs, at which time the next load increment is applied.

In the load control Newton-Raphson method, the unbalanced joint forces depend upon the interactive relationship between the deformed geometry of the structure and the external loads acting on it. Under non-proportional loading, gravity load and lateral load may be applied sequently and the deformation of the structure induced by the lateral load is independent of the gravity load. As a result, the load control Newton-Raphson approach can not be used directly when non-proportional loading systems are involved. In this study the method was used for proportional loadings only. The convergence of both the CSA iterative method and Newton-Raphson method are rapid, usually requiring only one or two cycles of iteration.

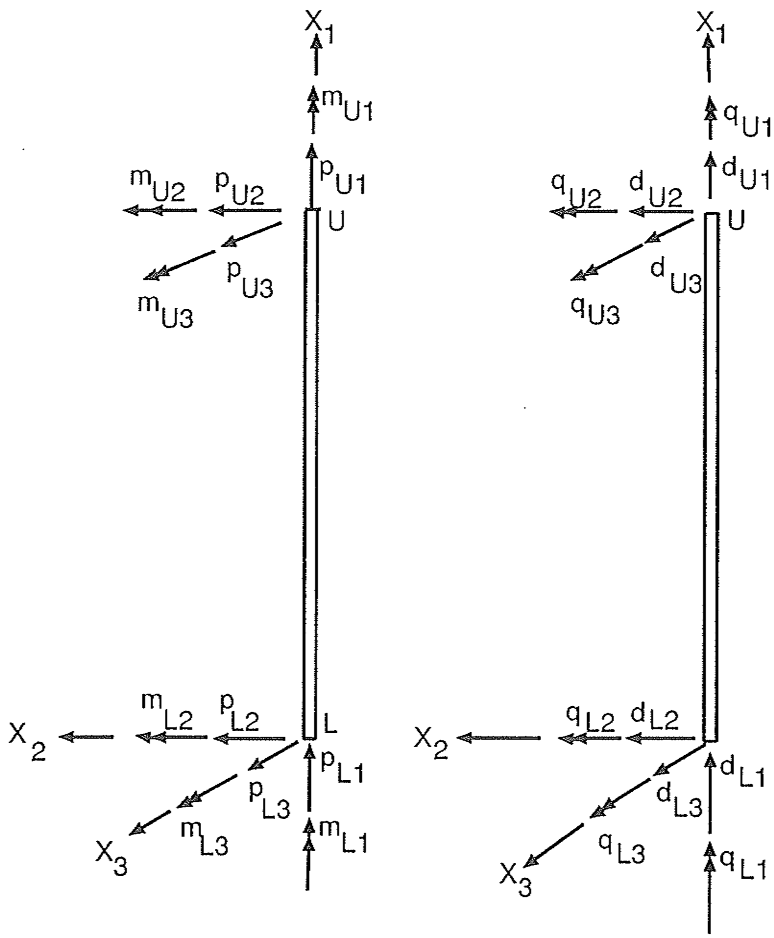


Fig. 4.1 Typical Column U - L

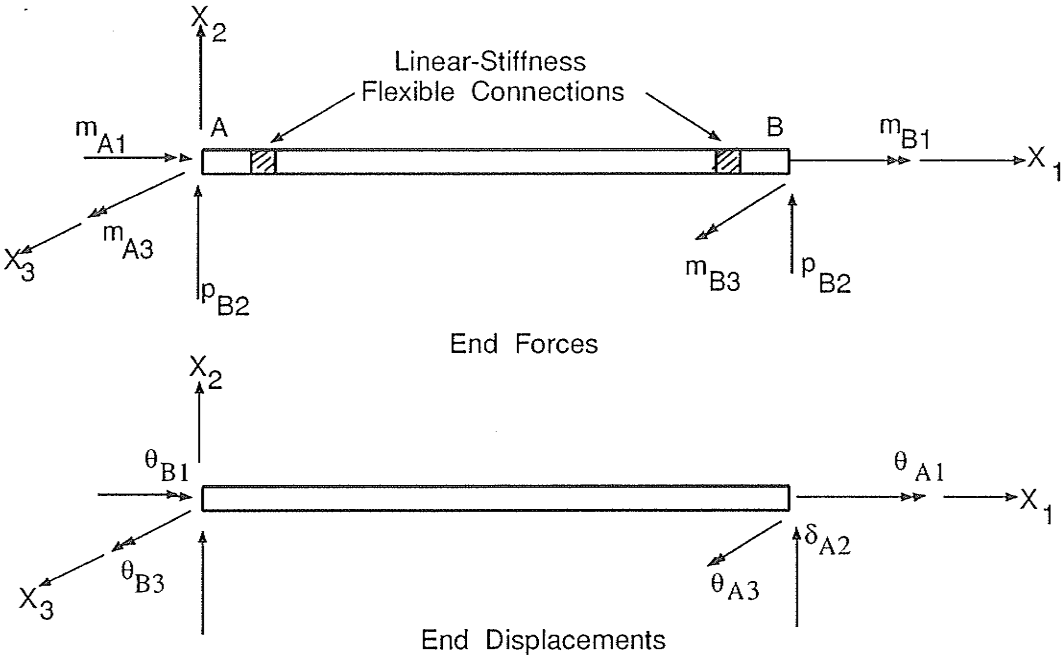


Fig. 4.2 Typical Beam A - B

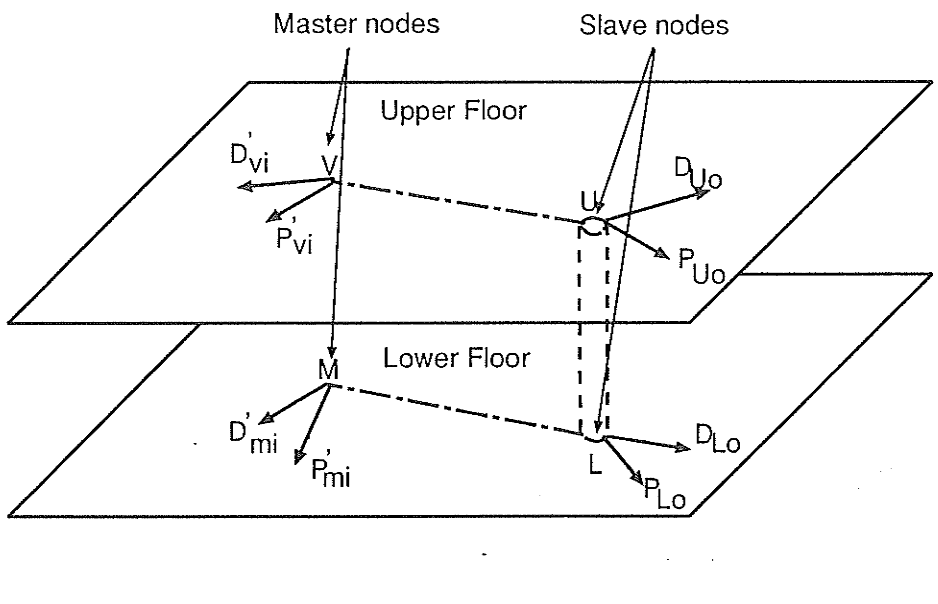


Fig. 4.3 End Forces and Displacements, Column U - L

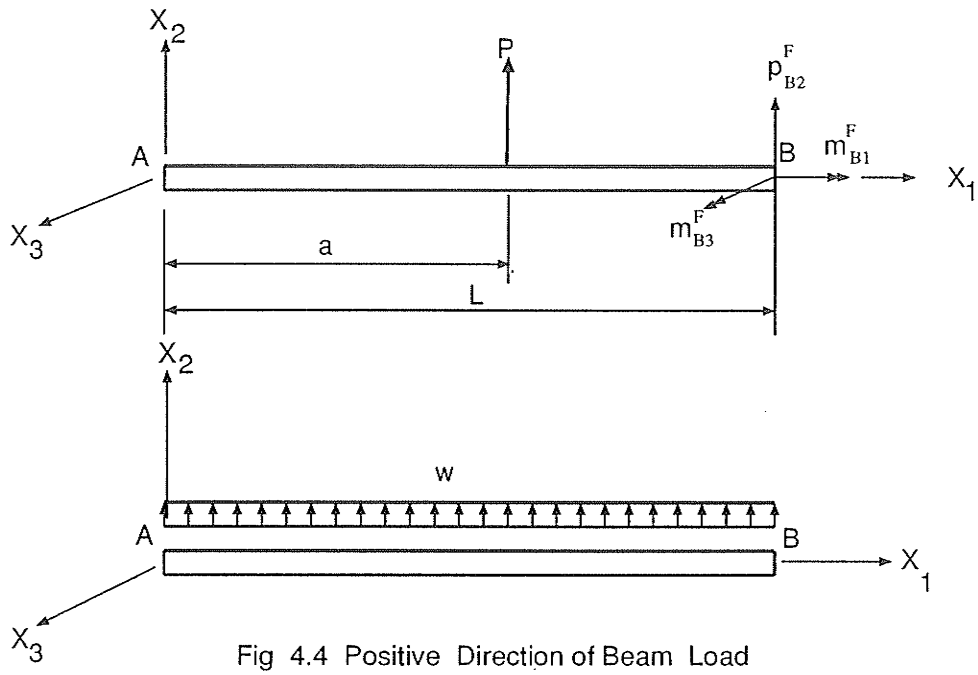


Fig 4.4 Positive Direction of Beam Load

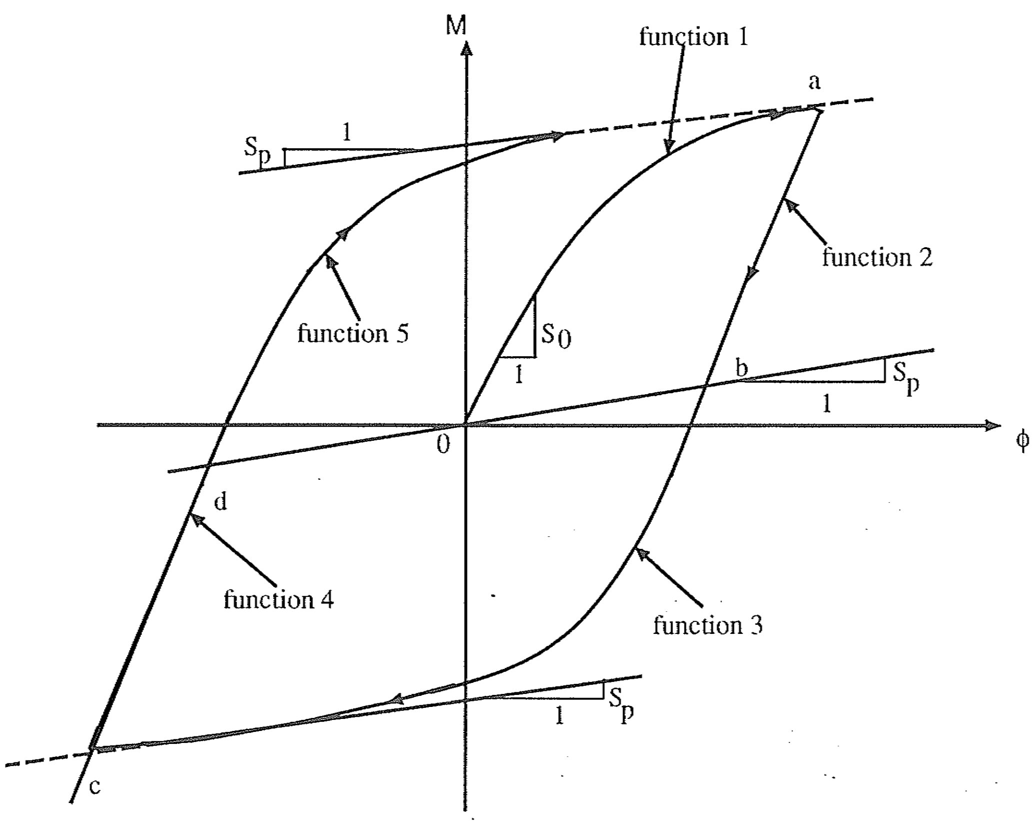


Fig. 4.5 Idealized Moment - Rotation Diagram

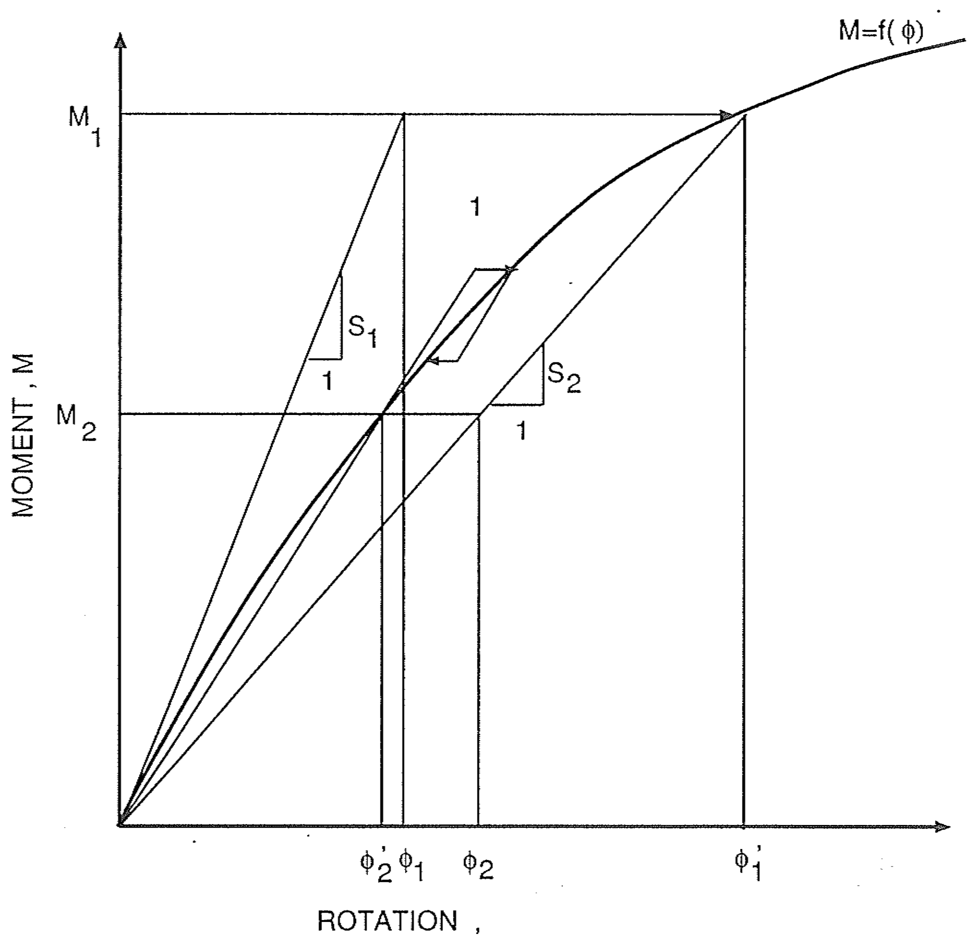


Fig. 4.6 Modification of Connection Flexibility

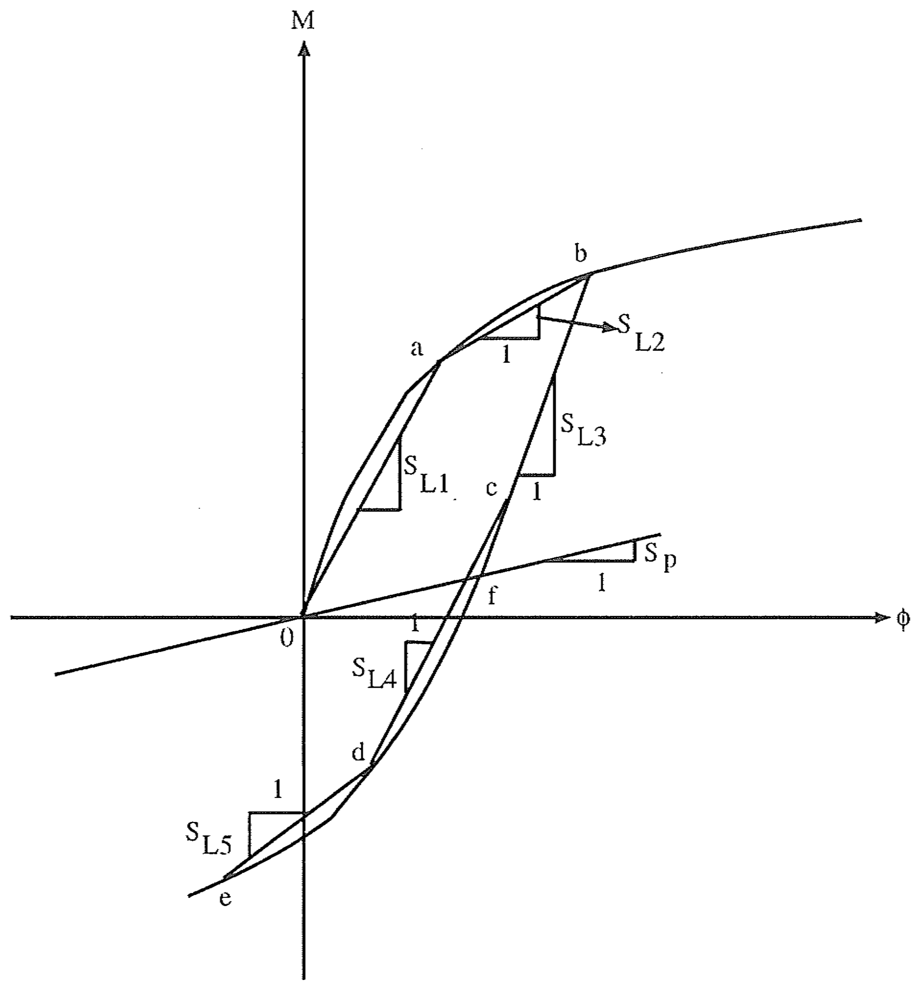


Fig. 4.7 Idealized Moment - Rotation Diagram for Sequence Load

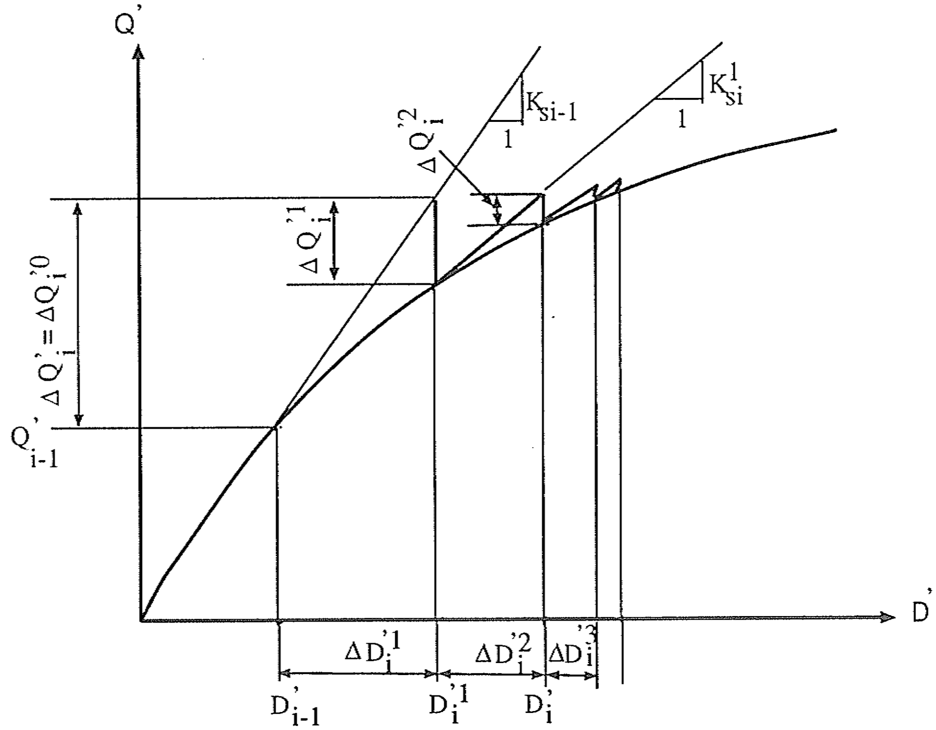


Fig. 4.8 Load - Displacement Diagram

CHAPTER 5

NONLINEAR ANALYSIS COMPUTER PROGRAM

5.1 Introduction

A structural analysis FORTRAN microcomputer program, FLEXMAIN, was developed in this study. The program performs the structural analysis of three-dimensional, flexibly connected rectangular steel frames loaded by any number of sequential, non proportional loading systems. The structure is assumed to have rigid in-plane floor diaphragms at all floor levels and bracing bays may be incorporated on any column line. The program is able to account for the moment versus rotational-deformation behaviour of the beam-to-column connections.

The nonlinear connection M - ϕ behaviour under either monotonic or cyclic loading can be modeled using a Richard-Abbott function and functions derived from it. The connection M - ϕ behaviour can be developed in either of two ways. For double web angle connections, a standardized version of the Richard-Abbott function and its related functions have been incorporated. The standardized function, which was obtained by curve fitting all available experimental M - ϕ data, is expressed in terms of the geometric parameters which influence the connection behaviour. Thus, for double web angle connections, it is necessary only input the connection geometric parameters and the M - ϕ functions are generated automatically.

For other connection types, it is necessary to input the four parameters which define a Richard-Abbott M - ϕ curve. Those parameters can be estimated by reference to a connection classification system, as described previously. Alternatively, if experimental M - ϕ data are available, a Richard-Abbott function can be fitted to them using a program developed by Attiogbe and Morris (1991). The Richard-Abbott parameters thus obtained can be input. Finally, any beam-to-column connection may be assumed to be rigid, to have a flexural moment release, or to have any linear flexural stiffness. All column bases are assumed to be fixed.

Either gravity loading or horizontal (wind) loading can be accommodated. Gravity loads may be applied as joint loads or as either uniformly distributed or concentrated member loads. Horizontal loading is applied as a three-component vector at the master node at each floor level.

For a given loading system, the program performs repeated cycles of linear analysis and connection stiffness modification in order to evaluate a linear secant stiffness for each connection such that moment and rotational deformation computed in the final linear analysis correspond to a point on its nonlinear M -ϕ diagram. Thus, the final linear analysis yields the correct structural displacements and internal forces. The iterative analysis procedure is repeated for each new loading system.

Using the load control Newton-Raphson method (Chen and Lui, 1991), the program account for column P-Δ effects. Alternatively, the program incorporates an iterative procedure (CSA, 1989) to account approximately for the P - Δ effects. As well, the effects of the column axial forces on their axial stiffness (the P - δ effects) are incorporated.

A flow diagram for the program is presented in Fig. 5.1

5.2 Data Input

Data input is interactive and the program is reasonably "user friendly". Thus the user is prompted for the necessary input, some data checking is done as the information is being input.

The first block of data describes the type of analysis desired. Either a single linear analysis or an iterative nonlinear analysis may be specified. For the former, linear stiffness values for all beam-to-column connections must be input. For the latter, the user must input the data needed to permit the generation of the connection moment-rotation functions. Next, the number of sequential loading system is specified. For a single monotonic loading system, that number is one. Finally, the user may specify either the approximate iterative procedure or the "exact' Newton-Raphson procedure to account for column P - Δ effects.

The second data block provides a description of the structure. The SI system of units is used throughout. The following data must be input:

a) the number of bays in each of the horizontal directions, X'_1 then X'_2;

b) the number of storeys;

c) the bay widths, in meters, in each horizontal direction, X'_1 then X'_2;

d) the storey heights, in meters, bottom storey to top

e) the X'_1 and X'_2 coordinates, in meters, of the master nodes at the floors;

f) for each beam and column, the torsion constant (in mm^4), the second moment of area about the member X_2 axis (in mm^4), the second moment of area about the member X_3 axis (in mm^4) and the cross sectional area (in mm^2);

g) the number of braces and the number of braced frames.

With the above information, the numbering and global coordinates of all joints, the topology of the structure (the interconnectivity of all members and joints) and the numbering and stiffness matrices for all members are generated automatically.

The third data block provides a description of the bracing elements. Cross bracing elements are numbered from the top floor to the bottom floor, from one braced bay to another. Each braced bay is defined as a bay having a cross-bracing element from the top floor to the bottom floor in that bay. The following data must be input.

a) the bracing element number;

b) the joint number at the lower and upper ends;

c) the bracing element cross-sectional area, in mm^2.

The fourth data block provides a description of the loading systems. For each loading system, the following data are required.

(1) Joint load (concentrated) :

- the loaded joint number;
- the load component in X_1' or X_2' direction;
- the load components in X_3' or moment about X_1' direction;
- moment about X_2' or X_3' direction.

(2) Beam concentrated load :

- the beam member number;
- load components in X_2' direction;
- the distance of the force from the start of the beam.

(3) Beam uniformly distributed load :

- the beam member number;
- uniformly distributed load in the X_2' direction.

5.3 Linear Analysis

The stiffness method is used to perform the linear analysis. The program employs an in-core, constant band width equation solver. Because of symmetry of the structure stiffness matrix, only the elements below the main diagonal are stored, as an one-dimensional array. The beam stiffness coefficients, which incorporate the effects of flexible connections at the ends of the beams, are recalculated each time they are needed. The member fixed-end forces are also dependent on the connection characteristics and thus they are recalculated for each linear analysis.

5.4 Program Output

The program output includes the following information.

a) a summary of the input, including the analysis type (linear or nonlinear) and associated assumptions, the structural geometry and member and connection properties, and descriptions of the loading systems considered.

b) intermediate analysis results, including the Richard-Abbott parameters used to represent the M - ϕ functions for all connections, connection moments and rotational deformations and the current M - ϕ functions following the application of each loading system, and the cumulative joint displacements and member end forces following the application of each loading system; and

c) final analysis results, including the final joint displacements and member end forces, final connection moments and rotational deformations, and a summary of the connection M - ϕ functions used for the final loading system.

The user has the option of suppressing the intermediate results.

5.5 Termination Criteria

Because of the nonlinear behaviour of the connections the load-deformation behaviour of the structure is nonlinear. Consequently an iterative analysis procedure, which involves repeated cycles of linear analysis, is used. In each cycle terms of the stiffness matrix are updated using the latest information obtained from the previous cycle of iteration. The convergence criterion compares all connection rotational deformations obtained in the current cycle with those obtained from the Richard-Abbott function. If the difference is less than a specified tolerance for each connection, convergence is assumed to have occurred. Thus, the convergence criterion for each connection is written as

$$\left|\frac{\phi_l-\phi_r}{\phi_l}\right|\leq\varepsilon \tag{5.1}$$

where

ϕ_l= connection rotation obtained from the current linear analysis

ϕ_r= connection rotation obtained from the non-linear Richard-Abbott function

ε = tolerance limit

The iterative calculation is needed also to account for the P-Δ and P-δ effects, because for a second-order analysis the equilibrium equations are formulated with respect to the deformed geometry of the structure. That geometry is not known in advance and is constantly changing with the applied loads. The deformed configuration of the structure at the end of each cycle of calculation is used as the basis for the formulation of equilibrium equations for the next cycle. The element end forces for the current cycle are calculated using updated element end displacements. They are then compared with those which were calculated from the previous cycle, and the unbalanced forces for all elements are calculated from :

$$\Delta Q=R_j-R_{j-1} \tag{5.2}$$

where

ΔQ = unbalanced force vector

R_j = member end force vector of current cycle

R_{j-1} = member end force vector of previous cycle

Thus the convergence criterion for each member end force can be written as

$$| \Delta Q | \leq \varepsilon \tag{5.3}$$

where ε = tolerance limit

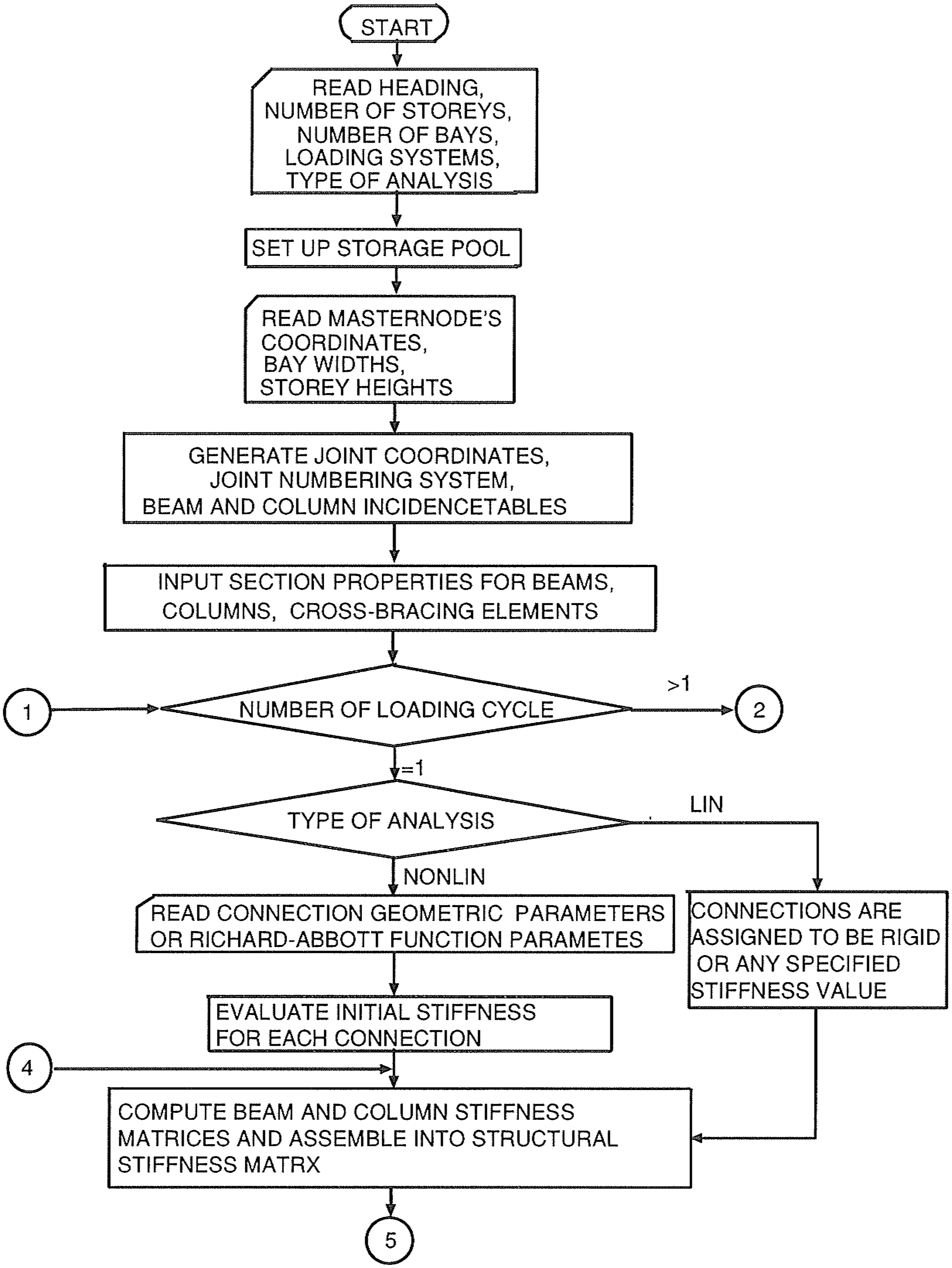


Fig.51. Program Flow Chart

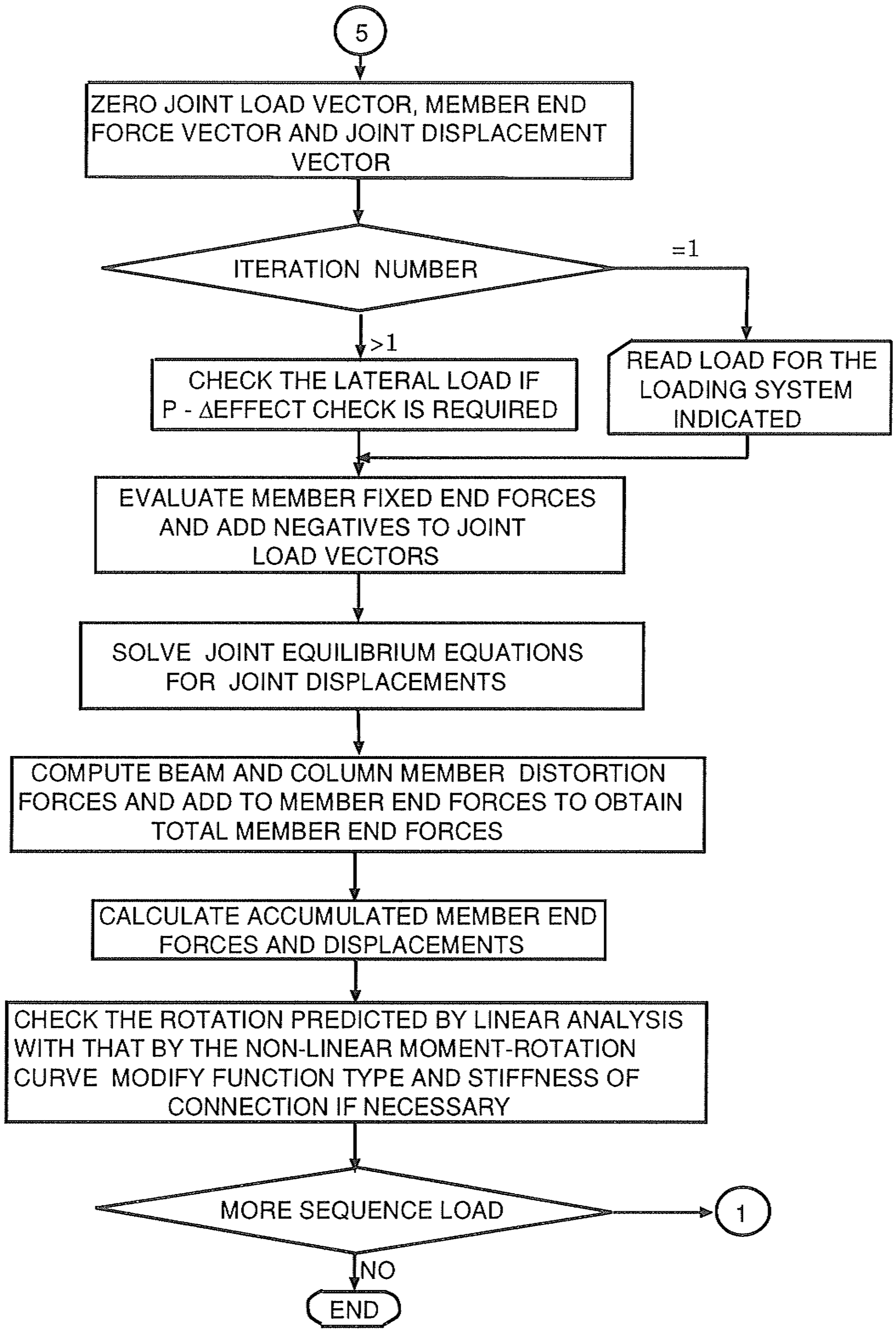


Fig. 5.1 Continued

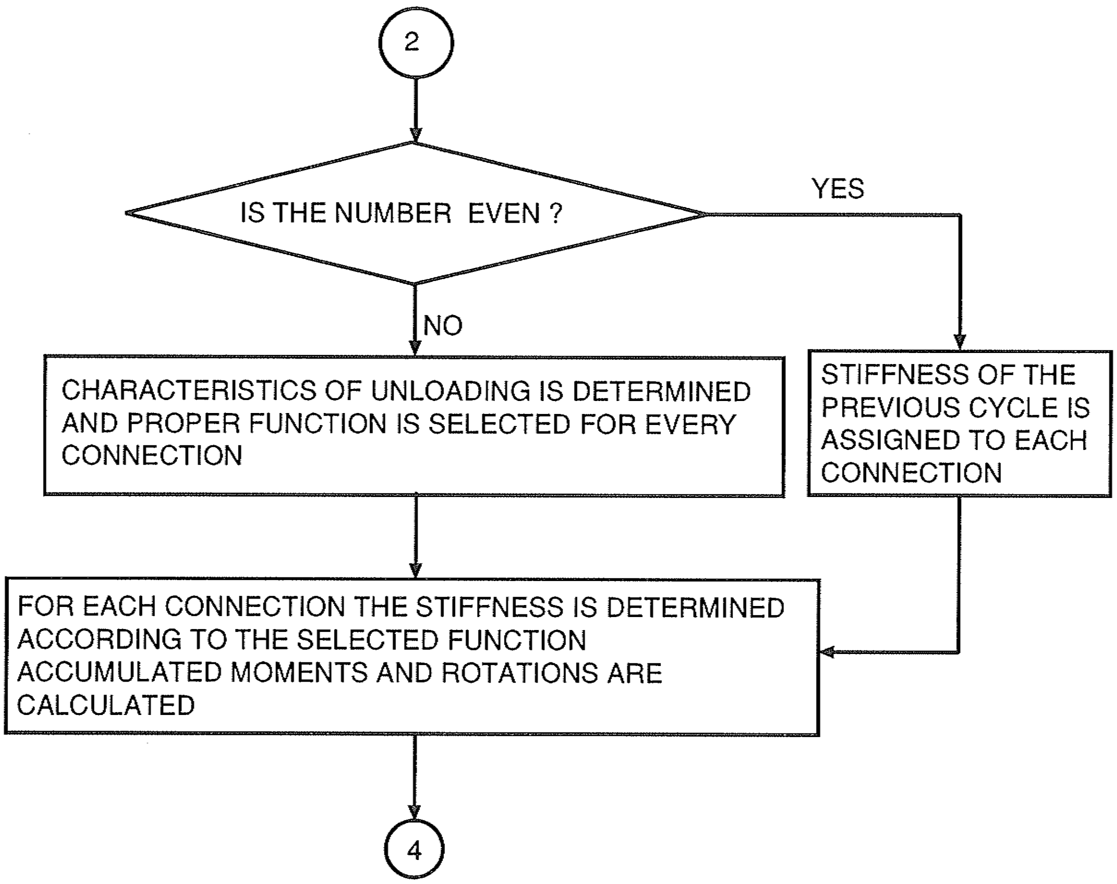


Fig. 5.1 Continued

CHAPTER 6

SENSITIVITY STUDIES

6.1 Introduction

In this chapter, 5 examples are presented to illustrate the operation of the analysis program and the effects of connection behaviour on several aspects of frame behaviour. Examples have been selected which involve simple structures and loadings, but which illustrate the significance of connection deformation and the validity and capabilities of the analysis program. While the program analyzes three-dimensional frames, for the sake of simplicity they are represented here as two-dimensional ones.

6.2 Validity of the Analysis Program

Example 1

To verify the accuracy of the analysis procedure implemented in the program FLEXMAIN, the pinned-base, unbraced plane frame shown in Fig. 6.1 was analyzed, under four different loading sequences, each culminating in a 42 kN/m uniformly distributed beam load, which was subsequently removed and the structure re-analyzed to evaluate residual effects. For the first loading sequence, the 42 kN/m load was applied in a single step. Next, it was applied in 3 steps of 14 kN/m, then in 6 steps of 7 kN/m and finally in 12 steps of 3.5 kN/m.

The moment resistance of the beam-to-column connections was assumed to be approximately 55 percent of the beam M_p value and their initial stiffness corresponded to the EUROCODE 3 rigid semi-rigid boundary. Their M - ϕ behaviour was represented in terms of the Richard-Abbott function, with the following parameters.

$$M_o=0.544M_p=173.25kN/m$$

$$S_i=25\frac{EI_b}{L_b}=93000KN.m/rad$$

$$\phi_o=\frac{M_0}{S_i}=\frac{173.25}{93000}=0.00186$$

$$s_p=0$$

$$n=7$$

The M vs ϕ plots for the beam-to-column connections for the four loading sequences are shown in Fig. 6.2. As expected, the connection M and ϕ values and the frame displacements and internal forces under full load and after load removal were found to be independent of the number of load steps. The Richard-Abbott function representation of the M-ϕ behaviour, which is not plotted, is the envelope of M-ϕ points on the ascending branch of the curves. The residual connection rotational deformations of

0.00369 radians following load removal were consistent with the computed residual moment of 11.2 kN m in the beam.

This example confirmed the ability of the analysis program to trace the nonlinear behaviour of the connections and compute the same structural forces and deflections regardless of the proportional loading history.

Example 2

The frame in Fig. 6.1 was next subjected to the following sequence of loadings :

1. factored dead plus live gravity load of 28 kN/m on the beam ;
2. wind load of 46 kN at roof level, applied to the right ;
3. wind load removed ;
4. wind load applied to the left ;
5. wind load removed ;
6. gravity load removed.

The beam bending moment diagrams and the horizontal drift subsequent to each loading application are shown in Fig. 6.3. For comparison purposes the results for a similar rigidly connected frame are shown in square brackets in the figure.

At all loading stages with the gravity load in place, the beam static moment, $wL^2/8$ = 350 kN m, was maintained, as it must be. For the flexibly connected frame, during the cycle of wind load reversal the beam end moments reduced from 89.5 to 80.8 kN m, indicating the shakedown effect. Following the removal of all load, the constant beam bending moment of 11.8 kN m reflected the inelastic deformation of the connections. The residual drift of 7 mm following removal of the wind load resulted in reduced drift (45 mm) under the reversed wind load.

Comparison of the moments for the flexibly-connected and the rigidly-connected frames shows that the beam mid-span moment for the former were 6 percent larger than for the rigid frame, the connection moments being correspondingly smaller. The initial wind-load drift of the flexibly connected frame was 18 percent larger than that for the rigid frame.

The M and ϕ values for the left and right flexible connections, designated L and R, at all loading stages (points 1 through 6) are shown in Fig. 6.4. Under gravity load, the moments at both connections were 89.5 kN m (points L1 and R1). With the application of wind load to the right, the moment at the left connection decreased, while the right connection experienced increased moment (L2 and R2). Upon removal of the wind load the connection moments were again equal (L3 and R3). Then with the wind applied to the left, the connection moment magnitudes, indicated by L4 and R4, approximately reversed from those at point 2. With the removal of the wind load, the connection moments again became equal (L5 and R5), and when the gravity load was removed equal residual connection moments of 11.8 kN m (L6 and R6) remained.

6.3 Effect of Connection Behaviour on Lateral Force Distribution

Example 3

A three-bay, three-story braced frame similar to one considered by Zaremba (1986) is shown in Fig. 6.5. In order to examine the effects of connection behaviour on the internal force distribution, the frame was analyzed under the following four factored load cases

1. dead load plus gravity live load, shown in Fig. 6.5;
2. dead load plus a checkerboard pattern of gravity live load, shown in Fig. 6.6;
3. simultaneous gravity load plus wind load for both full and checkerboard gravity loading; and
4. sequential gravity load and wind load.

The Richard-Abbott function was used to model the connection M-ϕ behaviour of the partial-strength beam-to-column connections, with moment resistance of 60 percent of the beam M_p and the following EUROCODE 3 stiffness characteristics:

1. perfectly rigid;
2. on the boundary between the rigid and semi-rigid classification, with the following of Richard-Abbott parameters

   roof beams:

   M_o=719 kN m, ϕ_o=0.006626 rad., n=3.26, S_p=0 kNm/rad.

floor beams:

M_o=1001 kN m, ϕ_o=0.00647 rad., n=3.28, S_p=0 kNm/rad;

3. at the middle of the semi-rigid classification, with parameters as above except with M_o values reduced by 50 percent; and
4. on the semi-rigid nominally-pinned boundary, with the following parameters.

   roof beams:

   M_o=3.92 kN m, ϕ_o=0.000556 rad., n=1, S_p=6550 kN m/rad.

   floor beams:

   M_o=7.072 kN m, ϕ_o=0.000735 rad., n=1.7, S_p=9555 kN m/rad.

The beam and column end moments for the full-loading and checkerboard-loading cases and for the three sets of connection behaviour are summarized in Fig. 6.7 and Fig. 6.8 respectively. It can be seen that the connection M - ϕ characteristics had a very significant effect on the member end moments. For the full gravity load case, the beam end moments for the flexibly connected frames averaged 79, 62, and 20 percent of those for the rigid frame, the larger values corresponding to the stiffer connections. The corresponding values for the exterior columns were 92, 80, and 28 percent. Under dead load and a checkerboard pattern of live load, similar moment differences occurred. These results suggest that it may be grossly unconservative to under-estimate connection stiffness and thus the moments transmitted to the columns in a braced frame.

6.4 Effect of Connections on Frame Lateral Drift

Example 4

The 11 story unbraced frame shown in Fig. 6.9, which is similar to one considered by Ang and Morris (1984), was analyzed under proportional and then sequential gravity and wind load. Separate analyses were performed assuming rigid beam-to-column connections, then full strength connections whose M-ϕ behaviour was on the boundary between the EUROCODE3 rigid and semi-rigid classifications. The Richard-Abbott function was used to model the connection M-ϕ behaviour of the full-strength connections, and the Richard-Abbott parameters used were

- boundary between the rigid and semi-rigid classifications:

  M_o=933 kN m, ϕ_o=0.002123 rad., n=2.16, S_p=0 kN m/rad; and

- middle of the semi-rigid classification:

  M_o=466.5 kN m, ϕ_o=0.002123, n=2.16, S_p=o kN m/rad.

For each structure and loading, two analyses were performed, the first with P-Δ effects ignored, the second with them accounted for.

For the various connection types, loading sequences and analysis assumptions the frame drift at roof level, and the normalized drift, based upon a value of 1.00 for the rigidly connected frame with no P-Δ effects, are summarized in Table 6.1 and plotted in Fig.

6.9. The roof-level drifts for the frames with rigid - semi-rigid and with semi-rigid connections were 25 and 117 percent larger than that for the rigid frame. By comparison, the drift amplification due to the P-Δ effect was 11, 16, and 35 percent for the three frames. Because full strength connections were used, with the same beam size at all floors, the connection deformations were relatively small. Had partial strength connections been used, their effect on drift amplification would have been larger. However, even with the full strength connections the drift amplification due to connection deformation was significantly larger than that due to the P-Δ effect.

For the flexibly connected frames the drift under sequential loading was 5 to 15 percent smaller than that under proportional loading. A similar phenomenon has been observed by Deierlein (1991).

The frame member end moments under non-proportional loading are shown in Fig. 6.10. The moments of the frame with rigid connections are shown in the first line, while normalized moment (based upon those for the rigid frame = 1.0) for connection M - ϕ curves on the rigid-semi-rigid boundary and at the middle of the semi-rigid range are in the second and third lines, respectively.

Beam-to-Column Connection Type	Analysis Condition	Drift at Roof Level (mm)	Normalized Drift
Rigid	No P-Δ effect	125	1.00
	Proportional, P-Δ	140	1.12
	Sequential, P-Δ	140	1.12
Rigid - Semi-Rigid Boundary	No P-Δ effect	157	1.26
	Proportional, P-Δ	182	1.46
	Sequential, P-Δ	173	1.38
Semi-rigid	No P-Δ effect	272	2.18
	Proportional, P-Δ	367	2.94
	Sequential, P-Δ	313	2.50

Table 6.1 Effect of Connection Behaviour on Lateral Drift, Example 4

Comparison of the member end moments for the various connection types and loading arrangements reveals the following.

1. The beam end moments at the leeward ends of the beams in the rigidly connected frame were approximately 5 percent larger than those in the frame with connections on the rigid-semi-rigid boundary, and 10 to 30 percent larger than those for the frame with semi-rigid connections. The beam end moments at the windward ends of the beams were relatively small and erratic.

2. The column end moments at the leeward side of the rigidly connected frame were approximately 10 percent larger than those in the frame with connections on the rigid-semi-rigid boundary and 10 to 30 percent larger than those for the frame with semi-rigid connections. The column end moments at the windward side were again relatively small.

3. It was observed that for the flexibly connected frames, the member end moments under non - proportional loading were approximately 10 percent smaller than those under similar proportional loading.

6.5 Effect of Connection Behaviour on Column End Restraint

Example 5

To examine the effect of connection behaviour on the end restraint provided to the columns of a frame, and thus their effective length factors, the braced frame shown in Fig. 6.11 was analyzed first assuming connections with M - ϕ curves on the rigid - semi-rigid boundary, then with ones at the middle of the semi-rigid classification and finally with ones on the semi-rigid - nominally-pinned boundary. The corresponding Richard-Abbott parameters were similar to those used in Example 3.

Column A - B in Fig. 6.11 was considered. The rotational deformations for the beam-to-column connections at its upper and lower ends were determined by analyzing the frame. The connection tangent stiffnesses, C, were then computed using the appropriate Richard-Abbott functions. The effective connection stiffnesses, C^*, at the top and bottom of column AB were computed using Eq. (2.8). In the computation, the current tangent stiffness was used for the connection on one side of the column, while the unloading stiffness (equal to the initial stiffness) was used for the connection on the otherside. Finally, the column end restraint factors G_A and G_B were evaluated using Eq. (2.11), and the column effective length factors, K were obtained from the Julian and Lawrence alignment charts. The analysis results are presented in Table 6.2 for the three sets of connection behaviour considered.

It can be seen from the Table that the end restraint factors increased from 0.44 for the stiffest connection to 1.75 for the most flexible one. Correspondingly, the column effective length factor, K, increased from 0.67 to 0.84. The latter 25 percent increase would correspond approximately to a 55 percent reduction in the strength of the column. Obviously, connection deformation may have a significant effect upon column end restraint and thus column strength.

In order to examine the effect of load level on column end restraint in a flexibly connected frame, the frame in Fig. 6.11, with connections at the middle of semi-rigid classification, was analyzed under full gravity load and then under 50 percent of that load. The analysis results are presented in Table 6.3. Naturally, because the connection tangent stiffness become smaller as the load increased, the column effective length factor increased. It can be seen, however, that a load increase of 50 percent produced a corresponding increase in effective length factor of only 3 percent.

		$\phi x10^{-3}$	$Cx10^{7}$	$C^{*}x10^{6}$	G_A,G_B	K
Boundary Rigid-Semi - Rigid	18	3.236	15.0	30.72	0.44	.67
	19	2.999	15.1	30.72		
	28	3.281	15.0	30.72		
	29	3.023	15.1	30.72		
Semi-rigid	18	5.582	6.71	24.38	0.55	.70
	19	5.276	6.84	24.58		
	28	5.651	6.68	24.38		
	29	5.325	6.82	24.58		
Boundary Semi-rigid - pinned	18	11.688	0.955	7.68	1.75	.84
	19	11.704	0.955	7.68		
	28	11.736	0.955	7.68		
	29	11.708	0.955	7.68		

Table 6.2 Effect of Connection on End Restraint, Example 5

		$\phi X10^{-3}$	$CX10^{7}$	$C^{*}X10^{6}$	G_A,G_B	K
Full Load	18	5.582	6.71	24.38	0.55	0.70
	19	5.276	6.84	24.58		
	28	5.651	6.68	24.38		
	29	5.325	6.82	24.58		
Half Load	18	2.585	7.63	25.54	0.52	0.68
	19	2.461	7.65			
	28	2.615	7.63			
	29	2.474	7.65			

Table 6.3 Effect of Load Level on End Restraint, Example 5

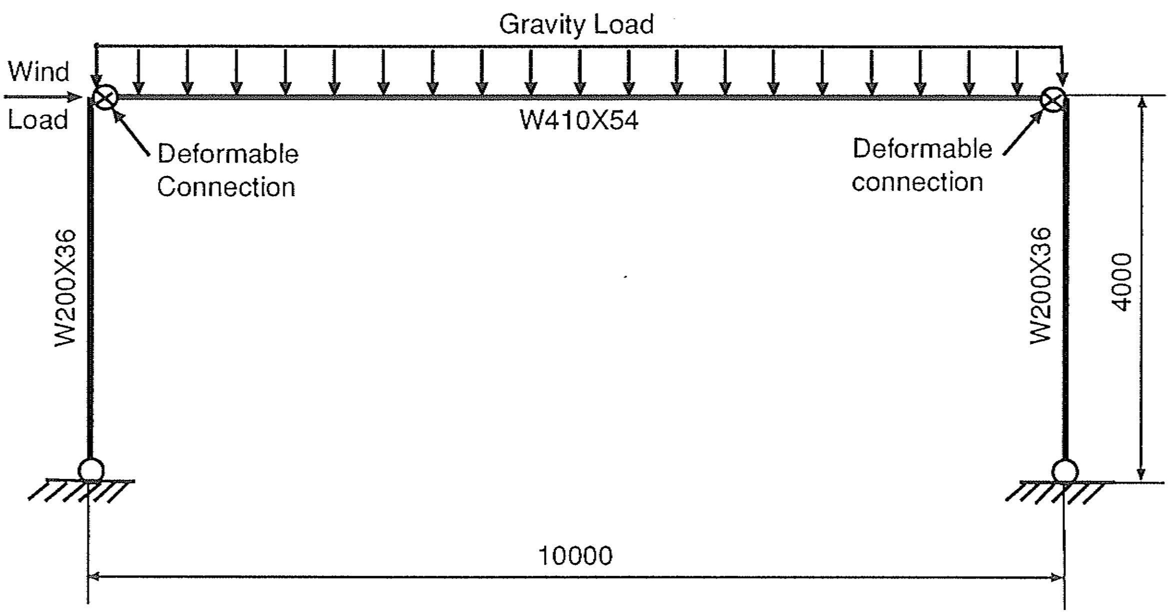


Fig. 6.1 Flexibly Connected Frame , Examples 1 and 2

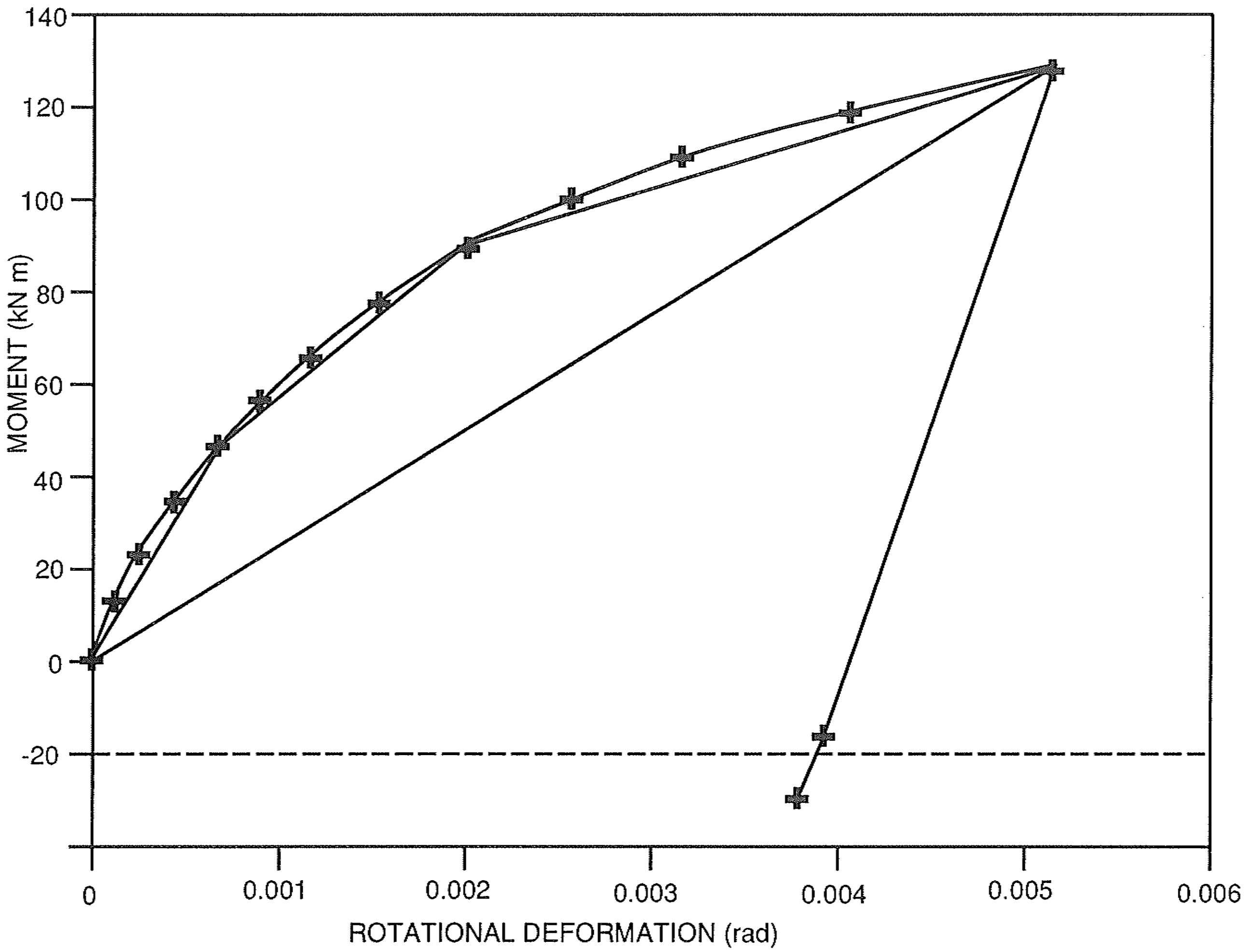


Fig. 6.2 Connection Moments and Rotatio, Example 1

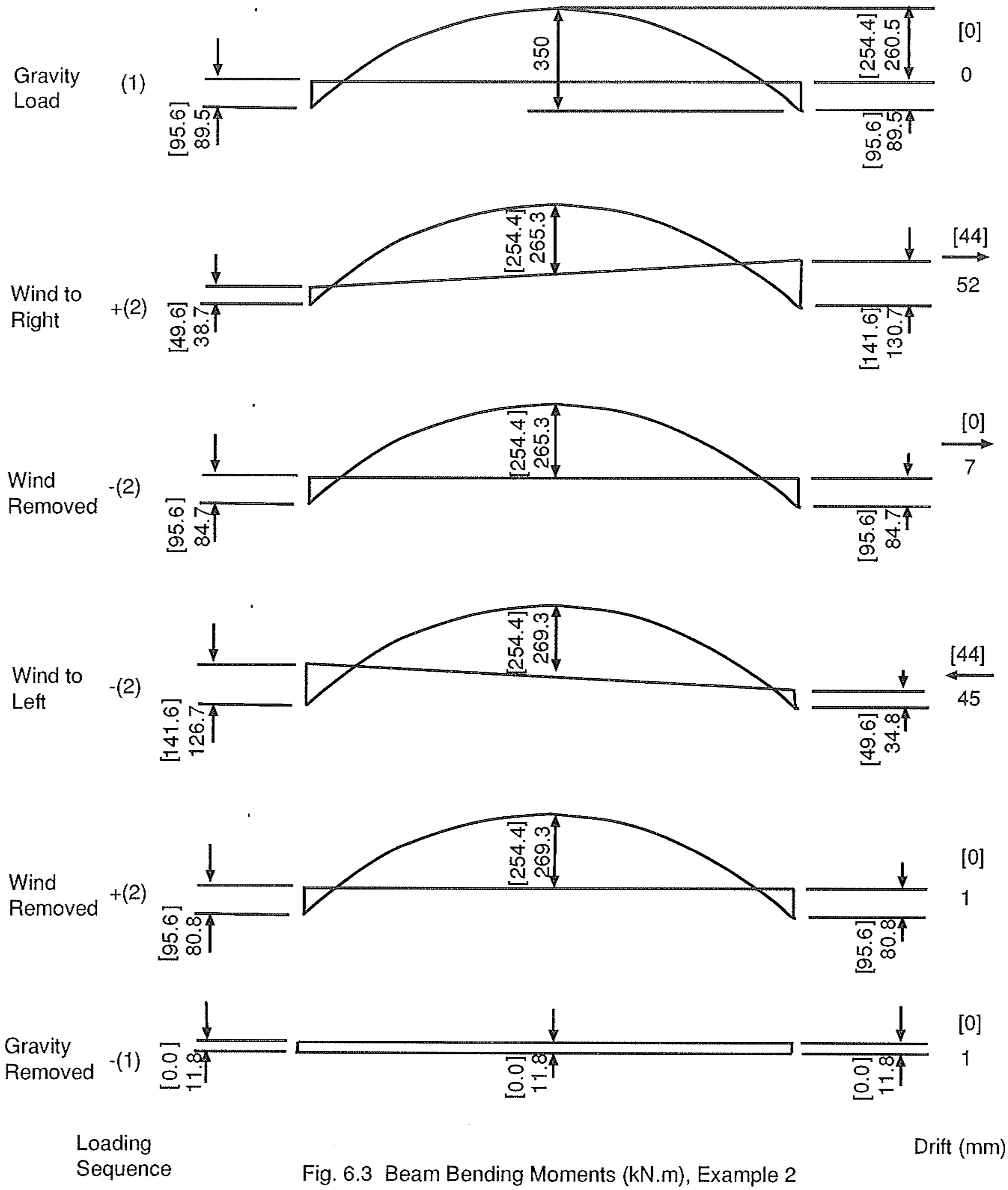


Fig. 6.3 Beam Bending Moments (kN.m), Example 2

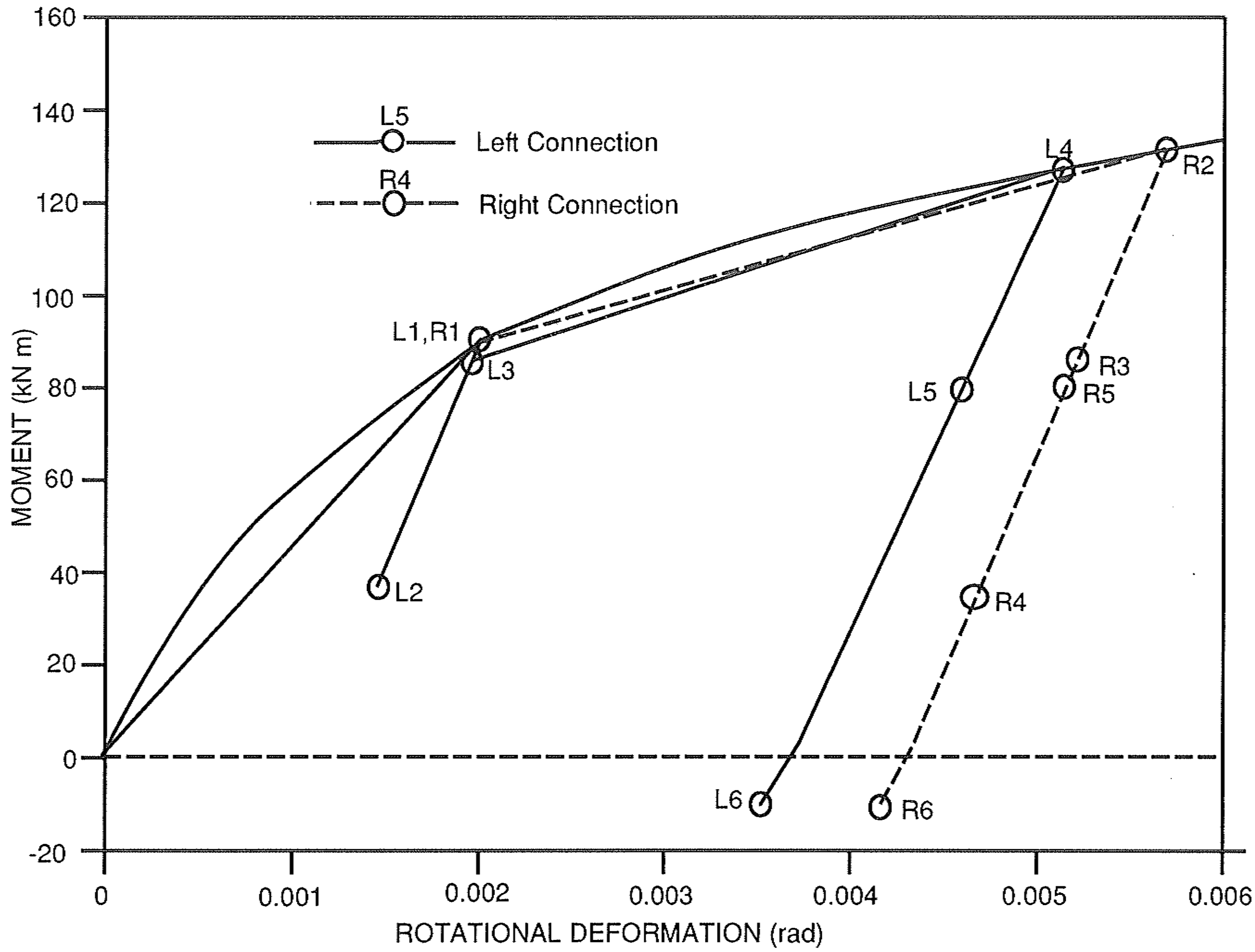


Fig. 6.4 Connection Moments and Rotations, Example 2

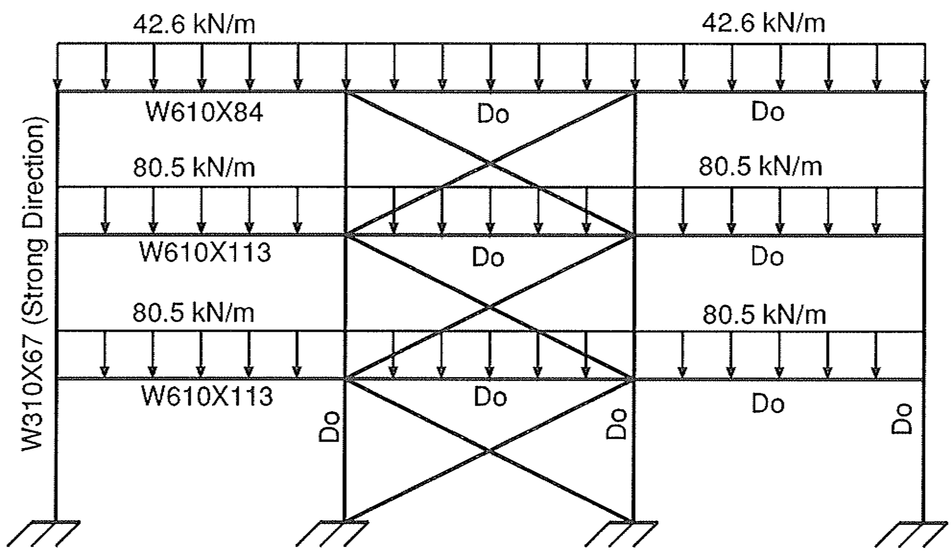


Fig. 6.5 Braced Frame, Example 3

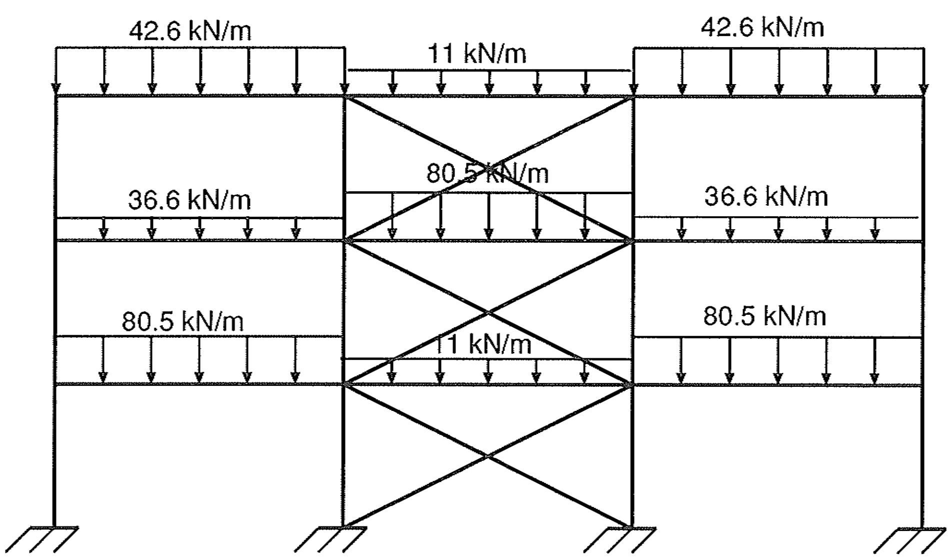
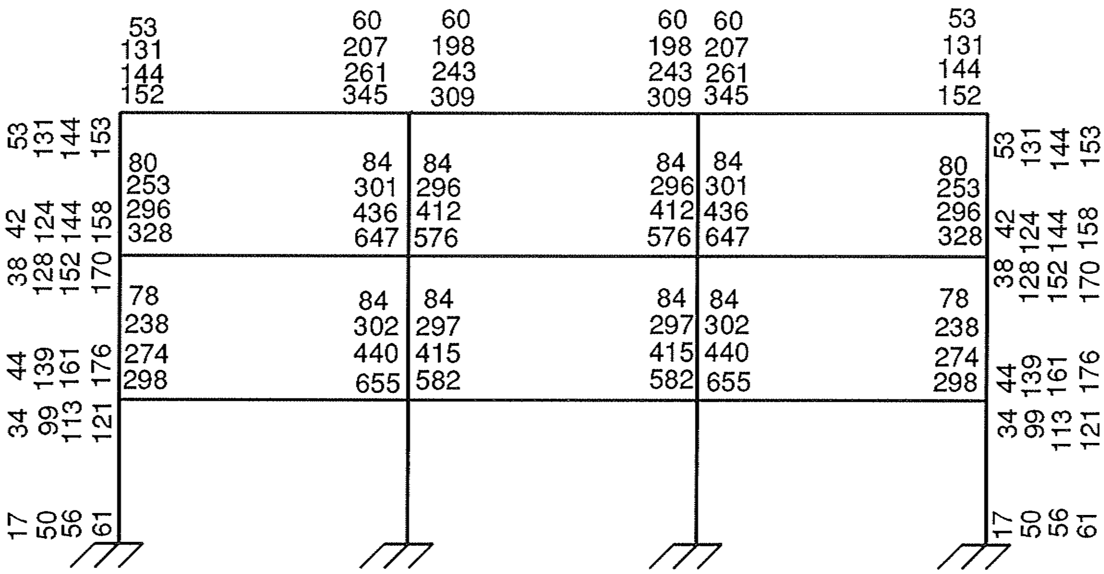


Fig. 6.6 Braced Frame and Checker Board Load, Example 3



Connections - semi-rigid - nominally pinned boundary
- semi-rigid
- rigid - semi-rigid boundary
- rigid

Fig. 6.7 Member End Moments Under Gravity Load, Example 3

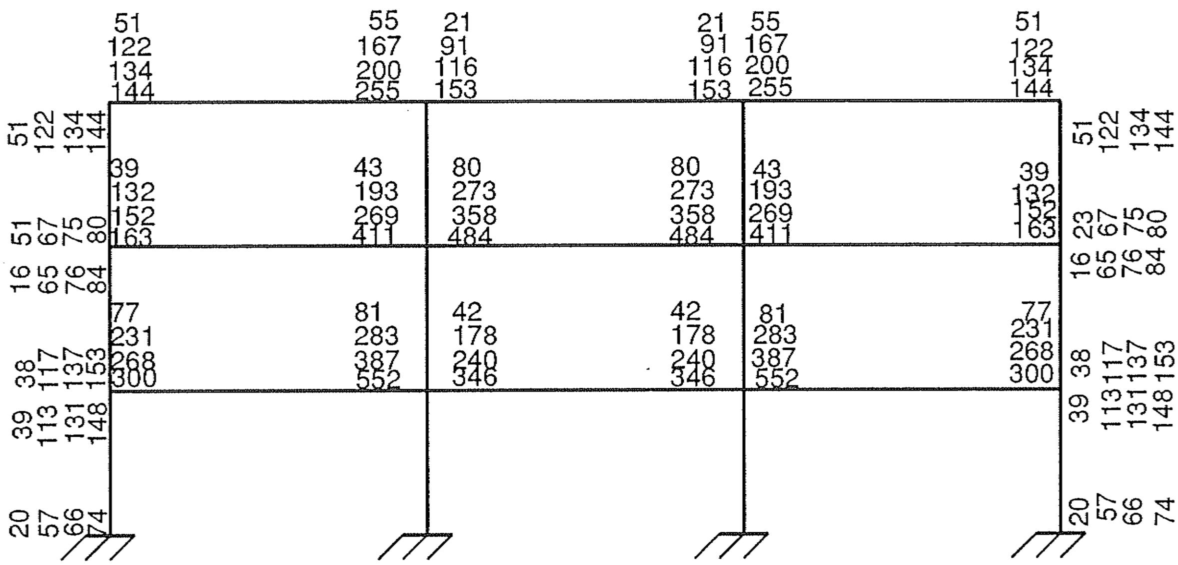


Fig. 6.8 Member End Moments Under Checker Board Gravity Load

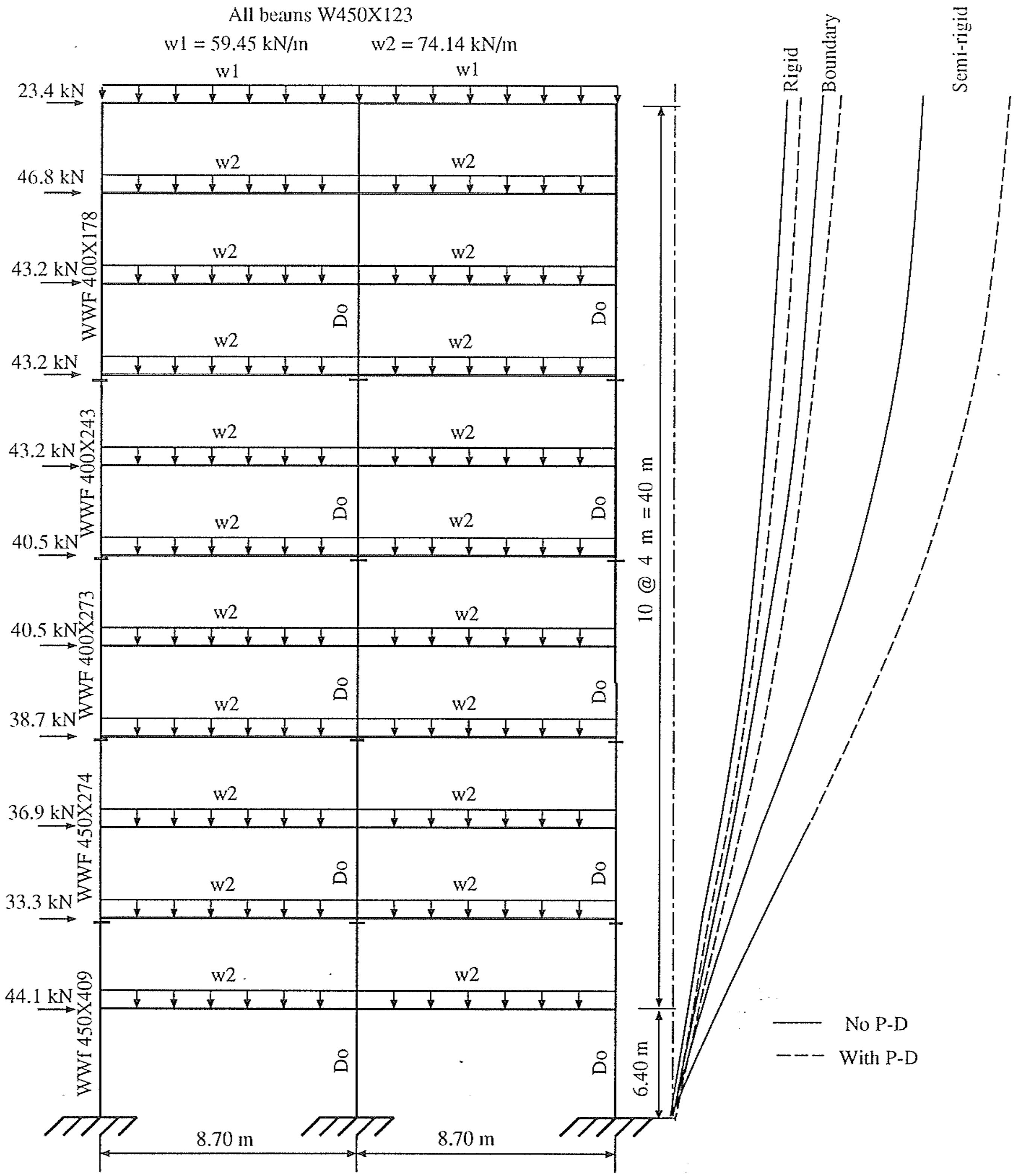


Fig. 6.9 11 Story Unbraced Frame, Example 4

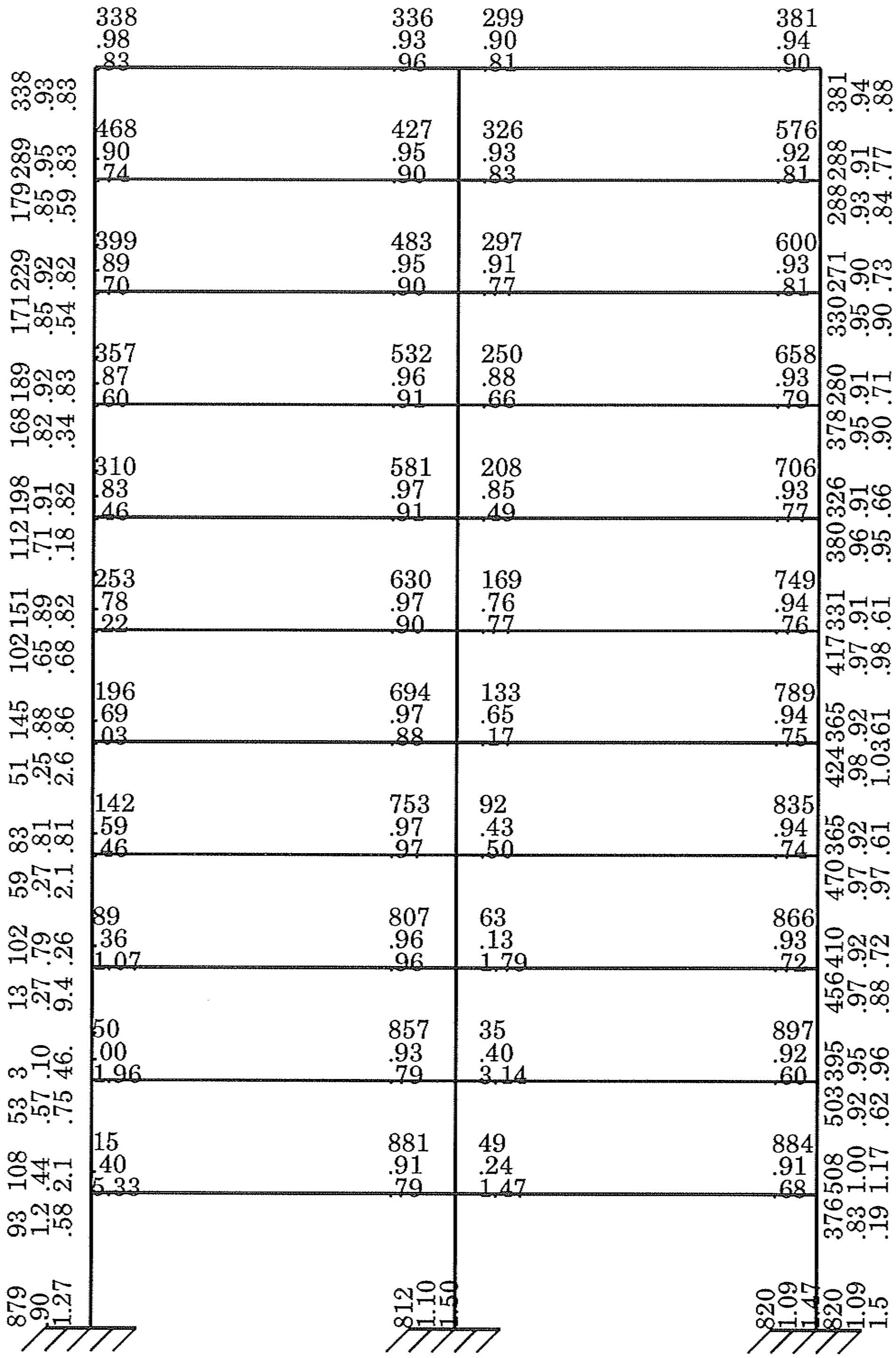
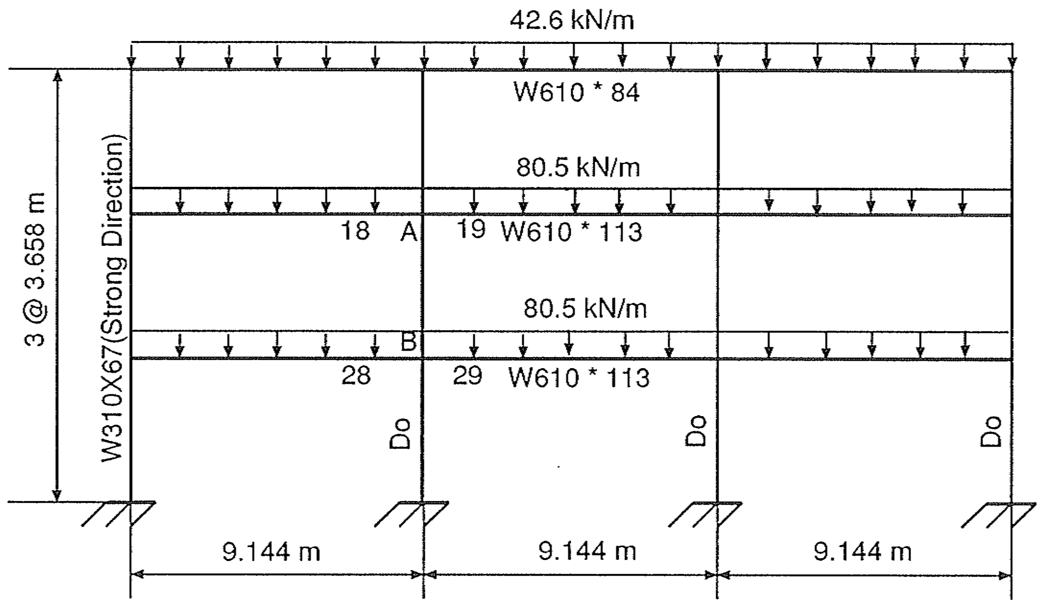


Fig. 6.10 Moment Distribution for Non-proportional Loading System, Example 4



Connections - semi-rigid - nominally pinned boundary
- semi-rigid
- rigid - semi-rigid boundary

Fig. 6.11 Braced Frame, Example 5

CHAPTER 7

CONCLUSION

7.1 Conclusions

In this study, the moment versus rotational behaviour of steel beam-to-column connections under cyclic loading has been modeled using five functions, all derived from the Richard-Abbott function. Those functions have been incorporated into a FORTRAN computer program which performs a nonlinear structural analysis of rectangular, three dimensional steel frames loaded by sequential, non-proportional loading systems. The program accounts for the nonlinear behaviour of the beam-to-column connections by performing repeated cycles of linear analysis and connection stiffness modification to arrive at linear connection stiffnesses corresponding to points on their nonlinear M - ϕ curves. The final linear analysis thus yields the correct displacements and internal forces for the nonlinear structure. Sequential, non-proportional loading systems are accommodated by incorporating the iterative procedure into an incremental one in which the appropriate loading or unloading connection M - ϕ functions are selected for the current loading increment. The effects of axial force on the bending stiffness of the columns (the P - δ effects) have been accounted for and procedures have been incorporated to account for the translations of the loads as the structure displaces (the P - Δ effects).

A description has been presented of the connection classification system which has been incorporated into the EUROCODE3 steel design specification as a preliminary design tool. That system classifies beam-to-column connections in terms of their strength and stiffness relative to those of the beam that is being connected. The boundaries between the connections are piece-wise linear. It has been demonstrated in this study that a convenient and accurate alternative is to replace the piece-wise linear boundaries by Richard-Abbott functions.

The nonlinear analysis computer program has been used to conduct sensitivity studies to determine the effects of connection behaviour on the behaviour of flexibly connected frames. Those studies showed that connection deformation may have a significant effect on the internal force distribution in a frame. In one example the beam end moments in a braced frame with semi - rigid connections were 38 percent smaller than those in a similar rigid frame. Connection deformation may also significantly amplify the lateral drift of a multi-storey frame. For an 11 storey unbraced frame, the drift amplification when substituting semi - rigid connections for rigid ones was 117 percent of the amplification due to the P - Δ effect. Finally, it has been shown that the flexibility of the beam-to-column connections in a frame may substantially affect the column effective length factors. For a second storey column in a 3 storey frame, replacing rigid beam-to-column connections by ones with M - ϕ curves on the boundary between semi-rigid - nominally - pinned resulted in a 25 percent increase in an effective length factor.

Examples have demonstrated that for flexibly connected frames, the lateral drift may be at least 15 percent larger under proportional loading than under non-proportional loading.

7.2 Recommendations for Further Study

The computer program developed in this study generally requires that the M - ϕ behaviour of the beam-to-column connections be input in terms of Richard-Abbott function parameters. Given the current state of the art, the only reliable source of M - ϕ data for the variety of connection types and geometries currently in use is physical tests of connection specimens. Recently, accurate Richard-Abbott functions have been fitted to almost all of the experimental M - ϕ data available in the world. The Richard-Abbot parameters and the corresponding connection geometric parameters for those data should be incorporated into the analysis program. It will then be able to simply read the connection types and geometries and perform the analysis.

In a number of countries there have been attempts to effect fabrication savings by standardizing connection geometries. A side benefit to such standardization would be that M - ϕ data would be required for fewer connection geometries. A survey of standardization attempts in different countries should be compared with the set of currently-available experimental M - ϕ data in order to determine what additional connection testing needs to be done.

Very few sets of M - ϕ data are available for connections between steel columns and steel-concrete composite beams. Attempts should be made to standardize connections for composite construction and to perform tests on the standard composite connections, so that the program can be extended to include composite frames.

Further sensitivity studies should be conducted in order to determine how sensitive frames of various geometries are to variations in connection behaviour. Such studies should reveal whether connection behaviour warrants being modeled extremely accurately or only slightly more accurately than it has been in the past.

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APPENDIX A

PROGRAM LISTING

```
C
C$NOARRAYCHECK
C$NOEX
C$NOWA
C ************************************************************
C *
C *   FLEXFRM: UNLOADING ANALYSIS OF THREE-DIMENSIONAL
C *            RECTANGULAR STEEL FRAMES WITH
C *            NONLINEAR BEAM-COLUMN CONNECTIONS
c *
C *    PROGRAMMED BY GLEN MORRIS AND JITIAN HUANG
C *
C ************************************************************
C *
C *     PROGRAM LIMITATIONS:
C *
C *        (1) MAXIMUM OF 20 BEAM-COLUMN CONN.
C *            TYPES PERMITTED
C *        (2) SIZE OF STRUCTURE LIMITED BY
C *            SIZE OF STORAGE POOL A
C *
C ************************************************************
      IMPLICIT REAL*8 (A-H,O-Z)
      DIMENSION A(27000),USER(40),DSCRP(40)
      COMMON /IPT/ NST, NBAY1, NBAY3
      COMMON /GEN/ NTJ, NFB, NFC, NBAND, NEQ, NXM
      COMMON /TBRACE/ NFT, NBF
      COMMON /MAT/ EM, GM
      COMMON /ALOAD/ NLJ, NLMC, NLMU, NLT1, NLT2, NLT3
      COMMON /SECT/ BP(4,20), CP(4,20)
      COMMON /SECT2/ TP(20)
      COMMON /TEMPB1/NPRT,NFIL,IPDELT
      COMMON /COM1/ NLOAD
      COMMON /COMR/ R,A
      LOGICAL SSI,PPY
      CHARACTER USER,DSCRP
      NLOADTM=1
7     INT=0
      RL=0.
      CALL CLSC
      WRITE(*,'(//10X,''PRINTED OUTPUT ? YES or NO:'')')
      CALL YN(NPRT)
C
C     UNIT 3 = PROGRAM OUTPUT
C
      IF(NPRT.EQ.1)THEN
        OPEN(3,FILE='PRN')
      ELSE
        OPEN(3,FILE='RESULTS1.OUT')
      END IF
C
C     NEW SCREEN, PRINT PROGRAM TITLE AND DESCRIPTION
```

```
C
      CALL CLSC
      WRITE(*,'(2A,I2.2,A,I2.2,A\)')CHAR(27),'[',5,';',30,'H'
      CALL VIDON
      WRITE(*,'(''FLEXFRM'')')
      CALL VIDOF
      WRITE(*,1000)
1000 FORMAT(/9X,'         PROGRAM TO ANALYZE RECTANGULAR          '/
    2      9X,'      THREE-DIMENSIONAL STEEL FRAMES          '/
    3      9X,'      WITH FLEXIBLE BEAM-TO-COLUMN            '/
    4      9X,'      CONNECTIONS                             ')
C
C   ASK WHETHER TO USE OLD INPUT FILE OR CREATE NEW ONE
C
      WRITE(*,'(////20X,''IS THIS A NEW FILE? YES or NO:'')')
      CALL YN(NFIL)
C
C   NEW SCREEN
      CALL CLSC
      IF(NFIL.EQ.1)GO TO 100
C
C   IF USING OLD INPUT FILE (UNIT 2), READ FROM IT:
C        USER IDENTIFICATION
C        PROJECT DESCRIPTION
C        NUMBER OF STOREYS, NST
C        NO. OF BAYS IN X-DIRECTION, NBAY1
C        NO. OF BAYS IN Z-DIRECTION, NBAY3
C        UNITS: IUNITS=1-SI
C        P-DELTA EFFECT INCLUDED? IPDELT=1-YES, 2-NO
C        MODULUS OF ELASTICITY, EM
C        SHEAR MODULUS, GM
C
      OPEN(2,FILE='STSPEC.IN')
      READ(2,1010)(USER(J),J=1,40)
      READ(2,1010)(DSCRP(J),J=1,40)
1010 FORMAT(40A1)
      READ(2,*)NST,NBAY1,NBAY3,IUNITS,IPDELT,EM,GM,NLJ,NLMU,NLMC,NTA,NFT
     *,NBF,NLOAD,NCOMB,NGEOM
      CLOSE(2)
      GOTO 131
C
C   IF CREATING NEW INPUT FILE, PROMPT FOR REQUIRED INFORMATION
C
100  WRITE(*,'(//10X,''USER NAME:'')')
      CALL BACK(1,35)
      CALL VIDON
      READ(5,1010)(USER(J),J=1,40)
      CALL VIDOF
120  CONTINUE
      WRITE(*,'(//10X,''PROJECT DESCRIPTION:'')')
      CALL BACK(1,35)
      CALL VIDON
```

```
      READ(5,1010)(DSCRP(J),J=1,40)
      CALL VIDOF
      WRITE(*,'(//10X,''NUMBER OF STOREYS:'')')
      CALL RD(NST,RL,55)
      WRITE(*,'(//10X,''NUMBER OF BAYS IN X DIRECTION:'')')
      CALL RD(NBAY1,RL,55)
      WRITE(*,'(//10X,''NUMBER OF BAYS IN Z DIRECTION:'')')
      CALL RD(NBAY3,RL,55)
      WRITE(*,'(//10X,''NUMBER OF BRACES:'')')
      CALL RD(NFT,RL,55)
      WRITE(*,'(//10X,''NUMBER OF BRACED FRAMES:'')')
      CALL RD(NBF,RL,55)
      WRITE(*,'(//10X,''NUMBER OF LOADS:'')')
      CALL RD(NLOAD,RL,55)
      WRITE(*,'(//10X,''NUMBER OF CONNECTION CONBINATIONS:'')')
      CALL RD(NCOMB,RL,55)
      WRITE(*,'(//10X,''FOR GEOMETRIC PARAMETERS INPUT 2:'')')
      CALL RD(NGEOM,RL,55)
C
C     NEW SCREEN
      CALL SC
C
C     INPUT MEASUREMENT UNITS - 1=SI
C
      IUNITS=1
      WRITE(*,'(//10X,''UNITS: 1= SI'')')
122   CALL RD(INT,RL,55)
      IF(INT.EQ.1.OR.INT.EQ.2)GO TO 124
      WRITE(*,'(10X,''MUST SELECT 1  PLEASE RE-ENTER'')')
      CALL BACK(2,55)
      GO TO 122
124   IUNITS=INT
      E=0.
      G=0.
C
C      SI UNITS
C
      WRITE(*,'(//10X,''MODULUS OF ELASTICITY (kN/mm^2):''/10X,''("ENT
     *ER" GIVES DEFAULT OF 200)'')')
      EM = 200.
      CALL RD(II,E,55)
      IF(E.NE.0.)EM=E
      WRITE(*,'(//10X,''SHEARING MODULUS (kN/mm^2):''/10X,''(99NTER" G
     *IVES DEFAULT OF 80)'')')
      GM=80.
      CALL RD(II,G,55)
      IF(G.NE.0.)GM=G
      WRITE(*,1020)
1020  FORMAT(//10X,'CONSTRUCTION TYPE (SELECT ONE):'/
     *15X,          '1 = PINNED CONNECTIONS'/
     *15X,          '2 = RIGID CONNECTIONS'/
     *15X,          '3 = CONSTANT-STIFFNESS CONNECTIONS'/
```

```
     *15X,           '4 = NONLINEAR CONNECTIONS')
125  CALL RD(NTA,RL,55)
     IF(NTA.GE.1.AND.NTA.LE.4)GO TO 130
     WRITE(*,1022)
1022 FORMAT(10X,'MUST SELECT 1,2, 3 OR 4.  RE-ENTER')
     CALL BACK(2,55)
     GO TO 125
C     NEW SCREEN
130  CALL SC
     WRITE(*,'(//10X,''P-DELTA EFFECT INCLUDED? YES or NO:'')')
     CALL YN(IY)
     IPDELT=1
     IF(IY.EQ.2)IPDELT=2
C    NEW SCREEN
     CALL SC
     WRITE(*,'(//10X,''NUMBER OF LOADED JOINTS:'')')
     CALL RD(NLJ,RNLJ,55)
     WRITE(*,'(//10X,''NUMBER OF UNIFORMLY LOADED BEAMS:'')')
     CALL RD(NLMU,RNLMU,55)
     WRITE(*,'(//10X,''NUMBER OF BEAMS WITH CONCENTRATED LOADS:'')')
     CALL RD(NLMC,RNLMC,55)
131  WRITE(*,'(////10X,''EDIT USER ID, PROJECT DESCRPITION,''/
    *10X,''STRUCTURAL PARAMETERS ? YES or NO:'')')
     CALL YN(IY)
     IF (IY.NE.1)GO TO 290
C    NEW SCREEN
     CALL CLSC
     WRITE(*,'(//10X,''USER IDENTIFICATION:     '',40A1)')(USER(J),J=1,
    *40)
     WRITE(*,1025)
1025 FORMAT(/10X,'ANY CHANGE ?  YES or NO:')
     CALL YN(IY)
     IF(IY.NE.1)GO TO 132
     WRITE(*,'(//10X,''INPUT NEW USER IDENTIFICATION: ''\)')
     CALL VIDON
     READ(*,1010)(USER(J),J=1,40)
     CALL VIDOF
132  WRITE(*,'(//10X,''PROJECT DESCRIPTION:     '',40A1)')(DSCRP(J),J=1
    *,40)
     WRITE(*,1025)
     CALL YN(IY)
     IF(IY.NE.1)GO TO 134
     WRITE(*,'(//10X,''INPUT NEW PROJECT DESCRIPTION: '',\)')
     CALL VIDON
     READ(*,1010)(DSCRP(J),J=1,40)
     CALL VIDOF
C    CLEAR SCREEN
134  CALL SC
     WRITE(*,1030)NST,NBAY1,NBAY3,NFT,NBF,NLOAD,NCOMB,NGEOM
1030 FORMAT (//10X,'NUMBER OF STOREYS . . . . . . . . . . . . ',I10/
    *       10X,'NUMBER OF BAYS IN X DIRECTION . . . . . . ',I10/
    *       10X,'NUMBER OF BAYS IN Z DIRECTION . . . . . . ',I10/
```

```
     *      10X,'NUMBER OF BRACES  . . . . . . . . . . . . ',I10/
     *      10X,'NUMBER OF BRACED FRAMES . . . . . . . . . ',I10/
     *      10X,'NUMBER OF LOADS . . . . . . . . . . . . . ',I10/
     *      10X,'NUMBER OF CONNECTION COMBINATION. . . . . ',I10/
     *      10X,'KIND OF CONNECTION PARAMETERS INPUT . . . ',I10/)
      WRITE(*,1032)
 1032 FORMAT(//10X,'TO MODIFY, INPUT NEW VALUE.  OTHERWISE "RETURN"')
      WRITE(*,'(/10X,''NUMBER OF STOREYS                    ''I5)')NST
      CALL ED(NST,55)
      WRITE(*,'(10X,''NUMBER OF BAYS IN X-DIRECTION        '',I5)')NBAY1
      CALL ED(NBAY1,55)
      WRITE(*,'(10X,''NUMBER OF BAYS IN Z-DIRECTION        '',I5)')NBAY3
      CALL ED(NBAY3,55)
      WRITE(*,'(10X,''NUMBER OF BRACES                     '',I5)')NFT
      CALL ED(NFT,55)
      WRITE(*,'(10X,''NUMBER OF BRACED FRAMES              '',I5)')NBF
      CALL ED(NBF,55)
      WRITE(*,'(10X,''NUMBER OF LOADS                      '',I5)')NLOAD
      CALL ED(NLOAD,55)
      WRITE(*,'(10X,''NUMBRER OF CONNECTION COMBINATION    '',I5)')NCOMB
      CALL ED(NCOMB,55)
      WRITE(*,'(10X,''KIND OF CONNECTION PARAMETERS INPUT '',I5)')NGEOM
      CALL ED(NGEOM,55)
C     NEW SCREEN
      CALL SC
 138  GO TO (150,160,170,180)NTA
 150  WRITE(*,1050)
 1050 FORMAT(//10X,'PINNED BEAM-TO-COLUMN CONNECTIONS ASSUMED')
      GO TO 190
 160  WRITE(*,1060)
 1060 FORMAT(//10X,'RIGID BEAM-TO-COLUMN CONNECTIONS ASSUMED')
      GO TO 190
 170  WRITE(*,1070)
 1070 FORMAT(//10X,'LINEAR-STIFFNESS BEAM-COLUMN CONNECTIONS ASSUMED')
      GO TO 190
 180  WRITE(*,1080)
 1080 FORMAT(//10X,'NONLINEAR BEAM-TO-COLUMN CONNECTIONS ASSUMED')
 190  WRITE(*,1025)
      CALL YN(IY)
      IF(IY.EQ.2)GO TO 192
      WRITE(*,'(//10X,''TO CHANGE, USE FOLLOWING CODE.'')')
      WRITE(*,1020)
 191  CALL RD(NTA,RNTA,55)
      IF(NTA.GE.1.AND.NTA.LE.4)GO TO 192
      WRITE(*,1022)
      CALL BACK(1,5)
      GO TO 191
 192  CALL SC
 196  GO TO(200,210)IPDELT
 200  WRITE(*,1090)
 1090 FORMAT(//10X,'P-DELTA EFFECTS INCLUDED IN ANALYSIS')
      GO TO 220
```

```
210  WRITE(*,1100)
1100 FORMAT(//10X,'P-DELTA EFFECTS NOT INCLUDED IN ANALYSIS')
220  WRITE(*,1025)
     CALL YN(IY)
     IF(IY.NE.1)GO TO 228
     IPDELT=3-IPDELT
     GO TO 196
C    NEW SCREEN
228  CALL SC
229  CONTINUE
230  WRITE(*,1110)
1110 FORMAT(//10X,'SI UNITS USED')
250  WRITE(*,1025)
     CALL YN(IY)
     IF(IY.NE.1)GO TO 255
     IUNITS=3-IUNITS
     GO TO 229
C    NEW SCREEN
255  CALL SC
260  EM=200.
     GM=80.
     WRITE(*,1130)EM,GM
1130 FORMAT(//10X,'MODULUS OF ELASTICITY (kN/mm^2) . . . . .',F10.1//
    *10X,'SHEAR MODULUS (kN/mm^2) . . . . . . . . .',F10.1)
     WRITE(*,1032)
     WRITE(*,1033)EM
1033 FORMAT(//10X,'MODULUS OF ELASTICITY          ',F10.1)
     CALL EDR(EM,55)
     WRITE(*,1034)GM
1034 FORMAT(/10X,'SHEAR MODULUS                  ',F10.1)
     CALL EDR(GM,55)
280  CALL SC
     WRITE(*,1150)NLJ,NLMU,NLMC
1150 FORMAT(//10X,'NUMBER OF LOADED JOINTS . . . . . . . . . ',I10
    *      /10X,'NUMBER OF UNIFORMLY LOADED BEAMS. . . . . ',I10
    *      /10X,'NUMBER OF BEAMS WITH CONCENTRATED LOADS . ',I10/)
     WRITE(*,1032)
     WRITE(*,'(//10X,''NUMBER OF LOADED JOINTS:'')')
     CALL RD(NLJ,RNLJ,55)
     WRITE(*,'(//10X,''NUMBER OF UNIFORMLY LOADED BEAMS:'')')
     CALL RD(NLMU,RNLMU,55)
     WRITE(*,'(//10X,''NUMBER OF BEAMS WITH CONCENTRATED LOADS:'')')
     CALL RD(NLMC,RNLMC,55)
C    WRITE(*,'(/10X,''NUMBER OF LOADED JOINTS              '',I5)')NLJ
C    CALL ED(NLJ,55)
C    WRITE(*,'(10X,''NUMBER OF UNIFORMLY LOADED BEAMS      '',I5)')NLMU
C    CALL ED(NLMU,55)
C    WRITE(*,'(10X,''NUMBER OF BEAMS WITH CONCENT. LOADS   '',I5)')NLMC
C    CALL ED(NLMC,55)
C
C    STORE INPUT DATA FOR FUTURE USE AS AN INPUT FILE
C
```

```
290  OPEN(2,FILE='STSPEC.IN')
     WRITE(2,1010)(USER(J),J=1,40)
     WRITE(2,1010)(DSCRP(J),J=1,40)
     WRITE(2,*)NST,NBAY1,NBAY3,IUNITS,IPDELT,EM,GM
     WRITE(2,*)NLJ,NLMU,NLMC,NTA,NFT,NBF,NLOAD,NCOMB,NGEOM
     CLOSE(2)
     JPD=IPDELT
C
C    WRITE INPUT DATA TO OUTPUT FILE
C
     WRITE(3,'(///30X,''FLEXFRM'')')
     WRITE(3,1000)
     WRITE(3,1170) (USER(J),J=1,40)
     WRITE(3,1180) (DSCRP(J),J=1,40)
1170 FORMAT(/,10X,'USER IDENTIFICATION....... ',40A1)
1180 FORMAT(/,10X,'PROJECT DISCRIPTION....... ',40A1)
     WRITE(3,'(/10X)')
     WRITE(3,1030)NST,NBAY1,NBAY3,NFT,NBF,NLOAD,NCOMB,NGEOM
     GO TO (300,310,320,330)NTA
300  WRITE(3,1050)
     GO TO 340
310  WRITE(3,1060)
     GO TO 340
320  WRITE(3,1070)
     GO TO 340
330  WRITE(3,1080)
340  GO TO (350,360)IPDELT
350  WRITE(3,1090)
     GO TO 370
360  WRITE(3,1100)
370  CONTINUE
380  WRITE(3,1110)
     WRITE(3,'(/15X,''STRUCTURE DIMENSIONS . . . . . . . . m''/15X,
    *          ''CROSS-SECTIONAL DIMENSIONS . . . . . mm''/15X,
    *          ''LOADS AND FORCES . . . . . . . . . . kN''/15X,
    *          ''ELASTIC MODULI . . . . . . . . . . . kN/mm^2''/,1
    *5X,''DISPLACEMENTS  . . . . . . . . . . . mm, radians''/)')
     WRITE(3,1130)EM,GM
400  CONTINUE
     NITER=1
C
C      TO AVOID ZERO SUBSCRIPT IN LOAD VECTOR
C
     NLT1=NLJ
     NLT2=NLMC
     NLT3=NLMU
     IF(NLT1 .EQ. 0) NLT1=1
     IF(NLT2 .EQ. 0) NLT2=1
     IF(NLT3.EQ.0)NLT3=1
     PPY=.FALSE.
     IF(IPDELT.EQ.1) PPY=.TRUE.
C
```

```
C     NFRJ = NO. OF FREE NODES, INCLUDING MASTER NODE
C     NSJ = NO. OF SUPPORT JOINTS
C     NTJ = TOTAL NO. OF JOINTS
C     NFB = TOTAL NO. OF BEAMS
C     NFC = TOTAL NO. OF COLUMNS
C     NBAND = BAND WIDTH
C     NEQ = NO OF EQUATIONS
C
      NFRJ=((NBAY1 + 1)*(NBAY3 + 1) + 1)*NST
      NSJ=(NBAY1 + 1)*(NBAY3 + 1) + 1
      NFB=(NBAY3*(NBAY1 + 1) + NBAY1*(NBAY3 + 1))*NST
      NFC=(NBAY3 + 1)*(NBAY1 + 1)*NST
      NTJ=NFRJ + NSJ
      NBAND=(NTJ/(NST + 1))*2*3
      NEQ=NTJ*3
      NXM=NEQ*NBAND
C
C     SET UP STORAGE MAP
C     N1  CN(3,NTJ)       R*8  JOINT COORIDINATES
C     N2  INB(2,NFB)      I*4  BEAM INCIDENCE TABLE
C     N3  SPAN1(NBAY1)    R*8  LENGTH/BAY IN X1 DIRECTION
C     N4  SPAN3(NBAY3)    R*8  LENGTH/BAY IN X3 DIRECTION
C     N5  DHE(NST)        R*8  HEIGHT PER STORY
C     N6  INC(2,NFC)      I*4  COLUMN INCIDENCE TABLE
C     N7  IBP(NFB)        I*4  BEAM SECTION  IDENTIFICATION NO.
C     N8  ICP(NFC)        I*4  COLUMN SECTION  IDENTIFICATION NO.
C     N9  IBCT(2,NFB)     I*4  CONNECTION TYPE IDENTIFICATION NO.
C     N10 FK(2,NBF)       R*8  CONNECTION MOMENT MULTIPLICATION FACTOR
C     N11 CS(2,NFB)       R*8  CONNECTION STIFFNESS
C     N12 SS(NBAND,NEQ)   R*8  GLOBAL STIFFNESS MATRIX
C     N13 JD(3,NTJ)       R*8  CALCULATED DISPLACEMENT VECTOR
C     N14 AJL(3,100)      R*8  JOINT LOAD VECTOR
C     N15 BFA(3,NFB)      R*8  BEAM A END FORCES
C     N16 BFB(3,NFB)      R*8  BEAM B END FORCES
C     N17 CFA(3,NFC)      R*8  COLUMN A END FORCES
C     N18 CFB(3,NFC)      R*8  COLUMN B END FORCES
C     N19 CIA(3,NFC)      R*8  COLUMN A END IN-PLANE FORCES
C     N20 CIB(3,NFC)      R*8  COLUMN B END IN-PLANE FORCES
C     N21 ALMC(3,100)     R*8  BEAM CONCENTRATED LOAD
C     N22 ALMU(100)       R*8  BEAM UNIFORMLY DISTRIBUTED LOAD
C     N23 MU(100)         I*4  UNIFORMILY LOADED BEAM MEMBER NOS.
C     N24 MJ(100)         I*4  LOADED JOINT NUMBER
C     N25 MC(100)         I*4  CONCENTRATED LOADED BEAM MEMBER NOS.
C     N26 INT(2,NFT)      I*4  BRA AUSEINCIDENCE TABLE
C     N27 ITP(NFT)        I*4  BRACING PROPERTY IDENTIFICATION NUMBER
C     N28 TFA(NFT)        R*8  BRACING A END FORCES
C     N29 TFB(NFT)        R*8  BRACING B END FORCES
C     N30 AMO(2,NFB)
C     N31 RO(2,NFB)
C     N32 AN(2,NFB)
C     N33 SP(2,NFB)
C     N34 BENDF(2,NFB)
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C     N35 ROTAT(2,NFB)
C     N36 BENDFAB(2,NFB)
C     N37 ROTATAB(2,NFB)
C     N38 BENDFL(2,NFB)
C     N39 NTYPE(2,NFB)
C     N40 ROTAT3(2,NFB)
C     N41 BENDF3(2,NFB)
C     N42 ICT(2,NCOMB)
C     N43 SIGN0(2,NFB)
C     N44 DISP(3,NTJ)
C     N45 ROT(3,NTJ)
C     N46 RR(2,NFB)
C     N47 PP(NFC)
C     N48 JDD(3,NTJ)
C
      N1=1
      N2=N1+3*NTJ
      N3=N2+2*NFB
      N4=N3+NBAY1
      N5=N4+NBAY3
      N6=N5+NST
      N7=N6+2*NFC
      N8=N7+NFB
      N9=N8+NFC
      N10=N9+2*NFB
      N11=N10+2*NFB
      N12=N11+2*NFB
      N13=N12+NXM
      N14=N13+NEQ
      N15=N14+3*100
      N16=N15+3*NFB
      N17=N16+3*NFB
      N18=N17+3*NFC
      N19=N18+3*NFC
      N20=N19+3*NFC
      N21=N20+3*NFC
      N22=N21+3*100
      N23=N22+100
      N24=N23+100
      N25=N24+100
      N26=N25+100
      N27=N26+2*NFT
      N28=N27+NFT
      N29=N28+NFT
      N30=N29+NFT
      N31=N30+2*NFB
      N32=N31+2*NFB
      N33=N32+2*NFB
      N34=N33+2*NFB
      N35=N34+2*NFB
      N36=N35+2*NFB
      N37=N36+2*NFB
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```
      N38=N37+2*NFB
      N39=N38+2*NFB
      N40=N39+2*NFB
      N41=N40+2*NFB
      N42=N41+2*NFB
      N43=N42+2*NCOMB
      N44=N43+2*NFB
      N45=N44+3*NTJ
      N46=N45+3*NTJ
      N47=N46+3*NTJ
      N48=N47+NFC
C
401   CONTINUE
      NMODIF=0
      NPDT=1
      NPD=1
      NCOUNT=1
      IF (NLJ.EQ.0.AND.NLOADTM.EQ.1) IPDELT=2
405   IF (REAL(NLOADTM)/2.EQ.REAL(NLOADTM/2)) THEN
        NTA=NTA-1
        IPDELT=2
      ELSE IF (NLT1.NE.1.AND.NLOADTM.GT.1) THEN
        IPDELT=JPD
      END IF
      IF(NLOADTM.GT.1) GO TO 8
408   CALL GENINP(A(N1),A(N2),A(N3),A(N4),A(N5),A(N6),A(N7),A(N8))
      IF(NLOADTM.EQ.1.AND.NITER.EQ.1) CALL FUNCTY(A(N2),A(N34),A(N35),
     *A(N36),A(N37),A(N39),A(N40),A(N41))
6     CALL CINFO(A(N13),A(N9),A(N2),A(N11),A(N15),A(N16),NTA,
     *        A(N30),A(N31),A(N32),A(N33),A(N34),A(N35),A(N36),
     *        A(N37),A(N39),A(N40),A(N41),A(N42),NCOMB,A(N43),
     *        NCOUNT,NLOADTM,A(N46),NGEOM)
8     CONTINUE
C
      IF(REAL(NLOADTM)/2.EQ.REAL(NLOADTM/2).OR.NLOADTM.EQ.1.OR.NTA.NE.4)
     *GO TO 4
      CALL UNLOAD(A(N9),A(N11),A(N30),A(N31),A(N32),A(N33),A(N34),A(N35)
     *        ,A(N36),A(N38),A(N39),A(N40),A(N41),A(N43),NLOADTM)
4     IF (NFT.EQ.0)GO TO 410
      CALL INTRS(A(N26),A(N27),NLOADTM)
410   CALL COL(A(N1),A(N6),A(N8),A(N12),A(N47),NITER,NLOADTM)
      CALL BEAM(A(N1),A(N2),A(N7),A(N11),A(N12))
      IF(NBF.GT.0)CALL BRACE(A(N1),A(N26),A(N12),A(N27))
      IF(NITER.GT.1.OR.NPDT.GT.1)GO TO 420
      CALL LOAD(A(N1),A(N2),A(N7),A(N11),A(N13),
     áA(N14),A(N15),A(N16),A(N21),A(N22),A(N23),A(N24),A(N25),NLOADTM)
      GO TO 430
420   CALL RELOAD(A(N1),A(N2),A(N7),A(N11),A(N13),A(N14),A(N15),A(N16),A
     *(N21),A(N22),A(N23),A(N24),A(N25))
430   CALL SPT(A(N12),A(N13),NLOADTM,NITER)
      CALL EQN(A(N1),A(N12),A(N13),A(N44),A(N45))
      CALL BMF(A(N1),A(N2),A(N7),A(N11),A(N13),A(N15),A(N16))
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      CALL CMF(A(N1),A(N6),A(N8),A(N13),A(N17),A(N18),A(N19),A(N20),
     *        A(N47),NLOADTM,NITER)
      IF( NBF.GT.0)CALL FBRACE(A(N1),A(N26),A(N27),A(N13),A(N28),A(N29))
      NPCODE=0
      NCCODE=0
C
C     P-DELTA AND CONNECTION DEFORMATION NOT INCLUDED IN ANALYSIS
C
      IF(IPDELT.EQ.2.AND.NTA.LT.4)GO TO 440
C
C     CONNECTION DEFORMATION AND P-DELTA INCLUDED IN ANALYSIS
C
      IF(IPDELT.EQ.1.AND.NTA.EQ.4)GO TO 450
C
C     ONLY P-DELTA INCLUDED IN ANALYSIS
C
      IF(IPDELT.EQ.1.AND.NTA.LT.4)GO TO 450
C
C     ONLY CONNECTION DEFORMATION INCLUDED IN ANALYSIS
C
      IF(IPDELT.EQ.2.AND.NTA.EQ.4)GO TO 460
C
450   IF(NPDT.LT.10)GO TO 470
      WRITE(*,'('' P-DELTA EFFECT DOES NOT STABILIZE'')')
      NPCODE=2
      IF(NTA.LT.4)GO TO 440
      GO TO 460
C
470   IF(NPD.GT.0)GO TO 480
      NPCODE=1
      IF(NTA.LT.4)GO TO 440
      GO TO 460
C
480   CALL PDELTA(A(N1),A(N2),A(N6),A(N7),A(N8),A(N11),A(N13),A(N14),
     *        A(N47), NPD,NPDT)
      NPDT=NPDT+1
488   IF(NTA.LT.4)GO TO 410
C
460   IF(NITER.LT.10)GO TO 490
C
      WRITE(*,'('' STIFFER CONNECTION NEEDED'')')
      GOTO 510
C
490   IF(NPCODE.EQ.2)GO TO 440
      IF(NCOUNT.GT.0)GO TO 500
      NCCODE=1
      IF(NPCODE.EQ.1.AND.NCCODE.EQ.1)GO TO 440
      IF(IPDELT.EQ.2)GO TO 440
      GO TO 410
C
500   CALL ITER(A(N13),A(N2),A(N9),A(N11),A(N15),A(N16),NCOUNT,A(N30),
     *        A(N31),A(N32),A(N33),A(N34),A(N35),A(N36),A(N37),A(N39),
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```
     *        A(N40),A(N41),NLOADTM,A(N42),NCOMB,A(N43),A(N46),NITER)
      NITER=NITER+1
      IF(IPDELT.EQ.2)GO TO 410
      IF(NPCODE.EQ.1)NPD=1
      GO TO 410
440   CONTINUE
510   CALL OUT(A(N2),A(N6),A(N13),A(N15),A(N16),A(N17),A(N18),A(N30),
     *        A(N31),A(N34),A(N35),A(N36),A(N37),A(N39),A(N40),A(N41),
     *        NLOADTM,A(N44),A(N45),A(N46),NTA)
      IF (REAL(NLOADTM)/2.EQ.(NLOADTM/2)) THEN
        NTA=NTA+1
        IPDELT=JPD
      END IF
      IF(NLOADTM.EQ.(2*NLOAD-1)) GO TO 404
      NLOADTM=NLOADTM+1
C     NEW SCREEN
      CALL SC
      WRITE(*,444) NLOADTM
C
      IF(REAL(NLOADTM)/2.NE.REAL(NLOADTM/2)) GO TO 400
C
444   FORMAT(//10X,'THE NUMBER OF LOAD SYSTEM IS :',I5/)
      WRITE(*,'(//10X,''NUMBER OF LOADED JOINT:'')')
      CALL RD(NLJ,RNLJ,55)
      WRITE(*,'(//10X,''NUMBER OF UNIFORMLY LOADED BEAMS:'')')
      CALL RD(NLMU,RNLMU,55)
      WRITE(*,'(//10X,''NUMBER OF BEAMS WITH CONCENTRATED LOADS:'')')
      CALL RD(NLMC,RNLMC,55)
C
C     NEW SCREEN
      CALL SC
      WRITE(*,1150) NLJ,NLMU,NLMC
      WRITE(*,1032)
      WRITE(*,'(//10X,''NUMBER OF LOADED JOINT:'',I5)')NLJ
      CALL ED(NLJ,55)
      WRITE(*,'(//10X,''NUMBER OF UNIFORMLY LOADED BEAMS:'',I5)')NLMU
      CALL ED(NLMU,55)
      WRITE(*,'(//10X,''NUMBER OF BEAMS WITH CONCENTRATED LOADS:'',I5)')
     *NLMC
      CALL ED(NLMC,55)
      GO TO 400
404   CLOSE(3)
      CLOSE(9)
      STOP
      END
C
C     THIS SUBROUTINE SETS BLUE BACKGROUND, WHITE PRINTING,
C     CLEARS SCREEN AND PLACES CURSOR AT "HOME"
      SUBROUTINE CLSC
      WRITE(*,'(4(A)\)')CHAR(27),'[2J',CHAR(27),'[44;37m'
      RETURN
      END
```

```
C
C   THIS SUBROUTINE TURNS ON REVERSE VIDEO
    SUBROUTINE VIDON
    WRITE(*,'(2(A)\)')CHAR(27),'[7m'
    RETURN
    END
C
C   THIS SUBROUTINE TURNS OFF REVERSE VIDEO
    SUBROUTINE VIDOF
    WRITE(*,'(2(A)\)')CHAR(27),'[0;44;37m'
    RETURN
    END
C
C   THIS SUBROUTINE STARTS NEW SCREEN OR TERMINATES EXECTUTION
    SUBROUTINE SC
    CHARACTER S/'S'/
    WRITE(*,'(//10X,''TYPE "STOP" TO QUIT, "ENTER" TO CONTINUE'')')
    IS=0
    READ(*,'(A)')S
    IF(S.EQ.'S')STOP '          PROGRAM EXECUTION TERMINATED'
    CALL CLSC
    RETURN
    END
C
C   THIS SUBROUTINE READS AN ALPHANUMERIC STRING AND
C    TRIES TO CONVERT IT INTO A NUMBER
    SUBROUTINE RD(INT,RL,IC)
    REAL*8 RL
    CHARACTER IN(14)/'1','2','3','4','5','6','7','8','9','0','-',' ',
   *'.','+'/
    CHARACTER INP(10)
    IE=0
3888 CALL BACK(1,IC)
    CALL VIDON
    READ (*,'(10A1)')INP
    RL=0.
    M=1
    L=0
    ISIGN=1
    DO 100 K=1,10
    I=11-K
    J=-1
    IF(INP(I).NE.IN(13)) GO TO 20
      RL=RL/10.**L
      L=0
    GO TO 100
20  IF(INP(I).EQ.IN(12).OR.INP(I).EQ.IN(14))GO TO 100
    DO N=1,9
      IF(INP(I).EQ.IN(N))J=N
    END DO
    IF(INP(I).EQ.IN(10))J=0
    IF(INP(I).EQ.IN(11))THEN
```

```
        ISIGN=-1
        GO TO 100
      ELSE IF(J.LT.0)THEN
        GO TO 50
      END IF
      RL=RL+J*10**L
      L=L+1
      GO TO 100
50    CALL VIDOF
      WRITE(*,'(10X,''PLEASE RE-ENTER                        '')')
      IE=1
      CALL BACK(2,IC)
      WRITE(*,'(''          '')')
      GO TO 3888
100   CONTINUE
      RL=RL*ISIGN
      INT = RL
      CALL VIDOF
      IF(IE.EQ.0)GO TO 110
      WRITE(*,'(10X,''                               '')')
      CALL BACK(3,1)
      WRITE(*,'(/)')
110   CONTINUE
      RETURN
      END
C
C     THIS SUBROUTINE EDITS INTEGER DATA
C
      SUBROUTINE ED(IN,J)
      REAL*8 RL
      CALL RD(INT,RL,J)
      IF(INT.NE.0)IN=INT
      RETURN
      END
C
C     THIS SUBROUTINE EDITS REAL DATA
C
      SUBROUTINE EDR(RE,J)
      REAL*8 RE,RL
      CALL RD(INT,RL,J)
      IF(RL.NE.0.)RE=RL
      RETURN
      END
C
C     THIS SUBROUTINE MOVES CURSOR BACK IR ROWS AND TO COLUMN IC
      SUBROUTINE BACK(IR,IC)
      WRITE(*,'(A,A,I2.2,A\)')CHAR(27),'[',IR,'A'
      WRITE(*,'(A,A,I2.2,A\)')CHAR(27),'[',IC,'C'
      RETURN
      END
C
C     THIS SUBROUTINE INTERPRETS A YES OR NO RESPONSE
```

```
      SUBROUTINE YN(NANS)
      CHARACTER*1 Y
      NANS=1
      IE=0
   10 CALL BACK(1,55)
      CALL VIDON
      READ(*,'(A1)')Y
      CALL VIDOF
      IF(Y.EQ.'Y'.OR.Y.EQ.'y'.OR.Y.EQ.'N'.OR.Y.EQ.'n')GO TO 20
      WRITE(*,'(10X,''PLEASE RE-ENTER      '')')
      IE=1
      CALL BACK(2,55)
      WRITE(*,'(''           '')')
      GO TO 10
   20 IF(Y.NE.'Y'.AND.Y.NE.'y')NANS=2
      IF(IE.EQ.0)GO TO 30
      WRITE(*,'(10X,''                    '')')
      CALL BACK(2,1)
      WRITE(*,'(/)')
   30 CONTINUE
      RETURN
      END
C
C     THIS SUBROUTINE EXECUTES A PRINTER FORM FEED
C
      SUBROUTINE FF
      WRITE(3,'(A)')CHAR(12)
      RETURN
      END
C
C     THIS SUBROUTINE GENERATES THE JOINT NUMBERS, JOINT COORIDINATES
C     BEAM MEMBER NUMBERS AND COLUMN MEMBER NUMBERS, BEAM AND COLUMN
C     INCIDENCE TABLES FOR THE RECTANGULAR STRUCTURE ANALYZED.
C
      SUBROUTINE GENINP(CN,INB,SPAN1,SPAN3,DHE,INC,IBP,ICP)
      IMPLICIT REAL*8 (A-H,O-Z)
      COMMON /IPT/NST,NBAY1,NBAY3
      COMMON /GEN/NTJ,NFB,NFC,NBAND,NEQ,NXM
      COMMON /SECT/BP(4,20),CP(4,20)
      COMMON /TEMPB1/NPRT,NFIL,IPDELT
      DIMENSION CN(3,NTJ),SPAN1(NBAY1),SPAN3(NBAY3),DHE(NST),INB(2,NFB),
     *INC(2,NFC),IBP(NFB),ICP(NFC)
      REAL*8 MNODE1,MNODE3
C
C     IF USING OLD INPUT FILE, READ FROM "DIMENS.IN":
C         MASTER NODE COORDINATES- X1' AND X3'
C         LENGTH OF EACH BAY IN X1' DIRECTION
C         LENGTH OF EACH BAY IN X3' DIRECTION
C         HEIGHT OF EACH STOREY
C
      IF(NFIL.EQ.1)GO TO 50
C
```

```
      OPEN(2,FILE='DIMENS.IN')
      READ(2,*)MNODE1,MNODE3,HEI,(SPAN1(N),N=1,NBAY1),(SPAN3(N),N=1,NBAY
     *3),(DHE(N),N=1,NST)
      CLOSE(UNIT=2)
      GO TO 90
C
C     OTHERWISE INPUT THE INFORMATION
C
50    CALL CLSC
      WRITE(*,'(//10X,''INPUT THE FOLLOWING DIMENSIONS IN ''\)')
60    WRITE(*,'(''METRES'')')
80    WRITE(*,'(//10X,''X COORDINATE OF MASTER NODE:'')')
      CALL RD(INT,MNODE1,55)
      WRITE(*,'(//10X,''Z COORDINATE OF MASTER NODE:'')')
      CALL RD(INT,MNODE3,55)
      CALL SC
      WRITE(*,'(//10X,''BAY LENGTHS IN X DIRECTION:'')')
      DO N=1,NBAY1
        CALL RD(INT,SPAN1(N),55)
        WRITE(*,'(/)')
      END DO
      CALL SC
      WRITE(*,'(//10X,''BAY LENGTHS IN Z DIRECTION:'')')
      DO N=1,NBAY3
        CALL RD(INT,SPAN3(N),55)
        WRITE(*,'(/)')
      END DO
      CALL SC
      WRITE(*,'(//10X,''STOREY HEIGHTS(TOP TO BOTTOM):'')')
      HEI=0.
      DO N=1,NST
        CALL RD(INT,DHE(N),55)
        HEI=HEI+DHE(N)
        WRITE(*,'(/)')
      END DO
C
C     EDIT ABOVE INPUT ?
C
90    WRITE(*,1005)
1005  FORMAT(//10X,'EDIT:'/15X,'COORDINATES OF MASTER NODE'/15X,'BAY LEN
     *GTHS'/15X,'STOREY HEIGHTS ?  YES or NO:')
1000  FORMAT(//10X,'EDIT INPUT ? YES or NO:')
      CALL YN(IY)
      IF(IY.NE.1)GO TO 100
      CALL CLSC
      WRITE(*,1010)MNODE1,MNODE3
1010  FORMAT(//10X,'X COORDINATE OF MASTER NODE . . . . . . ',F10.1/
     *          10X,'Z COORDINATE OF MASTER NODE . . . . . . ',F10.1/)
      WRITE(*,1020)
1020  FORMAT(//10X,'TO MODIFY, INPUT NEW VALUE.  OTHERWISE "RETURN"')
      WRITE(*,'(10X,''X COORDINATE OF MASTER NODE        '',F10.1)')MNODE1
      CALL EDR(MNODE1,55)
```

```
      WRITE(*,'(10X,''Z COORDINATE OF MASTER NODE       '',F10.1)')MNODE3
      CALL EDR(MNODE3,55)
      CALL SC
      WRITE(*,1020)
      WRITE(*,'(//10X,''BAY LENGTHS IN X DIRECTION''//10X,'' BAY   LENGT
     *H''/)')
      DO N=1,NBAY1
        WRITE(*,'(10X,I4,15X,F10.1)')N,SPAN1(N)
        CALL EDR(SPAN1(N),55)
      END DO
      CALL SC
      WRITE(*,1020)
      WRITE(*,'(//10X,''BAY LENGTHS IN Z DIRECTION''//10X,'' BAY   LENGT
     *H''/)')
      DO N=1,NBAY3
        WRITE(*,'(10X,I4,15X,F10.1)')N,SPAN3(N)
        CALL EDR(SPAN3(N),55)
      END DO
      CALL SC
      WRITE(*,1020)
      WRITE(*,'(//10X,''STOREY HEIGHTS''/10X,''STOREY       HEIGHT''//)')
      HEI=0.
      DO N=1,NST
        WRITE(*,'(10X,I4,15X,F10.1)')N,DHE(N)
        CALL EDR(DHE(N),55)
        HEI=HEI+DHE(N)
      END DO
C
C     WRITE TO "OLD" INPUT FILE:
C         COORDINATES OF MASTER NODE (MNODE1, MNODE3)
C         TOTAL HEIGHT OF STRUCTURE
C         BAY SPANS IN X AND Z DIRECTIONS
C         STOREY HEIGHTS
C
100   OPEN(2,FILE='DIMENS.IN')
      WRITE(2,*)MNODE1,MNODE3,HEI,(SPAN1(N),N=1,NBAY1),(SPAN3(N),N=1,NBA
     *Y3),(DHE(N),N=1,NST)
      CLOSE(2)
C
C     GENERATE JOINT NUMBERS AND COORDINATES
C
      NJ=0
      TEMP1=0.D0
      TEMP3=0.D0
      NSH=NST+1
      NBAY1P=NBAY1+1
      NBAY3P=NBAY3+1
      DO N=1,NSH
        NJ=NJ+1
        CN(1,NJ)=TEMP1
        CN(2,NJ)=HEI
        CN(3,NJ)=TEMP3
```

```
      DO NN=1,NBAY1P
        DO MN=1,NBAY3
          NJ=NJ+1
          CN(1,NJ)=TEMP1
          CN(2,NJ)=HEI
          CN(3,NJ)=CN(3,NJ-1)+SPAN3(MN)
        END DO
        IF(NN.EQ.NBAY1P)GO TO 140
        TEMP1=TEMP1+SPAN1(NN)
        NJ=NJ+1
        CN(1,NJ)=TEMP1
        CN(3,NJ)=TEMP3
        CN(2,NJ)=HEI
      END DO
140   NJ=NJ+1
      CN(1,NJ)=MNODE1
      CN(2,NJ)=HEI
      CN(3,NJ)=MNODE3
      TEMP1=0.D0
      IF(N.EQ.NSH)GO TO 150
      HEI=HEI-DHE(N)
150   CONTINUE
    END DO
C
    WRITE(*,'(//10X,''DO YOU WANT JOINT COORDINATES AND''/10X,''MEMBER
   * INCIDENCE TABLES ?  YES or NO:      '')')
    CALL YN(IJC)
    IF(IJC.NE.1)GO TO 240
    WRITE(*,1030)
1030 FORMAT(//10X,'JOINT NUMBERS AND COORDINATES'/10X,'JOINT      X COO
   *RDINATE    Y COORDINATE   Z COORDINATE')
    DO N=1,NTJ
      WRITE(*,1040)N,(CN(K,N),K=1,3)
1040  FORMAT(10X,I5,3(6X,F10.2))
    END DO
C
C   CONVERT TO IN. OR MM.
C
240 FACT=1000.
    DO KK=1,NTJ
      CN(1,KK)=CN(1,KK)*FACT
      CN(2,KK)=CN(2,KK)*FACT
      CN(3,KK)=CN(3,KK)*FACT
    END DO
C
C   GENERATE BEAM INCIDENCE TABLE
C    NB = FIRST JOINT
C    NE = LAST JOINT
C
    NB=1
    NE=(NBAY1+1)*(NBAY3+1)
    NEB=NE
```

```
      JS=NB-1
      JE=JS+1
      MM=0
      DO NN=1,NST
        DO N=1,NBAY1P
          DO M=1,NBAY3
            MM=MM+1
            JS=JS+1
            JE=JE+1
            INB(1,MM)=JS
            INB(2,MM) = JE
          END DO
          JS=JE
          JE=JS+1
        END DO
        JS=NB
        JE=JS+NBAY3+1
        DO N=1,NBAY3P
          DO M=1,NBAY1
            MM=MM+1
            INB(1,MM)=JS
            INB(2,MM)=JE
            JS=JE
            JE=JE+(NBAY3+1)
          END DO
          JS=NB+N
          JE=JS+NBAY3+1
        END DO
        NB=NE+1
        NE=NB+NEB
        JS=NB
        JE=JS+1
        NB=NB+1
      END DO
C
C  GENERATE COLUMN INCIDENCE TABLE
C
      MM=0
      NEC=NFC/NST
      NE=0
      NB=NFC/NST+1
      JS=NB
      JE=NE
      DO NN=1,NST
        DO N=1,NBAY1P
          DO M=1,NBAY3P
            MM=MM+1
            JS=JS+1
            JE=JE+1
            INC(1,MM)=JS
            INC(2,MM)=JE
          END DO
```

```
       END DO
       NE=NB
       NB=NB+NEC+1
       JS=NB
       JE=NE
     END DO
C
     IF(IJC.NE.1) GO TO 290
     CALL SC
     WRITE(*,1045)
1045 FORMAT(//10X,'BEAM INCIDENCE TABLE'/10X,'BEAM',5X,'JOINT AT A END'
    *,5X,'JOINT AT B END')
     NBE=NFB/NST
     WRITE(*,'(//)')
     MM=1
     MN=NBE
     DO N=1,NST
       WRITE(*,1050)N
1050   FORMAT(//10X,'STOREY LEVEL:',I5/)
       DO M=MM,MN
         WRITE(*,1060)M,INB(1,M),INB(2,M)
       END DO
1060     FORMAT (10X,I5,10X,I5,10X,I5)
         MM=MN+1
         MN=(N+1)*NBE
     END DO
     CALL SC
     WRITE(*,1070)
1070 FORMAT (//10X,'COLUMN INCIDENCE TABLE'/10X,'COLUMN',5X,'JOINT AT A
    * END',5X,'JOINT AT BEND')
     MM=1
     MN=NEC
     DO N=1,NST
       WRITE(*,1050)N
       DHE(N)=DHE(N)*FACT
       DO M=MM,MN
         WRITE(*,1060)M,INC(1,M),INC(2,M)
       END DO
       MM=MN+1
       MN=(N+1)*NEC
     END DO
290  CALL SC
     IF(NFIL.EQ.1)GO TO 300
C
C    READ FROM "OLD" INPUT FILE:
C         - NUMBER OF BEAM CROSS-SECTION TYPES
C         - CROSS-SECTIONAL PROPERTIES FOR EACH
C
     OPEN(2,FILE='BMPROP.IN')
     READ(2,*)NBP,((BP(J,I),J=1,4),I=1,NBP)
     CLOSE(2)
     GO TO 340
```

```
C
C   OR INPUT NEW BEAM CROSS-SECTIONAL PROPERTIES
C
300 WRITE(*,'(//10X,''INPUT NUMBER OF BEAM CROSS-SECTION TYPES'')')
    CALL RPROP(NBP,BP)
340 WRITE(*,'(//10X,''EDIT BEAM CROSS-SECTION PROPERTIES ?''/10X,''YES
   * or NO:'')')
    CALL YN(IY)
    IF(IY.EQ.2)GO TO 360
C
C   EDIT BEAM CROSS-SECTIONAL PROPERTIES
C
    CALL EPROP(NBP,BP)
C
C   WRITE BEAM CROSS-SECTION PROPERTIES TO "OLD" FILE
C
360 OPEN(2,FILE='BMPROP.IN')
    WRITE(2,*)NBP,((BP(J,I),J=1,4),I=1,NBP)
    CLOSE(2)
C
C   READ FROM "OLD" INPUT FILE:
C       - NUMBER OF COLUMN CROSS-SECTION TYPES
C       - CROSS-SECTIONAL PROPERTIES FOR EACH
C
370 CALL SC
    IF(NFIL.EQ.1)GO TO 380
    OPEN(2,FILE='COLPROP.IN')
    READ(2,*)NCP,((CP(J,I),J=1,4),I=1,NCP)
    CLOSE(2)
    GO TO 390
C
C   OR INPUT NEW COLUMN CROSS-SECTIONAL PROPERTIES
C
380 WRITE(*,'(//10X,''INPUT NUMBER OF COLUMN CROSS-SECTION TYPES'')')
    CALL RPROP(NCP,CP)
390 WRITE(*,'(//10X,''EDIT COLUMN CROSS-SECTION PROPERTIES ?''/10X,''Y
   *ES or NO:'')')
    CALL YN(IY)
    IF(IY.EQ.2) GO TO 430
C
C   EDIT COLUMN CROSS-SECTIONAL PROPERTIES
C
    CALL EPROP(NCP,CP)
C
C   WRITE COLUMN CROSS-SECTIONAL PROPERTIES TO "OLD" FILE
C
430 OPEN(2,FILE='COLPROP.IN',STATUS='UNKNOWN')
    WRITE(2,*)NCP,((CP(J,I),J=1,4),I=1,NCP)
    CLOSE(2)
C
C   READ BEAM TYPE IDENTIFIERS FROM "OLD" INPUT FILE
C
```

```
      IF (NFIL.EQ.1) GO TO 440
      OPEN(2,FILE='BMTYP.IN',STATUS='OLD')
      READ(2,*)(IBP(I),I=1,NFB)
      CLOSE(2)
      GO TO 450
C
C     OR INPUT BEAM TYPE IDENTIFIERS
C
440   CALL SC
      WRITE (*,1140)
1140  FORMAT(//10X,'BEAM "TYPES" (1, 2 ,3, ETC.), IN FOLLOWING ORDER'/15
     *X,'- AXES IN Z DIRECTION, IN ORDER OF INCREASING Z;'/15X,'- AXES I
     *N X DIRECTION, IN ORDER OF INCREASING X;'/15X,'- BOTTOM FLOOR TO T
     *OP'/)
      WRITE(*,'(10X,''TOTAL NUMBER OF BEAMS ='',I3//)')NFB
      CALL RTYPE(NFB,IBP,NBP)
C
C     EDIT BEAM TYPE IDENTIFIERS
C
450   WRITE(*,'(//10X,''EDIT BEAM TYPE IDENTIFIERS ?''/10X,''YES or NO''
     *)')
      CALL YN(IY)
      IF(IY.EQ.2)GO TO 460
      CALL CLSC
      WRITE(*,1140)
      CALL ETYPE(NFC,IBP,NBP)
C
C     WRITE BEAM TYPE IDENTIFIERS TO "OLD" FILE
C
460   OPEN(2,FILE='BMTYP.IN',STATUS='UNKNOWN')
      WRITE(2,*)(IBP(I),I=1,NFB)
      CLOSE(2)
C
C     READ COLUMN TYPE IDENTIFIERS FROM "OLD" INPUT FILE
C
      IF(NFIL.EQ.1)GO TO 470
      OPEN(2,FILE='COLTYP.IN')
      READ(2,*)(ICP(I),I=1,NFC)
      CLOSE(2)
      GO TO 480
C
C     OR INPUT COLUMN TYPE IDENTIFIERS
C
470   CALL SC
      WRITE (*,1160)
1160  FORMAT(//10X,'COLUMN "TYPES" (1, 2 ,3, ETC.), IN FOLLOWING ORDER'/
     *15X,'- ALONG Z GRID LINES, IN ORDER OF INCREASING Z;'/15X,'- THEN
     *IN ORDER OF INCREASING X;'/15X,'- THEN BOTTOM FLOOR TO TOP'/)
      WRITE(*,'(10X,''TOTAL NUMBER OF COLUMNS ='',I3//)')NFC
      CALL RTYPE(NFC,ICP,NCP)
C
C     EDIT COLUMN TYPE IDENTIFIERS
```

```
C
480  WRITE(*,'(//10X,''EDIT COLUMN TYPE IDENTIFIERS ?''/10X,''YES or NO
    *'')')
     CALL YN(IY)
     IF(IY.EQ.2)GO TO 490
     CALL CLSC
     WRITE(*,1160)
     CALL ETYPE(NFC,ICP,NCP)
C
C    WRITE COLUMN TYPE IDENTIFIERS TO "OLD" FILE
C
490  OPEN(2,FILE='COLTYP.IN',STATUS='UNKNOWN')
     WRITE(2,*)(ICP(I),I=1,NFC)
     CLOSE(2)
C
C    WRITE INPUT DATA TO OUTPUT FILE
C
C    MASTER NODE COORDINATES, BAY LENGTHS AND STOREY HEIGHTS
C
     CALL FF
     WRITE(3,'(//10X,''X AND Z COORDINATES OF MASTER NODE:''/5X,2(5X,F1
    *0.1))')MNODE1,MNODE3
     WRITE(3,'(//10X,''SPAN LENGTHS IN X DIRECTION:''/5X,4(5X,F10.1))')
    *(SPAN1(N),N=1,NBAY1)
     WRITE(3,'(//10X,''SPAN LENGTHS IN Z DIRECTION:''/5X,4(5X,F10.1))')
    *(SPAN3(N),N=1,NBAY3)
     WRITE(3,'(//10X,''STOREY HEIGHTS (TOP TO BOTTOM):''/(5X,4(5X,F10.1
    *)))')(DHE(I),I=1,NST)
     IF(IJC.EQ.2)GO TO 500
     CALL FF
     WRITE(3,1030)
     DO N=1,NTJ
       WRITE(3,1040)N,(CN(K,N)/FACT,K=1,3)
     END DO
     CALL FF
     WRITE(3,1045)
     NBE=NFB/NST
     MM=1
     MN=NBE
     DO N=1,NST
       WRITE(3,1050)N
       DO M=MM,MN
         WRITE(3,1060)M,(INB(K,M),K=1,2)
       END DO
         MM=MN+1
         MN=(N+1)*NBE
     END DO
     CALL FF
     WRITE(3,1070)
     MM=1
     MN=NEC
     DO N=1,NST
```

```
        WRITE(3,1050)N
        DO M=MM,MN
          WRITE(3,1060)M,(INC(K,M),K=1,2)
        END DO
        MM=MN+1
        MN=(N+1)*NEC
      END DO
C
C     BEAM TYPES AND CROSS-SECTIONAL PROPERTIES
C
500   CALL FF
      WRITE(3,'(//10X,''BEAM TYPES AND CROSS-SECTIONAL PROPERTIES'')')
      WRITE(3,1102)
1102  FORMAT(10X,'TYPE    TORS. CONST.    IYY       IZZ        AREA')
1100  FORMAT(//10X,'INPUT CROSS-SECTIONAL PROPERTIES FOR EACH TYPE')
510   WRITE(3,1110)
1110  FORMAT(30X,'(MM^4/10^6)               (MM^2)'/)
530   DO I=1,NBP
        WRITE(3,1090)I,(BP(J,I),J=1,4)
1090    FORMAT(10X,I3,4F12.1)
      END DO
C
C     BEAM TYPE IDENTIFIERS
C
      WRITE(3,1140)
      WRITE(3,1150)(IBP(I),I=1,NFB)
1150  FORMAT(8X,20I3/)
C
C     COLUMN TYPES AND CROSS-SECTIONAL PROPERTIES
C
      CALL FF
      WRITE(3,'(//10X,''COLUMN TYPES AND CROSS-SECTIONAL PROPERTIES'')')
      WRITE(3,1102)
540   WRITE(3,1110)
C
C     COLUMN TYPE IDENTIFIERS
C
560   DO I=1,NCP
        WRITE(3,1090)I,(CP(J,I),J=1,4)
      END DO
      WRITE (3,1160)
      WRITE(3,1150)(ICP(I),I=1,NFC)
      RETURN
      END
C
C     THIS SUBROUTINE READS MEMBER CROSS-SECTIONAL PROPERTIES
C
      SUBROUTINE RPROP(NBP,BP)
      IMPLICIT REAL*8 (A-H,O-Z)
      COMMON /TEMPB1/NPRT,NFIL,IPDELT
      DIMENSION BP(4,20)
      CALL RD(NBP,RL,55)
```

```
      WRITE(*,'(//10X,''INPUT CROSS-SECTIONAL PROPERTIES FOR EACH TYPE''
     *)')
      WRITE(*,'(10X,''TYPE    TORS. CONST.    IYY      IZZ        AREA''
     *)')
310   WRITE(*,'(30X,''(MM^4/10^6)            (MM^2)''/)')
330   DO I=1,NBP
        WRITE(*,'(10X,I5)')I
        J=11
        DO K=1,4
          J=J+11
          CALL RD(INT,BP(K,I),J)
        END DO
      END DO
      RETURN
      END
C
C     THIS SUBROUTINE EDITS MEMBER CROSS-SECTIONAL PROPERTIES
C
      SUBROUTINE EPROP(NBP,BP)
      IMPLICIT REAL *8 (A-H,O-Z)
      COMMON /TEMPB1/NPRT,NFIL,IPDELT
      DIMENSION BP(4,20)
      CALL CLSC
      WRITE(*,'(//10X,''TO MODIFY, INPUT NEW VALUE.  OTHERWISE "RETURN"'
     *')')
      WRITE(*,'(//10X,''NUMBER OF BEAM CROSS-SECTION TYPES   '',I5)')NBP
      CALL ED(NBP,55)
      WRITE(*,'(//10X,''PROPERTIES OF BEAM CROSS-SECTION TYPES'')')
      WRITE(*,'(10X,''TYPE    TORS. CONST.    IYY      IZZ        AREA''
     *)')
      DO I=1,NBP
        WRITE(*,'(10X,I3,4F11.1/)')I,(BP(K,I),K=1,4)
        J=11
        DO K=1,4
          J=J+11
          CALL EDR(BP(K,I),J)
        END DO
      END DO
      RETURN
      END
C
C     THIS SUBROUTINE READS MEMBER TYPE IDENTIFIERS
C
      SUBROUTINE RTYPE(NFB,IBP,NBP)
      IMPLICIT REAL*8 (A-H,O-Z)
      DIMENSION IBP(NFB)
      J=7
      DO I=1,NFB
        IE=0
        J=J+3
        IF(J.LE.67)GO TO 445
        J=10
```

```
      WRITE(*,'(/)')
445   CALL RD(IBP(I),RL,J)
      IF(IBP(I).LE.0.OR.IBP(I).GT.NBP)THEN
       IE=1
       WRITE(*,'(10X,''INVALID TYPE, PLEASE RE-ENTER'')')
       CALL BACK(2,J)
       WRITE(*,'(''   '')')
       GO TO 445
      ELSE
      END IF
      IF(IE.EQ.0)GO TO 446
      WRITE(*,'(10X,''                                   '')')
      CALL BACK(3,1)
      WRITE(*,'(/)')
446   CONTINUE
     END DO
     RETURN
     END
C
C    THIS SUBROUTINE EDITS MEMBER TYPE IDENTIFIERS
C
     SUBROUTINE ETYPE(NFB,IBP,NBP)
     IMPLICIT REAL*8 (A-H,O-Z)
     DIMENSION IBP(NFB)
     WRITE(*,'(8X,20I3/)')(IBP(I),I=1,NFB)
     WRITE(*,'(//)')
     J=7
     DO I=1,NFB
      IE=0
      J=J+3
      IF(J.LE.67)GO TO 455
      J=10
      WRITE(*,'(/)')
455   CALL ED(IBP(I),J)
      IF(IBP(I).LE.0.OR.IBP(I).GT.NBP)THEN
       IE=1
       WRITE(*,'(10X,''INVALID TYPE, PLEASE RE-ENTER'')')
       CALL BACK(2,J)
       WRITE(*,'(''   '')')
       GO TO 455
      ELSE
      END IF
      IF(IE.EQ.0)GO TO 456
      WRITE(*,'(10X,''                                   '')')
      CALL BACK(3,1)
      WRITE(*,'(/)')
456   CONTINUE
     END DO
     RETURN
     END
C    THIS SUBROUTINE SETS CONNECTION STIFFNESSES FOR "PINNED" OR "RIGID"
C    READS STIFFNESSES FOR "LINEAR" CONNECTIONS, OR READS PARAMETERS AND
```

```
C   GENERATES MOMENT-ROTATION FUNCTIONS FOR NONLINEAR CONNECTIONS
C
    SUBROUTINE CINFO(JL,IBCT,INB,CS,BFA,BFB,NTA,AMO,RO,AN,SP,BENDF,
   *            ROTAT,BENDFAB,ROTATAB,NTYPE,ROTAT3,BENDF3,
   *            ICT,NCOMB,SIGN0,NCOUNT,NLOADTM,RR,NGEOM)
    IMPLICIT REAL*8 (A-H,O-Z)
    COMMON /GEN/NTJ,NFB,NFC,NBAND,NEQ,NXM
    COMMON /MAT/EM,GM
    COMMON /TEMPB1/NPRT,NFIL,IPDELT
    COMMON /COMR/ R
    DIMENSION IBCT(2,NFB),CS(2,NFB),BFA(3,NFB),BFB(3,NFB),
   *      CST(2,30),ICTYP(100),P(4,2,30),FS(2,30),PP(5),AMO(2,NFB),
   *      RO(2,NFB),AN(2,NFB),SP(2,NFB),BENDF(2,NFB),ROTAT(2,NFB),
   *      BENDFAB(2,NFB),NTYPE(2,NFB),ROTATAB(2,NFB),PG(5,2,5),
   *      INB(2,NFB),JL(3,NTJ),ROTAT3(2,NFB),BENDF3(2,NFB),
   *      ICT(2,NCOMB),SIGN0(2,NFB),RR(2,NFB)
C
    CALL CLSC
    GO TO(100,110,130,180)NTA
C
C   PINNED CONNECTIONS - SET CONNECTION STIFFNESSES VERY SMALL
C
100 SA=1.E-3
    SB=1.E-3
    GO TO 120
C
C    RIGID CONNECTIONS - SET CONNECTION STIFFNESSES VERY LARGE
C
110 SA=1.0E25
    SB=1.0E25
120 DO M=1,NFB
     CS(1,M)=SA
     CS(2,M)=SB
    END DO
    RETURN
C
C   CONSTANT STIFFNESS CONNECTIONS - INPUT STIFFNESSES FOR CONNECTIONS
C   AT START AND END OF ALL BEAMS
C
130 IF(NFIL.EQ.1)GO TO 140
C
C   READ CONNECTION STIFFNESSES FROM "OLD" INPUT FILE
C
    OPEN(2,FILE='LCSTIF.IN')
    READ(2,*)((CS(I,M),I=1,2),M=1,NFB)
    CLOSE(2)
    GO TO 150
C
C   OR INPUT CONNECTION STIFFNESS FOR BOTH ENDS OF ALL BEAMS
C
140 CALL SC
    WRITE(*,'(//10X,''LINEAR STIFFNESSES FOR THE TWO CONNECTIONS AT TH
```

```
    *E''/10X,''ENDS OF A GIVEN BEAM ARE ASSOCIATED WITH THAT BEAM.''/10
    *X,''INPUT STIFFNESSES FOR EACH CONN.-BEAM-CONN. COMBINATION'')')
     WRITE(*,1005)
1005 FORMAT(//10X,'TOTAL NUMBER OF CONNECTION-BEAM-CONNECTION'/10X
    *,'COMBINATIONS:',I5)
     CALL RD(NCOMB,RL,55)
     WRITE(*,'(/10X,''COMBINATION  START CONN.   END CONN.''/)')
     DO M=1,NCOMB
      J=15
      WRITE(*,'(10X,I5)')M
      DO K=1,2
       J=J+12
       CALL RD(INT,CST(K,M),J)
      END DO
     END DO
     CALL SC
     WRITE(*,1000)
1000 FORMAT(//10X,'CONN.-BEAM-CONN. COMBINATIONS (1, 2, 3, ETC.).'
    *       /10X,'IN FOLLOWING ORDER:  AXES IN Z DIRECTION,'
    *       /10X,'IN ORDER OF INCREASING Z, THEN INCREASING X;'
    *       /10X,'THEN AXES IN X DIRECTION, INCREASING X THEN Z;'
    *       /10X,'BOTTOM FLOOR TO TOP'/)
     CALL RTYPE(NFB,ICTYP,NCOMB)
150  WRITE(*,'(//10X,''EDIT LINEAR CONNECTION STIFFNESSES?''//10X,''YES
    * or NO'')')
     CALL YN(IY)
     IF(IY.NE.1)GO TO 160
C
C    EDIT LINEAR CONNECTION STIFFNESSES
C
     CALL SC
     WRITE(*,1015)
1015 FORMAT(//10X,'TO MODIFY, INPUT NEW VALUE.  OTHERWISE "RETURN"'/)
     WRITE(*,1005)NCOMB
     CALL ED(NCOMB,35)
     DO M=1,NCOMB
      WRITE(*,1010)M,CST(1,M),CST(2,M)
1010  FORMAT(10X,'COMBINATION  ',I5,'  STIFFNESSES ',2F11.1/)
      J=37
      DO I=1,2
       J=J+11
       CALL EDR(CST(I,M),J)
      END DO
     END DO
160  WRITE(*,'(//10X,''EDIT CONNECTION STIFFNESS COMBINATIONS''/10X,''Y
    *ES or NO'')')
     CALL YN(IY)
     IF(IY.NE.1)GO TO 170
     CALL SC
     WRITE(*,1015)
     CALL ETYPE(NFB,ICTYP,NCOMB)
170  DO K=1,NFB
```

```
        J=ICTYP(K)
        CS(1,K)=CST(1,J)
        CS(2,K)=CST(2,J)
      END DO
C
C     WRITE LINEAR CONNECTION STIFFNESSES TO "OLD" INPUT FILE
C
      OPEN(2,FILE='LCSTIF.IN')
      WRITE(2,*)((CS(I,M),I=1,2),M=1,NFB)
      CLOSE(2)
      RETURN
C
C      NONLINEAR CONNECTIONS
C
180   F1=1000.
      IED=1
C
C     INPUT CONNECTION PARAMETERS
C
     IF(NFIL.EQ.1)GO TO 182
C
C     READ NONLINEAR CONNECTION PARAMETERS, COMBIN. FROM "OLD" FILE
C
C
C     READ NONLINEAR CONNECTION PARAMETERS, COMBIN. FROM "OLD" FILE
C
      IF (NGEOM.EQ.1) GO TO 309
      OPEN(2,FILE='CONPARG.IN')
      READ(2,*)NCOMB,((ICT(I,M),I=1,2),M=1,NCOMB)
      READ(2,*)(((PG(J,I,M),J=1,5),I=1,2),M=1,NCOMB)
      CLOSE(2)
      GO TO 256
309   OPEN(2,FILE='CONPAR.IN')
      READ(2,*)NCOMB
      DO M=1,NCOMB
        READ(2,*) (ICT(I,M),I=1,2)
      END DO
      DO M=1,NCOMB
        DO I=1,2
          READ(2,*) (P(J,I,M),J=1,4)
        END DO
      END DO
      CLOSE(2)
      GO TO 258
C
C     OR INPUT CONNECTION PARAMETERS AND COMBINATIONS
C
182   WRITE(*,1030)
1030  FORMAT(//10X,'THE CONNECTIONS AT THE ENDS OF A GIVEN BEAM ARE'
     */10X,'ASSOCIATED WITH THAT BEAM.  INPUT THE CONNECTION TYPE'
     * /10X,'AND PARAMETERS FOR EACH CONN.-BEAM-CONN. COMBINATION'
     */10X,'IF CONNECTION GEOMETRIC PARAMETERS ARE INPUT'
```

```
     */10X,'TYPE 1 - DOUBLE WEB ANGLE CONNECTION IS AVAILABLE')
C
C     IED = 1, READ DATA;  2, EDIT DATA
C
      GO TO(176,178)IED
176   WRITE(*,1005)
      CALL RD(NCOMB,RL,55)
      GO TO 179
178   WRITE(*,1005)NCOMB
      WRITE(*,1015)
      CALL ED(NCOMB,55)
      CALL SC
179   DO M=1,NCOMB
       I=1
       WRITE(*,'(/10X,''COMBINATION '',I3,/10X,''CONNECTION TYPE AT "ST
     *ART"'')')M
185    GO TO(181,184)IED
181    CALL RD(ITP,RL,55)
       GO TO 183
184    WRITE(*,'(20X,I5/)')ICT(I,M)
       ITP=ICT(I,M)
       CALL ED(ITP,55)
183    IF(ITP.LE.0.OR.ITP.GT.5)THEN
         WRITE(*,'(10X,''INVALID TYPE, PLEASE RE-ENTER'')')
         CALL BACK(2,55)
         WRITE(*,'(''  '')')
         GO TO 185
       END IF
       ICT(I,M)=ITP
       KK=4
       IF (NGEOM.EQ.2) GO TO 303
       GO TO(202,204,206,208,209)ITP
C
C      THE FIST TYPE CONNECTION :
C
202    WRITE(*,'(//10X,''THE FIRST TYPE CONNECTION  IS PINNED END''/
     * 10X,''INPUT RO=100,AMO=0.1,AN=10,SP=10000''/)')
       GO TO 212
C
C      THE SECOND TYPE OF CONNECTION :
C
204    WRITE(*,'(//10X,''THE SECOND TYPE OF CONNECTIONN IS FIXED END''/
     * 10X,''INPUT RO=1.0E-20,AMO=1.0E+5,AN=10,SP=10000''/)')
       GO TO 212
C
C      THE THIRD TYPE OF CONNECTION :
C
206    WRITE(*,'(//10X,''THE THIRD TYPE OF CONNECTION''/
     * 10X,''REFFERENCE ANGLE,REFFERENCE MOMENT,AN,PLASTIC STIFFNESS
     *''/)')
       GO TO 212
C
```

```
C     THE FORTH TYPE OF CONNECTION :
C
208   WRITE(*,'(//10X,''THE FORTH TYPE OF CONNECTION      ''/
    *  10X,''REFFERENCE ANGLE,REFFERENCE MOMENT,AN,PLASTIC STIFFNESS
    *''/)')
      GO TO 212
C
C     THE FIFTH TYPE OF CONNECTION :
C
209   WRITE(*,'(//10X,''THE FIFTH TYPE OF CONNECTION''/
    *  10X,''REFFERENCE ANGLE,REFFERENCE MOMENT,AN,PLASTIC STIFFNESS
    *''/)')
212   GO TO (214,213) IED
213   WRITE(*,'(20X,4F10.2///)')(P(J,I,M),J=1,KK)
      DO J=1,KK
        PP(J)=P(J,I,M)
      END DO
      CALL EDPAR(KK,PP)
      GO TO 216
214   CALL RDPAR(KK,PP)
216   DO J=1,4
        P(J,I,M)=PP(J)
      END DO
      IF(I.EQ.2)GO TO 250
      I=3-I
      CALL CLSC
      WRITE(*,'(//10X,''CONNECTION AT MEMBER "END" SAME AS AT "START"?
    *''/10X,''YES or NO'')')
      CALL YN(IY)
      IF(IY.EQ.2)GO TO 260
      DO J=1,4
        P(J,I,M)=PP(J)
      END DO
      ICT(2,M)=ICT(1,M)
      GO TO 250
303   JJ=5
C
C     DOUBLE WEB CONNECTION:
C
302   WRITE(*,'(//10X,''DOUBLE WEB ANGLE CONNECTION''/
    *  10X,''ANGLE THICK, GAGE OF COL., ANGLE LENGTH,BEAM DEPTH,NUMBER
    *  OF BOLTS''/)')
C
312   GO TO (314,313) IED
313   WRITE(*,'(20X,4F10.2///)')(PG(J,I,M),J=1,JJ)
      DO J=1,JJ
        PP(J)=PG(J,I,M)
      END DO
      CALL EDPAR(JJ,PP)
      GO TO 216
314   CALL RDPAR(JJ,PP)
316   DO J=1,5
```

```
          PG(J,I,M)=PP(J)
        END DO
        IF(I.EQ.2)GO TO 250
        I=3-I
        CALL CLSC
        WRITE(*,'(//10X,''CONNECTION AT MEMBER "END" SAME AS AT "START"?
     *''/10X,''YES or NO'')')
        CALL YN(IY)
        IF(IY.EQ.2)GO TO 260
        DO J=1,5
          PG(J,I,M)=PP(J)
        END DO
        ICT(2,M)=ICT(1,M)
        GO TO 250
260     WRITE(*,'(/10X,''COMBINATION '',I3,/10X,''CONNECTION TYPE AT "EN
     *D"''/)')M
        GO TO 185
250     CONTINUE
      END DO
258   WRITE(*,'(//10X,''EDIT CONNECTION TYPES AND PARAMETERS?''/10X,''YE
     *S or NO'')')
      CALL YN(IY)
      IF(IY.EQ.2)GO TO 256
      CALL SC
      IED=2
      GO TO 182
C
C     READ CONN - BEAM - CONN COMBINATIONS FROM "OLD" FILE
C
256   GO TO(264,262)NFIL
262   OPEN(2,FILE='CONCOMB.IN')
      READ(2,*)(ICTYP(M),M=1,NFB)
      CLOSE(2)
      GO TO 266
C
C      OR INPUT CONNECTION COMBINATIONS FOR ALL BEAMS
C
264   WRITE(*,1000)
      CALL RTYPE(NFB,ICTYP,NCOMB)
C
C     EDIT CONNECTION COMBINATIONS FOR ALL BEAMS
C
266   DO M=1,NFB
        K=ICTYP(M)
        I=ICT(1,K)
        J=ICT(2,K)
        IBCT(1,M)=I
        IBCT(2,M)=J
      END DO
      WRITE(*,'(//10X,''EDIT CONN. - BEAM - CONN. COMBINATIONS''/10X,''Y
     *ES or NO'')')
      CALL YN(IY)
```

```
      IF(IY.EQ.2)GO TO 268
      CALL SC
      WRITE(*,'(//10X,''CONN. - BEAM - CONN. COMBINATIONS''/10X,I5'' COM
     *BINATIONS'')')NFB
      CALL ETYPE(NFB,ICTYP,NCOMB)
C
C     WRITE CONN - BEAM - CONN COMBINATIONS TO "OLD" FILE
C
268   OPEN(2,FILE='CONCOMB.IN')
      WRITE(2,*)(ICTYP(M),M=1,NFB)
      CLOSE(2)
C
C     COMPUTE MOMENT-ROTATION FUNCTIONS FOR NONLINEAR CONNECTIONS
C
      DO M=1,NFB
        K=ICTYP(M)
        DO I=1,2
         IF (NGEOM.EQ.2) GO TO 224
         GO TO(190,200,210,220,230)IBCT(I,M)
190      RO(I,M)=P(1,I,1)
         AMO(I,M)=P(2,I,1)
         AN(I,M)=P(3,I,1)
         SP(I,M)=P(4,I,1)
         GO TO 240
200      RO(I,M)=P(1,I,2)
         AMO(I,M)=P(2,I,2)
         AN(I,M)=P(3,I,2)
         SP(I,M)=P(4,I,2)
         GO TO 240
210      RO(I,M)=P(1,I,3)
         AMO(I,M)=P(2,I,3)
         AN(I,M)=P(3,I,3)
         SP(I,M)=P(4,I,3)
         GO TO 240
220      RO(I,M)=P(1,I,4)
         AMO(I,M)=P(2,I,4)
         AN(I,M)=P(3,I,4)
         SP(I,M)=P(4,I,4)
         GO TO 240
230      RO(I,M)=P(1,I,5)
         AMO(I,M)=P(2,I,5)
         AN(I,M)=P(3,I,5)
         SP(I,M)=P(4,I,5)
         GO TO 240
224      RO(I,M) =PG(1,I,1)**0.595*PG(2,I,1)**(-2.817)*PG(3,I,1)**
     *          4.737*PG(4,I,1)**(-0.784)*PG(5,I,1)**(-5.957)*0.001
         AMO(I,M)=PG(1,I,1)**1.136/(PG(2,I,1)**1.515)*PG(3,I,1)**1.139
     *          *PG(4,I,1)**0.258*PG(5,I,1)**0.309
         AN(I,M) =PG(1,I,1)**0.522*PG(2,I,1)**1.564/(PG(3,I,1)**1.073)/
     *          (PG(4,I,1)**0.737)*PG(5,I,1)**1.704
         SP(I,M) =PG(1,I,1)**0.955*PG(2,I,1)**2.044/(PG(3,I,1)**4.445)*
     *          PG(4,I,1)**0.327*PG(5,I,1)**7.555*1000.
```

```
240      CONTINUE
        CST(I,K)=(AMO(I,M)/RO(I,M))*F1
      END DO
    END DO
C
C   SET NONLINEAR CONNECTION FUNCTIONS FOR ALL BEAMS
C
270  DO M=1,NFB
      J=ICTYP(M)
      DO I=1,2
        CS(I,M)=CST(I,J)
      END DO
    END DO
C
C   WRITE NONLINEAR CONNECTION TYPES AND PARAMETERS TO "OLD" FILE
C
280  OPEN(2,FILE='CONPAR.IN')
    WRITE(2,*)NCOMB
    DO M=1,NCOMB
      WRITE(2,*)(ICT(I,M),I=1,2)
    END DO
    DO M=1,NCOMB
      DO I=1,2
        IF (NGEOM.EQ.2) THEN
          WRITE (2,*)(PG(J,I,M),J=1,5)
          GO TO 305
        END IF
        WRITE(2,*) (P(J,I,M),J=1,4)
305      CONTINUE
      END DO
    END DO
    CLOSE(2)
    CALL FF
    IF(NTA.GT.3)GO TO 410
    WRITE(3,'(/10X,''STRUCTURE HAS CONSTANT-STIFFNESS CONNECTIONS'')')
    WRITE(3,'(//10X,''LINEAR STIFFNESSES FOR THE TWO CONNECTIONS AT TH
   *E''/10X,''ENDS OF A GIVEN BEAM ARE ASSOCIATED WITH THAT BEAM.''/10
   *X,''STIFFNESSES FOR EACH CONN.-BEAM-CONN. COMBINATION''/10X,''BEAM
   *              CONNECTION STIFFNESSES'')')
    DO M=1,NFB
      WRITE(3,1010)M,CS(1,M),CS(2,M)
    END DO
    RETURN
410  WRITE(3,'(//10X,''STRUCTURE HAS NONLINEAR CONNECTIONS'')')
    WRITE(3,'(//10X,''THE CONNECTIONS AT THE ENDS OF A GIVEN BEAM ARE'
   *'/10X,''ASSOCIATED WITH THAT BEAM.  THE CONNECTION TYPES AND''
   * /10X,''PARAMETERS FOR EACH CONN.-BEAM-CONN. COMBINATION FOLLOW''
   * /10X,''THE CONNECTION TYPES ARE AVAILABLE ARE:''
   */15X, ''TYPE 1  -  THE FIRST TYPE CONNECTION''
   */15X, ''TYPE 2  -  THE SECOND TYPE CONNECTION''
   */15X, ''TYPE 3  -  THE THIRD TYPE OF CONNECTION''
   */15X, ''TYPE 4  -  THE FORTH TYPE OF CONNECTION''
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     */15X, ''TYPE 5  -  THE FIFTH TYPE OF CONNECTION'')')
      WRITE(3,1005)NCOMB
      WRITE(3,'(/10X,''COMBINATION  START CONN.   END CONN.''/)')
      DO M=1,NCOMB
        WRITE(3,'(5X,3I12)')M,(ICT(I,M),I=1,2)
      END DO
      WRITE(3,'(///10X,''CONNECTION PARAMETERS AT START / END OF MEMBER'
     *')')
      DO M=1,NCOMB
        WRITE(3,'(//10X,''COMBINATION'',(I10))')M
        WRITE(3,'(//10X,''REFFERENCE ANGLE; REFFERENCE MOMENT; AN;PLASTI
     *C STIFFNESS DEP'')')
      END DO
      DO M=1,NCOMB
        WRITE(3,7460) (P(J,1,M),J=1,4)
7460   FORMAT(10X,'START',4X,F8.5,5X,F10.0,1X,F7.1,5X,F7.1)
        WRITE(3,7470) (P(J,2,M),J=1,4)
7470   FORMAT(10X,'END',6X,F8.5,5X,F10.0,1X,F7.1,5X,F7.1)
      END DO
      CALL FF
      WRITE(3,1000)
      WRITE(3,'(10X,(20I3))')(ICTYP(M),M=1,NFB)
      RETURN
C
      ENTRY ITER(JL,INB,IBCT,CS,BFA,BFB,NCOUNT,AMO,RO,AN,SP,BENDF,
     *      ROTAT,BENDFAB,ROTATAB,NTYPE,ROTAT3,BENDF3,NLOADTM,
     *      ICT,NCOMB,SIGN0,RR,NITER)
C
      NCOUNT=0
C
      IF (NLOADTM.EQ.1) GO TO 1111
C
C     MODIFY FUNCTOINE 2 OR 4 IF NECCESSARY
C
90    DO 128 M=1,NFB
      DO 138 I=1,2
        AM1=BENDF(I,M)
        AM=BFA(3,M)
        IF(I.EQ.2) AM=BFB(3,M)
        AAM=BENDFAB(I,M)+AM
        NTY=NTYPE(I,M)
        II=IBCT(I,M)
        IF(II.EQ.1.OR.II.EQ.2) GO TO 138
        GO TO (136,137,134,135,138),NTY
136     IF (ABS(BENDF(I,M)).GT.ABS(AAM)) NTYPE(I,M)=2
        GO TO 138
137     IF(BENDF(I,M).LT.0) GO TO 937
        IF(AAM.GT.0.0.AND.AAM.LT.BENDF(I,M)) GO TO 138
        IF(AAM.LT.0.0) NTYPE(I,M)=3
        IF(AAM.GT.BENDF(I,M)) NTYPE(I,M)=1
        GO TO 138
134     IF (ABS(BENDF3(I,M)).GT.ABS(AAM)) NTYPE(I,M)=4
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      GO TO 138
937    IF(AAM.LT.0.0.AND.AAM.GT.BENDF(I,M)) GO TO 138
      IF(AAM.GT.0.0) NTYPE(I,M)=3
      IF(AAM.LT.BENDF(I,M)) NTYPE(I,M)=1
      GO TO 138
135    IF (BENDF3(I,M).GT.0.0) GO TO 737
      IF (AAM.LT.BENDF3(I,M)) NTYPE(I,M)=3
      IF (AAM.GT.0.0) NTYPE(I,M)=5
      GO TO 138
737    IF (AAM.GT.BENDF3(I,M)) NTYPE(I,M)=3
      IF (AAM.LT.0.0) NTYPE(I,M)=5
138  CONTINUE
128  CONTINUE
1111 CONTINUE
C
     DO M=1,NFB
      DO I=1,2
       NTY=NTYPE(I,M)
       IC=IBCT(I,M)
       AM=BFA(3,M)
       IF (I.EQ.2)AM=BFB(3,M)
       AM1=BENDF(I,M)
C
C - - SELECT SUITABLE FUNCTION ACCORDING TO ARRAY NTYPE
C
310  GO TO (11,21,31,41,51),NTY
C
C      FUNCTION FOR TYPE 1
C
11     AMS=AM+BENDFAB(I,M)
       RC=ROTATAB(I,M)
       SCO=AMO(I,M)/RO(I,M)*F1
       CALL ROTATIN (AMO,RO,AN,SP,BENDF,ROTAT,ROTAT3,BENDF3,I,M,AMS)
       RR(I,M)=R
       RP=(AM/CS(I,M))*F1
       IF(RR(I,M).EQ.0) THEN
         CS(I,M)=SCO
       ELSE
         CS(I,M)=DABS((2*AM/(R-RC+RP))*F1)
       END IF
       R=R-RC
       GO TO 50
C
C      FUNCTION FOR TYPE 2
C
21     R=AM/CS(I,M)*F1
       RP=R
       RR(I,M)=R
       CS(I,M)=SCO
       GO TO 50
C
C      FUNCTION FOR TYPE 3
```

```
C
31        AMS=AM+BENDFAB(I,M)
          SCO=AMO(I,M)/RO(I,M)*F1
          CALL ROTATIN (AMO,RO, AN,SP,BENDF,ROTAT,ROTAT3,BENDF3,I,M,AMS)
          RR(I,M)=R+(ROTAT(I,M)-BENDF(I,M)/SCO*F1)
          DELTAR=ROTAT(I,M)-BENDF(I,M)/SCO*F1
          RT=ROTATAB(I,M)-DELTAR
          AMM=AMS-BENDFAB(I,M)
          RP=(AMM/CS(I,M))*F1
          IF((R+RP).EQ.0) THEN
            CS(I,M)=SCO
          ELSE
            CS(I,M)=DABS((2.*AMM/(R-RT+RP))*F1)
          END IF
          R=R-RC
          GO TO 50
C
C         FUNCTION FOR TYPE 4
C
41        R=AM/(CS(I,M)/F1)
          RP=R
          RR(I,M)=R
          CS(I,M)=SCO
          GO TO 50
C
C         FUNCTION FOR TYPE 5
C
51        AMS=AM+BENDF(I,M)
          RC=ROTAT(I,M)
          CALL ROTATIN (AMO,RO,AN,SP,BENDF,ROTAT,ROTAT3,BENDF3,I,M,AMS)
          RR(I,M)=R
          RP=(AM/CS(I,M))*F1
          IF((R+RP).EQ.0) THEN
            CS(I,M)=SCO
          ELSE
            CS(I,M)=DABS((2.*AMS/(R-RC+RP))*F1)
          END IF
          R=R-RC
50        CONTINUE
C
340       IF(R.EQ.0) GO TO 350
          DR=DABS((R-RP)/R)
          IF(DR.LT.0.008.AND.NITER.GT.2) GO TO 350
888       NCOUNT=NCOUNT+1
350     CONTINUE
       END DO
     END DO
     DO M=1,NFB
       PRINT *,M,INB(1,M),NTYPE(1,M),INB(2,M),NTYPE(2,M)
     END DO
     CLOSE(2)
     RETURN
```

```
      END
C
C     THIS SUBROUTINE CALCULATE ROTATION ACCODING MOMENT
C
      SUBROUTINE ROTATIN(AMO,RO,AN,SP,BENDF,ROTAT,ROTAT3,BENDF3,I,M,AMS)
      IMPLICIT REAL*8 (A-H,O-Z)
      DIMENSION AMO(2,NFB),RO(2,NFB),AN(2,NFB),SP(2,NFB),BENDF(2,NFB),
     *          ROTAT(2,NFB),ROTAT3(2,NFB),BENDF3(2,NFB)
      COMMON /COMR/ R
      COMMON /GEN/NTJ,NFB,NFC,NBAND,NEQ,NXM
      COMMON /TEMPB1/NPRT,NFIL,IPDELT
      GUS(R)=SCO*R*((1.-SP(I,M)/SCO)/
     *      ((1.+DABS((1./RO(I,M)-SP(I,M)/AMO(I,M))*R)**AN(I,M))
     *      **(1./AN(I,M)))+SP(I,M)/SCO)-AMS
      SCO=AMO(I,M)/RO(I,M)
      F1=1000.
      R00=-5.00
      R01=5.00
40    EPS=1.0E-5
      FR00=GUS(R00)
      FR01=GUS(R01)
      IF(FR00*FR01.LT.0.D0) THEN
55      R02=(R00+R01)/2.0
        FR02=GUS(R02)
        IF(FR02*FR00.LT.0.D0) THEN
          FR01=FR02
          R01=R02
        ELSE
          FR00=FR02
          R00=R02
        END IF
          IF(ABS(FR02).LT.EPS) GO TO 104
          GO TO 55
104       CONTINUE
      ELSE
        WRITE(*,101)
101     FORMAT(2X,'NOTHING')
      END IF
      R=R02
      RETURN
      END
C
C     THIS SUBROUTINE READS NONLINEAR CONNECTION PARAMETERS
C
      SUBROUTINE RDPAR(KK,PP)
      IMPLICIT REAL*8 (A-H,O-Z)
      DIMENSION PP(5)
      J=15
      DO K=1,KK
       J=J+12
       CALL RD(INT,PP(K),J)
      END DO
```

```
      RETURN
      END
C
C     THIS SUBROUTINE EDITS NONLINEAR CONNECTION PARAMETERS
C
      SUBROUTINE EDPAR(KK,PP)
      IMPLICIT REAL*8 (A-H,O-Z)
      DIMENSION PP(5)
      J=15
      DO K=1,KK
        J=J+12
        CALL RD(INT,RL,J)
        IF(RL.NE.0.)PP(K)=RL
      END DO
      RETURN
      END
C
C     THIS SUBROUTINE PERFORM ARRAY NTYPE ETC.
C
      SUBROUTINE FUNCTY(INB,BENDF,ROTAT,BENDFAB,ROTATAB,NTYPE,ROTAT3,
     *                  BENDF3)
      IMPLICIT REAL*8(A-H,O-Z)
      COMMON/GEN/NTJ,NFB,NFC,NBAND,NEQ,NXM
      COMMON /COM1/ NLOAD
      DIMENSION BENDF(2,NFB),ROTAT(2,NFB),NTYPE(2,NFB),ROTAT3(2,NFB),
     *       BENDF3(2,NFB),INB(2,NFB),BENDFAB(2,NFB),ROTATAB(2,NFB)
      DO 71 M=1,NFB
      DO 81 I=1,2
        NTYPE(I,M)=1
        BENDF(I,M)=0.
        BENDFAB(I,M)=0.
        ROTAT(I,M)=0.
        ROTATAB(I,M)=0.
        ROTAT3(I,M)=0.
        BENDF3(I,M)=0.
81    CONTINUE
71    CONTINUE
      RETURN
      END
C$INCLUDE SOLVER.FOR
```

```
C
C  THIS SUBROUTINE PERFORMS AN ASSEMBLY OF THE COLUMN
C  STIFFNESS COEFFICIENTS FOR EACH INDIVIDUAL COLUMN.
C
      SUBROUTINE  COL( CN, INC, ICP, SS ,PP, NITER, NLOADTM)
      IMPLICIT REAL*8 (A-H,O-Z)
      COMMON /MAT/ EM, GM
      COMMON /SECT/ BP(4,20), CP(4,20)
      COMMON  /GEN/ NTJ, NFB, NFC, NBAND, NEQ,NXM
      COMMON /IPT/ NST, NBAY1, NBAY3
      INTEGER V, M, U, L
      DIMENSION  CN(3,NTJ), INC(2,NFC), SS(NXM), ICP(NFC), PP(NFC)
C
C
      KK(L1,M1,L3,M3) = (3*L1 - 3*M1 + L3 - M3)*NEQ + 3*M1 + M3 - 3
      DO  100 I = 1,  NXM
      SS(I) = 0.D0
100   CONTINUE
      MM = 1
      MN = NFC/NST
      DO  300 N = 1, NST
      M = (N + 1)*NTJ/(NST + 1)
      V = N*NTJ/(NST + 1)
      DO  310  J = MM, MN
      CALL  CELM( J, CN,ICP,INC, A, B, C, D, E, F, G, H, P, Q,
     1 U, L, V, M, ZU, XU, ZL, XL, DL)
      IF (NITER.EQ.1.AND.NLOADTM.EQ.1) PP(J)=0.D0
      PL=PP(J)/DL
      Q=Q+DL*DL*PL/30
      C=C-DL*DL*PL/7.5
      D=D-DL*PL/10
      F=F-1.2*PL
      K = KK(U,U,1,1)
      SS(K) = SS(K) + A
      K = KK(U,U,2,2)
      SS(K) = SS(K) + B
      K = KK(U,U,3,3)
      SS(K) = SS(K) + C
      K = KK(V,U,1,3)
      SS(K) = SS(K) + D
      K = KK(V,U,2,2)
      SS(K) = SS(K) - E
      K = KK(V,U,3,2)
      SS(K) = SS(K) + XU*E
      K = KK(V,U,3,3)
      SS(K) = SS(K) + ZU*D
      K = KK(V,V,1,1)
      SS(K) = SS(K) + F
      K = KK(V,V,2,2)
      SS(K) = SS(K) + G
      K = KK(V,V,3,1)
      SS(K) = SS(K) + ZU*F
```

```
K = KK(V,V,3,2)
SS(K) = SS(K) - XU*G
K = KK(V,V,3,3)
SS(K) = SS(K) + (ZU**2)*F + (XU**2)*G + H
K = KK(L,U,1,1)
SS(K) = SS(K) - A
K = KK(L,U,2,2)
SS(K) = SS(K) + P
K = KK(L,U,3,3)
SS(K) = SS(K) + Q
K = KK(L,V,2,2)
SS(K) = SS(K) - E
K = KK(L,V,2,3)
SS(K) = SS(K) + E*XL
K = KK(L,V,3,1)
SS(K) = SS(K) + D
K = KK(L,V,3,3)
SS(K) = SS(K) + D*ZL
K = KK(L,L,1,1)
SS(K) = SS(K) + A
K= KK(L,L,2,2)
SS(K) = SS(K) + B
K = KK(L,L,3,3)
SS(K) = SS(K) + C
K = KK(M,U,1,3)
SS(K) = SS(K) - D
K = KK(M,U,2,2)
SS(K) = SS(K) + E
K = KK(M,U,3,2)
SS(K) = SS(K) - XL*E
K = KK(M,U,3,3)
SS(K) = SS(K) - ZL*D
K = KK(M,V,1,1)
SS(K) = SS(K) - F
K = KK(M,V,1,3)
SS(K) = SS(K) - ZL*F
K = KK(M,V,2,2)
SS(K) = SS(K) - G
K = KK(M,V,2,3)
SS(K) = SS(K) + XL*G
K = KK(M,V,3,1)
SS(K) = SS(K) - ZL*F
K = KK(M,V,3,2)
SS(K) = SS(K) + XL*G
K= KK(M,V,3,3)
SS(K) = SS(K) - ZL**2*F -XL**2*G - H
K= KK(M,L,1,3)
SS(K) = SS(K) - D
K = KK(M,L,2,2)
SS(K) = SS(K) + E
K = KK(M,L,3,2)
SS(K) = SS(K) - XL*E
```

```
      K = KK(M,L,3,3)
      SS(K) = SS(K) - ZL*D
      K = KK(M,M,1,1)
      SS(K) = SS(K) + F
      K = KK(M,M,2,2)
      SS(K) = SS(K) + G
      K = KK(M,M,3,1)
      SS(K) = SS(K) + ZL*F
      K = KK(M,M,3,2)
      SS(K) = SS(K) - XL*G
      K = KK (M,M,3,3)
      SS(K) = SS(K) + (ZL**2)*F + (XL**2)*G + H
  310 CONTINUE
      MM = MN + 1
      MN = (N+1)*NFC/NST
  300 CONTINUE
      RETURN
      END
C
C
C
C     THIS SUBROUTINE COMPUTES THE STIFFNESS COEFFICIENTS
C     FOR EACH COLUMN MEMBER
C
C
      SUBROUTINE  CELM(J,CN,ICP,INC,A,B,C,D,E,F,G,H,
     1P,Q,U,L,V,M,ZU,XU,ZL,XL,DL)
      IMPLICIT REAL*8 (A-H,O-Z)
      COMMON /GEN/ NTJ, NFB, NFC, NBAND, NEQ, NXM
      COMMON /MAT/ EM, GM
      COMMON /SECT/ BP(4,20), CP(4,20)
      DIMENSION  CN(3,NTJ), INC(2,NFC), ICP(NFC)
      INTEGER  V, M, U, L
      REAL*8  I1, I2, I3
C
C
      MP = ICP(J)
      I1 = CP(1,MP)
      I2 = CP(2,MP)
      I3 = CP(3,MP)
      AA = CP(4,MP)
      L = INC(1,J)
      U = INC(2,J)
      X1 = CN(1,L)
      Y1 = CN(2,L)
      Z1 = CN(3,L)
      X2 = CN(1,U)
      Y2 = CN(2,U)
      Z2 = CN(3,U)
      X3 = CN(1,M)
      Y3 = CN(2,M)
      Z3 = CN(3,M)
```

```
      X4 = CN(1,V)
      Y4 = CN(2,V)
      Z4 = CN(3,V)
      DL = DABS(Y2 - Y1)
20    ZU = Z2 - Z4
      XU = X2 - X4
      ZL = Z1 - Z3
      XL = X1 - X3
      P = 2.0*EM*I2/DL
      B = 2.0*P
      E = 1.5*B/DL
      G = 2.0*E/DL
      Q = 2.0*EM*I3/DL
      C = 2.0*Q
      D = 1.5*C/DL
      F = 2.0*D/DL
      H = GM*I1/DL
      A = AA*EM/DL
      RETURN
      END
C
C
C
C     THIS SUBROUTINE PERFORMS AN ASSEMBLY OF THE
C     STIFFNESS COEFFICIENTS FOR EACH INDIVIDUAL BEAM
C     MEMBER INTO THE STRUCTURE STIFFNESS MATRIX.
C
C
      SUBROUTINE  BEAM( CN, INB, IBP, SC, SS )
      IMPLICIT REAL*8 (A-H,O-Z)
      COMMON  /GEN/ NTJ, NFB, NFC, NBAND, NEQ,NXM
      COMMON  /IPT/NST, NBAY1, NBAY3
      COMMON /MAT/ EM, GM
      COMMON /SECT/ BP(4,20), CP(4,20)
      DIMENSION  CN(3,NTJ), INB(2,NFB), SS(NXM), IBP(NFB), SC(2,NFB)
C
C
      KK(L1,M1,L3,M3) = (3*L1 - 3*M1 + L3 - M3)*NEQ + 3*M1+M3-3
      MM = 1
      MN = NFB/NST
      DO   100  N = 1, NST
      DO   200  M = MM, MN
C
C
C     SA, SB ARE THE CONNECTION STIFFNESS AT END A AND B
C     OF BEAM. IF THE CONNECTION IS ASSUMED TO BE RIGID,
C     SA AND SB IS SET TO A VERY LARGE NUMBER IN SUBROUTINE CINFO.
C
      SA=SC(1,M)
      SB=SC(2,M)
      CALL   BELM( M, CN,INB, IBP, SA, SB, S1, S2, S3, S4, S5, S22,
     1 S44, R11, R22, R23,R32, R33, I, J )
```

```
      K = KK(J,J,1,1)
      SS(K) = SS(K) + R11*S1*R11
      K = KK(J,J,2,1)
      SS(K) = SS(K) + R11*(-S2)*R23
      K = KK(J,J,2,2)
      SS(K) = SS(K) + R22*S3*R22 + R23*S4*R23
      K = KK(J,J,3,1)
      SS(K) = SS(K) + R33*(-S2)*R11
      K = KK(J,J,3,2)
      SS(K) = SS(K) + R32*S3*R22 + R33*S4*R23
      K = KK(J,J,3,3)
      SS(K) = SS(K) + R32*S3*R32 + R33*S4*R33
      K = KK(J,I,1,1)
      SS(K) = SS(K) + R11*(-S1)*R11
      K = KK(J,I,1,2)
      SS(K) = SS(K) + R11*(-S22)*R23
      K= KK(J,I,1,3)
      SS(K) = SS(K) + R11*(-S22)*R33
      K = KK(J,I,2,1)
      SS(K) = SS(K) +   R23 *S2*R11
      K = KK(J,I,2,2)
      SS(K) = SS(K) + R22*(-S3)*R22 + R23*S5*R23
      K = KK(J,I,2,3)
      SS(K) = SS(K) + R22*(-S3)*R32 + R23*S5*R33
      K = KK(J,I,3,1)
      SS(K) = SS(K) +   R33 *S2*R11
      K = KK(J,I,3,2)
      SS(K) = SS(K) + R32*(-S3)*R22 + R33*S5*R23
      K = KK(J,I,3,3)
      SS(K) = SS(K) + R32*(-S3)*R32 + R33*S5*R33
      K = KK(I,I,1,1)
      SS(K) = SS(K) + R11*S1*R11
      K = KK(I,I,2,1)
      SS(K) = SS(K) + R23*S22*R11
      K = KK(I,I,2,2)
      SS(K) = SS(K) + R22*S3*R22 + R23*S44*R23
      K = KK(I,I,3,1)
      SS(K) = SS(K) + R33*S22*R11
      K = KK(I,I,3,2)
      SS(K) = SS(K) + R32*S3*R22 + R33*S44*R23
      K = KK(I,I,3,3)
      SS(K) = SS(K) + R32*S3*R32 + R33*S44*R33
 200  CONTINUE
      MM = MN + 1
      MN = (N + 1)*NFB/NST
 100  CONTINUE
      RETURN
      END
C
C
C
C     THIS SUBROUTINE COMPUTES THE STIFFNESS COEFFICIENTS
```

```
C   FOR EACH BEAM MEMBER.
C
C
      SUBROUTINE   BELM( M, CN,INB, IBP, SA, SB, S1, S2, S3, S4, S5,
     1S22, S44, R11, R22, R23, R32, R33, I, J)
      IMPLICIT REAL*8 (A-H,O-Z)
      COMMON  /GEN/ NTJ, NFB, NFC, NBAND, NEQ, NXM
      COMMON /MAT/EM, GM
      COMMON /SECT/ BP(4,20), CP(4,20)
      DIMENSION CN(3,NTJ), INB(2,NFB), IBP(NFB)
      REAL*8 I1, I2, I3
C
C
      MP = IBP(M)
      I1 = BP(1,MP)
      I2 = BP(2,MP)
      I3 = BP(3,MP)
      AA=  BP(4,MP)
      I = INB(1,M)
      J = INB(2,M)
      X1 = CN(1,I)
      Y1 = CN(2,I)
      Z1 = CN(3,I)
      X2 = CN(1,J)
      Y2 = CN(2,J)
      Z2 = CN(3,J)
      R11 =1.
      R22 = 1.0
      R23 = 0.D0
      R32 = 0.D0
      R33 = 1.
      IF  ( Z1 .EQ. Z2 )  GOTO  10
      DL = DABS(Z1 - Z2)
      R22 = 0.
      R23 = -1.
      R32 = 1.
      R33 = 0.D0
      GOTO  20
10    DL = DABS(X1 - X2)
20    CONTINUE
      COM=DL**4/(12.*EM**2*I3**2)+DL**3/(3.*EM*I3*SA)
     á+ DL**3/(3.*EM*I3*SB) + DL**2/(SA*SB)
      S5=(DL**3/(6.*EM*I3))/COM
      S2=(DL**2/(2.*EM*I3)+DL/SA)/COM
      S4=(DL**3/(3.*EM*I3)+DL**2/SA)/COM
      S22=(DL**2/(2.*EM*I3)+DL/SB)/COM
      S3=GM*I1/DL
      S44=(DL**3/(3.*EM*I3)+DL**2/SB)/COM
      S1=(DL/(EM*I3)+1./SA+1./SB)/COM
      RETURN
      END
C
```

```
C   THIS SUBROUTINE SETS THE SUPPORT CONDITION BY ASSUMING
C   THAT THE BUILDING IS RIGIDLY CONNECTED TO THE FOUNDATION
C
    SUBROUTINE   SPT( SS, JL ,NLOADTM,NITER)
    IMPLICIT REAL*8 (A-H,O-Z)
    COMMON  /GEN/NTJ,NFB,NFC,NBAND,NEQ,NXM
    COMMON  /IPT/NST, NBAY1, NBAY3
    COMMON  /TEMPB1/NPRT,NFIL,IPDELT
    DIMENSION  SS(NXM),  JL(3,NTJ),ISR(20)
    REAL*8 JL
    KK(L1,M1,L3,M3) = (3*L1 - 3*M1 + L3 - M3)*NEQ + 3*M1 + M3 -3
C
C   IF USING OLD INPUT FILE,READ FROM ''SUPPORT.IN''
C     IF ISR(I)=0, MEAN FIXED END
C     IF ISR(I)=1, MEAN PINNED ABOUT X AXIS
C     IF ISR(I)=2, MEAN PINNED ABOUT Z AXIS
C     IF ISR(I)=3, MEAN PINNED ABOUT X AND Z AXIS
C
    NS=NTJ-NTJ/(NST+1)*NST-1
C
    IF (NFIL.EQ.1) GO TO 50
C
    OPEN(2,FILE='SUPPORT.IN')
    READ(2,*) (ISR(I),I=1,NS)
    CLOSE(UNIT=2)
    GO TO 90
C
C   OTHERWISE INPUT THE INFORMATION
C
50  CALL CLSC
      WRITE(*,'(//10X,''INPUT SUPPORT CONDITION''//
   *            10X,''INPUT 0 FOR FIXED END''/
   *            10X,''INPUT 1 FOR PINNED IN X AXIS''/
   *            10X,''INPUT 2 FOR PINNED IN Z AXIS''/
   *          10X,''INPUT 3 FOR PINNED IN BOTH OF X AND Z AXIS'')')
    DO I=1,NS
      ISR(I)=0
    END DO
    DO I=1,NS
      J=NTJ/(NST+1)*NST+I
      WRITE(*,'(//10X,''NUMBER OF THE JOINT :'',I5)') J
      WRITE(*,'(//10X,''IS THE JOINT FIXED ?:'')')
      CALL YN(IY)
      IF(IY.EQ.1) GO TO 150
      WRITE(*,'(10X,''IS THE JOINT FIXED IN X ?:'')')
      CALL YN(IY)
      IF(IY.EQ.1) GO TO 140
      ISR(I)=ISR(I)+1
140   WRITE(*,'(10X,''IS THE JOINT FIXED IN Z ?:'')')
      CALL YN(IY)
      IF(IY.EQ.1) GO TO 150
      ISR(I)=ISR(I)+2
```

```
150 CONTINUE
    END DO
90  OPEN (2,FILE='SUPPORT.IN')
    IF(NLOADTM.EQ.1.AND.NITER.EQ.1) THEN
    WRITE(3,'(//5X,''***** BOUNDARY CONDITION *****''//
   *        10X,''0:FOR FIXED END''/
   *        10X,''1:FOR PINNED IN X AXIS''/
   *        10X,''2:FOR PINNED IN Z AXIS''/
   *        10X,''3:FOR PINNED IN BOTH OF X AND Z AXIS'')')
    WRITE(3,120)
120 FORMAT(/5X,'JOINT NO.',5X,'SUPPORT')
    DO I=1,NS
      J=NTJ/(NST+1)*NST+I
      WRITE(2,*) ISR(I)
      WRITE(3,130) J,ISR(I)
    END DO
130 FORMAT(7X,I3,11X,I2)
    ENDIF
    CLOSE(2)
C
    NS = NTJ/(NST + 1) *NST + 1
    NNS=NTJ-NTJ/(NST+1)*NST-1
    DO 5  N = NS, NTJ
       IF (N.EQ.NTJ) GO TO 500
       L=1
       K = KK(N,N,L,L)
       SS(K) = 1.0D20
       JL(L,N) =JL(L,N)*1.0D20
       ICS=ISR(N-NS+1)
       IF(ICS.EQ.0) GO TO 500
       IF(ICS.EQ.1) THEN
         L=2
         GO TO 60
       END IF
       IF(ICS.EQ.2) THEN
         L=3
         GO TO 60
       END IF
       IF(ICS.EQ.3) THEN
         GO TO 5
       END IF
60  K = KK(N,N,L,L)
    SS(K) = 1.0D20
    JL(L,N) =JL(L,N)*1.0D20
    GO TO 5
500 DO L=1,3
      K = KK(N,N,L,L)
      SS(K) = 1.0D20
      JL(L,N) =JL(L,N)*1.0D20
    END DO
5   CONTINUE
    RETURN
```

```
      END
C
C
C
C     THIS SUBROUTINE SOLVES THE LINEAR FORCE-DISPLACEMENT
C     EQUATIONS BY CHOLESKY-DECOMPOSITION METHOD.
C
C
      SUBROUTINE  EQN (CN, A, JD,DISP,ROT)
      IMPLICIT REAL*8  (A-H,O-Z)
      COMMON  /GEN/ NTJ, NFB, NFC, NBAND, NEQ, NXM
      COMMON /IPT/ NST, NBAY1, NBAY3
      DIMENSION  A(NXM), CN(3,NTJ),DISP(3,NTJ),ROT(3,NTJ)
      REAL*8  JD(NEQ)
C
C
      DO M=1,NTJ
        DO I=1,3
          DISP(I,M)=0
          ROT(I,M)=0
        END DO
      END DO
9     NM=NEQ-1
      MS = NBAND
      JWT = 6
      DO 200  K=1,NM
      PIVOT = A(K)
      MR = MIN0(MS,NEQ-K+1)
      DO 180 L=2,MR
      LK = (L-1)*NEQ+K
      C = A(LK)/PIVOT
      IF(C)   150,180,150
 150  I=0
      J=K+L-1
      DO 160 JL=L,MR
      I=I+1
      IJ=(I-1)*NEQ+J
      JLK=(JL-1)*NEQ+K
 160  A(IJ) = A(IJ)-A(JLK)*C
 180  A(LK) = C
 200  CONTINUE
C
C     ' FORWARD SUBSTITUTION ' (LD)*Y=B TO STORE Y IN JD O
C
      M1 = MS-1
      JD(1) = JD(1)/A(1)
      DO 20 I=2,NEQ
      L=MAX0(1,I-M1)
      SUM =0.
      II=I+1
      I1=I-1
      DO 10 K=L,I1
```

```
      J=II-K
      JK=(J-1)*NEQ+K
      SUM=SUM+A(JK)*JD(K)*A(K)
10    CONTINUE
20    JD(I)=(JD(I)-SUM)/A(I)
C
C     ' BACK SUBSTITUTION '  LTR*X=Y   TO STORE X IN JD O
C
      DO 40 KK=1,NM
      I=NEQ-KK
      I1=MIN0(NEQ,I+M1)
      SUM=0.
      L =I+1
      DO 30 K=L,I1
      J=K-I+1
      JI=(J-1)*NEQ+I
      SUM = SUM+A(JI)*JD(K)
 30   CONTINUE
      JD(I) = JD(I)-SUM
 40   CONTINUE
      WRITE(*,300)
300   FORMAT(//5x,' ******* JOINT DISPLACEMENT '//
     º5x,'JOINT',6X,'TRANS X1',1H',7X,
     &'TRANS X2',1H',7X,'TRANS X3',1H'//)
C
C     TRANSLATE DISPLACEMENT AT MASTER NODE TO
C     FOR PRINTING DISPLACEMENT VALUES
C
C
      NSTT=NST+1
      NODE=1
C
      DO  55  N=1,  NSTT
      MSN=N*NTJ/NSTT
      NMSN=MSN-1
C
      K1=3*MSN-2
      K2=K1+1
      K3=K2+1
      TX=JD(K1)
      TZ=JD(K2)
      RY=JD(K3)
C
      DO  60  J=NODE,  NMSN
      ZU=CN(3,J)-CN(3,MSN)
      XU=CN(1,J)-CN(1,MSN)
      TRANX=TX+ZU*RY
      TRANZ=TZ-XU*RY
      I1=3*J-2
      I2=I1+1
      I3=I2+1
      DISP(1,J)=TRANX
```

```
      DISP(2,J)=JD(I1)
      DISP(3,J)=TRANZ
      WRITE(*,310) J, TRANX,JD(I1),TRANZ
60    CONTINUE
      NODE=MSN+1
55    CONTINUE
310   FORMAT(5x,I3,1X,3F15.2/
     &3F15.5)
      WRITE(*,301)
301   FORMAT(//5X,'***** JOINT ROTATION ****** '//
     &5x,'JOINT ',5X,' ROT-X1 ',1H',7X,' ROT-X2 ',1H',7X,' ROT-X3 ',1H')
      NODE=1
      DO 56 N=1,NSTT
      MSN=N*NTJ/NSTT
      NMSN=MSN-1
C
      K1=3*MSN-2
      K2=K1+1
      K3=K2+1
      RY=JD(K3)
C
      DO 61 J=NODE,NMSN
      I1=3*J-2
      I2=I1+1
      I3=I2+1
      ROT(1,J)=JD(I2)
      ROT(2,J)=RY
      ROT(3,J)=JD(I3)
6030  WRITE(*,311) J,JD(I2),RY,JD(I3)
61    CONTINUE
      NODE=MSN+1
56    CONTINUE
311   FORMAT(5X,I3,1X,3F15.5)
      RETURN
      END
C
C     THIS SUBROUTINE IS FOR ADDING AND OURPUTING RESULTS
C
      SUBROUTINE OUT(INB,INC,JL,BFA,BFB,CFA,CFB,AMO,RO,BENDF,ROTAT,
     *              BENDFAB,ROTATAB,NTYPE,ROTAT3,BENDF3,NLOADTM,DISP,
     *              ROT,RR,NTA)
      IMPLICIT REAL*8(A-H,O-Z)
      COMMON /GEN/NTJ,NFB,NFC,NBAND,NEQ,NXM
      COMMON /IPT/NST,NBAY1,NBAY3
      COMMON /TEMPB1/NPRT,NFIL,IPDELT,NX13
      COMMON /COM1/ NLOAD
      COMMON /COM9/ CFAFC
      DIMENSION INB(2,NFB),ROTAT(2,NFB),RR(2,NFB),
     *         BENDF(2,NFB),ROTATAB(2,NFB),BENDFAB(2,NFB),BFB(3,NFB)
     *         ,NTYPE(2,NFB),ROTAT3(2,NFB),BENDF3(2,NFB),AMO(2,NFB),
     *         RO(2,NFB),BFA(3,NFB),ROT(3,NTJ),DISP(3,NTJ),BFAC(3,300),
     *         BFBC(3,300),CFA(3,NFC),CFB(3,NFC),CFAC(3,300),
```

```
     *       CFBC(3,300),CFAFC(3,300),CFBFC(3,300),INC(2,NFC)
     *      ,DISPC(3,300),DISPFC(3,300),ROTC(3,300),ROTFC(3,300)
     *      ,BFAFC(3,300),BFBFC(3,300)
      REAL*8 JL(3,NTJ)
C
      IF(REAL(NLOADTM)/2.EQ.REAL(NLOADTM/2)) GO TO 100
      CALL CLSC
      IF((NLOADTM+1)/2.LT.NLOAD) GO TO 100
      WRITE(*,'(/10X,''RESULTS OF CURRENT LOAD PRINTTED? YES or NO:'')')
      CALL YN(NY)
100   CONTINUE
      F1=1000.
C
      IF(REAL(NLOADTM)/2.EQ.REAL(NLOADTM/2)) GO TO 333
      OPEN(2,FILE='FROTAT.OUT')
      DO 10 M=1,NFB
        DO 20 I=1,2
          IF(NTA.NE.4) THEN
            RR(I,M)=0.
          END IF
          NTY=NTYPE(I,M)
          GO TO (11,22,33,22,11),NTY
11        ROTAT(I,M)=RR(I,M)
          ROTATAB(I,M)=ROTAT(I,M)
          GO TO 66
22        ROTATAB(I,M)=ROTATAB(I,M)+RR(I,M)
          GO TO 66
33        ROTAT3(I,M)=RR(I,M)
          ROTATAB(I,M)=ROTAT3(I,M)
66        CONTINUE
          WRITE(2,1055) ROTAT(I,M),ROTAT3(I,M)
20      CONTINUE
10    CONTINUE
      CLOSE(2)
1055  FORMAT(2F15.5)
      IF(NLOADTM.GE.3) GO TO 444
      DO K=1,NTJ
        DO I=1,3
          DISPFC(I,K)=DISP(I,K)
          DISPC(I,K)=DISP(I,K)
          ROTC(I,K)=ROT(I,K)
          ROTFC(I,K)=ROT(I,K)
        END DO
      END DO
      DO M=1,NFB
        DO I=1,3
          BFAC(I,M)=BFA(I,M)
          BFAFC(I,M)=BFA(I,M)
          BFBC(I,M)=BFB(I,M)
          BFBFC(I,M)=BFB(I,M)
        END DO
      END DO
```

```
      DO M=1,NFC
        DO I=1,3
          CFAC(I,M)=CFA(I,M)
          CFAFC(I,M)=CFA(I,M)
          CFBC(I,M)=CFB(I,M)
          CFBFC(I,M)=CFB(I,M)
        END DO
      END DO
1085  FORMAT(3F12.2,3F12.5)
      OPEN(2,FILE='FMOMENT.OUT')
      DO 2028 M=1,NFB
        BENDF(1,M)=BFA(3,M)
        BENDF(2,M)=BFB(3,M)
        DO 2029 I=1,2
          BENDFAB(I,M)=BENDF(I,M)
          NTYPE(I,M)=1
          BENDF3(I,M)=0.
          WRITE(2,2030) BENDF(I,M),BENDFAB(I,M),BENDF3(I,M),
     *                  NTYPE(I,M)
2029    CONTINUE
2028  CONTINUE
      CLOSE(2)
      GO TO 1099
2030  FORMAT(3F15.2,3I5)
2031  FORMAT(2F15.2)
C
333   OPEN(2,FILE='BFAB2.OUT')
      DO M=1,NFB
        WRITE(2,2031) BFA(3,M),BFB(3,M)
      END DO
      CLOSE(2)
      GO TO 77
C
444   DO M=NTJ*((NLOADTM-1)/2)+1,NTJ*((NLOADTM+1)/2)
        DO I=1,3
          DISPC(I,M)=DISP(I,M-NTJ*(NLOADTM-1)/2)
          DISPFC(I,M)=DISPFC(I,M-NTJ)+
     *                DISP(I,M-NTJ*(NLOADTM-1)/2)
          ROTC(I,M)=ROT(I,M-NTJ*(NLOADTM-1)/2)
          ROTFC(I,M)=ROTFC(I,M-NTJ)+
     *              ROT(I,M-NTJ*(NLOADTM-1)/2)
        END DO
      END DO
      OPEN(2,FILE='FMOMENT.OUT')
      DO 1088 M=1,NFB
      DO 1089 I=1,2
        NTY=NTYPE(I,M)
        GO TO (1031,1032,1033,1032,1032),NTY
1031    IF(I.EQ.1) THEN
          BENDF(I,M)=BENDF(I,M)+BFA(3,M)
        ELSE
          BENDF(I,M)=BENDF(I,M)+BFB(3,M)
```

```
      END IF
      BENDFAB(I,M)=BENDF(I,M)
      GO TO 3089
1032  IF(I.EQ.1) THEN
        BENDFAB(I,M)=BENDFAB(I,M)+BFA(3,M)
      ELSE
        BENDFAB(I,M)=BENDFAB(I,M)+BFB(3,M)
      END IF
      GO TO 3089
1033  IF(I.EQ.1) THEN
       BENDF3(I,M)=BENDF3(I,M)+BFA(3,M)
      ELSE
       BENDF3(I,M)=BENDF3(I,M)+BFB(3,M)
      END IF
      BENDFAB(I,M)=BENDF3(I,M)
3089  WRITE(2,2030) BENDF(I,M),BENDFAB(I,M),BENDF3(I,M),NTYPE(I,M)
1089 CONTINUE
1088 CONTINUE
     DO M=NFB*((NLOADTM-1)/2)+1,NFB*((NLOADTM+1)/2)
       DO I=1,3
         BFAC(I,M)=BFA(I,M-NFB*(NLOADTM-1)/2)
         BFBC(I,M)=BFB(I,M-NFB*(NLOADTM-1)/2)
         BFAFC(I,M)=BFAFC(I,M-NFB)+
    *              BFA(I,M-NFB*(NLOADTM-1)/2)
         BFBFC(I,M)=BFBFC(I,M-NFB)+
    *              BFB(I,M-NFB*(NLOADTM-1)/2)
       END DO
     END DO
     CLOSE(2)
     DO M=NFC*((NLOADTM-1)/2)+1,NFC*((NLOADTM+1)/2)
       DO I=1,3
         CFAC(I,M)=CFA(I,M-NFC*(NLOADTM-1)/2)
         CFBC(I,M)=CFB(I,M-NFC*(NLOADTM-1)/2)
         CFAFC(I,M)=CFAFC(I,M-NFC)+
    *              CFA(I,M-NFC*(NLOADTM-1)/2)
         CFBFC(I,M)=CFBFC(I,M-NFC)+
    *              CFB(I,M-NFC*(NLOADTM-1)/2)
       END DO
     END DO
1099 CONTINUE
2032 FORMAT(I5)
4025 CONTINUE
     WRITE(3,900)
     DO M=1,NFB
       WRITE(3,950) M,(NLOADTM+1)/2,INB(1,M),RR(1,M),INB(2,M),RR(2,M)
     END DO
900  FORMAT(///10X,' ****   BEAM END ROTATION   ****'//
    *5X,'BEAM NO.',3X,'LOADCASE',2X,'START JO.',3X,'ROTATION',3X,
    *'END JOINT'3X,'ROTATION.')
950  FORMAT(8X,I2,10X,I1,8X,I2,7X,F8.6,6X,I2,7X,F8.6)
     IF((NLOADTM+1)/2.LT.NLOAD) GOTO 77
350  WRITE(3,300)
```

```
300 FORMAT(//12X,' ******* JOINT DISPLACEMENTS *******'//
   &5X,'JOINT',2X,'LOAD',8X,'X-TRANS ',7X,'Y-TRANS',8X,'Z-TRANS')
    WRITE(3,303)
303 FORMAT(12X,'CASE',9X,'(mm)',11X,'(mm)',11X,'(mm)'/)
    NSTT=NST+1
    NODE=1
    DO 5005 N=1,NSTT
      MSN=N*NTJ/NSTT
      NMSN=MSN-1
      DO 5010 J=NODE,NMSN
        IF(NY.NE.1) GO TO 340
        WRITE(3,310) J,(DISPC(I,J),I=1,3)
        DO L=2,NLOAD
          WRITE(3,3310)L,(DISPC(I,NTJ*(L-1)+J),I=1,3)
        END DO
340     DO L=1,NLOAD
          IF(L.GT.1) GO TO 3366
          IF(NY.EQ.1) GO TO 3366
          WRITE(3,3360)J,L,(DISPFC(I,NTJ*(L-1)+J),I=1,3)
          GO TO 3365
3366      WRITE(3,3320) L,(DISPFC(I,NTJ*(L-1)+J),I=1,3)
3365    CONTINUE
        END DO
310     FORMAT(4X,I2,7X,'1',1X,3F15.2)
3360    FORMAT(4X,I2,7X,I1,'C',3F15.2)
3310    FORMAT(12X,I2,1X,3F15.2)
3320    FORMAT(12X,I2,'C',3F15.2)
5010  CONTINUE
      NODE=MSN+1
5005 CONTINUE
370  WRITE(3,301)
301  FORMAT(//14X,' ******* JOINT ROTATIONS ******* '//
   &5X,'JOINT',2X,'LOAD',8X,' X-ROT ',8X,' Y-ROT ',8X,' Z-ROT '/)
     WRITE(3,312)
312  FORMAT(12X,'CASE',5X,'(RAD*10^3)',5X,'(RAD*10^3)',5X,'(RAD*10^3)
   *'//)
     NODE=1
     DO 1005 N=1,NSTT
       MSN=N*NTJ/NSTT
       NMSN=MSN-1
       DO 1010 J=NODE,NMSN
         IF(NY.NE.1) GO TO 360
         WRITE(3,329) J, ROTC(1,J),ROTC(2,J),ROTC(3,J)
         DO L=2,NLOAD
           WRITE(3,3330)L,(ROTC(I,NTJ*(L-1)+J),I=1,3)
         END DO
360      DO L=1,NLOAD
           IF(L.GT.1.OR.NY.EQ.1) GO TO 3376
           WRITE(3,3370)J,L,(ROTFC(I,NTJ*(L-1)+J),I=1,3)
           GO TO 3375
3376       WRITE(3,3340) L,(ROTFC(I,NTJ*(L-1)+J),I=1,3)
3375     CONTINUE
```

```
         END DO
329      FORMAT(4X,I2,7X,'1',1X,3F15.5)
3370     FORMAT(4X,I2,7X,I1,'C',3F15.5)
3330     FORMAT(12X,I2,1X,3F15.5)
3340     FORMAT(12X,I2,'C',3F15.5)
1010  CONTINUE
      NODE=MSN+1
1005 CONTINUE
355  CONTINUE
      WRITE(3,69)
69  FORMAT(///10X,' ****   BEAM END FORCES AND MOMENTS   ****'//
    *20X,'****   START   ****'//
    *5X,'BEAM NO.',1X,'JOINT',1X,'LOAD',10X,'AXIAL',12X,'MX',13X,'MZ')
      WRITE(3,199)
199 FORMAT(20X,'CASE',10X,'(kn)',11X,'(kn-m)',9X,'(kn-m)'/)
      DO 606 M=1,NFB
        IF(NY.NE.1) GO TO 4360
          WRITE(3,4329) M,INB(1,M), (BFAC(I,M),I=1,3)
          DO L=2,NLOAD
            WRITE(3,4330)L,(BFAC(I,NFB*(L-1)+M),I=1,3)
          END DO
4360      DO L=1,NLOAD
            IF(L.GT.1.OR.NY.EQ.1) GO TO 4376
            WRITE(3,4370)M,INB(1,M),L,(BFAFC(I,NFB*(L-1)+M),I=1,3)
            GO TO 4375
4376        WRITE(3,4340) L,(BFAFC(I,NFB*(L-1)+M),I=1,3)
4375      CONTINUE
          END DO
4329      FORMAT(7X,I2,6X,I2,5X,'1',1X,3F15.2)
4370      FORMAT(7X,I2,6X,I2,5X,I1,'C',3F15.2)
4330      FORMAT(21X,I2,1X,3F15.2)
4340      FORMAT(21X,I2,'C',3F15.2)
606  CONTINUE
      WRITE(3,68)
68  FORMAT(///10X,' ****   BEAM END  MOMENTS   ****'//
    *20X,'****   END   ****'//
    *5X,'BEAM NO.',1X,'JOINT',1X,'LOAD',10X,'AXIAL',12X,'MX',13X,'MZ')
      WRITE(3,199)
      DO 67 M=1,NFB
        IF(NY.NE.1) GO TO 5360
          WRITE(3,4329) M,INB(2,M), (BFBC(I,M),I=1,3)
          DO L=2,NLOAD
            WRITE(3,4330)L,(BFBC(I,NFB*(L-1)+M),I=1,3)
          END DO
5360      DO L=1,NLOAD
            IF(L.GT.1.OR.NY.EQ.1) GO TO 5376
            WRITE(3,4370)M,INB(2,M),L,(BFBFC(I,NFB*(L-1)+M),I=1,3)
            GO TO 5375
5376        WRITE(3,4340) L,(BFBFC(I,NFB*(L-1)+M),I=1,3)
5375      CONTINUE
          END DO
67  CONTINUE
```

```
      WRITE(3,35)
35    FORMAT(//10X,'*** COLUMN OUT-OF-PLANE END FORCES  ***'
     *//19X,'**** LOWER-END ****'//
     *5X,'COLUMN',3X,'JOINT',1X,'LOAD',10X,'AXIAL',12X,'MX',13X,'MZ')
      WRITE(3,199)
      DO 99 M=1,NFC
        IF(NY.NE.1) GO TO 6360
          WRITE(3,4329) M,INC(1,M), (CFAC(I,M),I=1,3)
          DO L=2,NLOAD
            WRITE(3,4330)L,(CFAC(I,NFC*(L-1)+M),I=1,3)
          END DO
6360      DO L=1,NLOAD
            IF(L.GT.1.OR.NY.EQ.1) GO TO 6376
            WRITE(3,4370)M,INC(1,M),L,(CFAFC(I,NFC*(L-1)+M),I=1,3)
            GO TO 6375
6376        WRITE(3,4340) L,(CFAFC(I,NFC*(L-1)+M),I=1,3)
6375      CONTINUE
          END DO
99    CONTINUE
      WRITE(3,34)
34    FORMAT(//10X,'*** COLUMN OUT-OF-PLANE END FORCE AND MOMENT ***'//
     *19X,'**** UPPER-END ****'//
     *5X,'COLUMN',3X,'JOINT',1X,'LOAD',10X,'AXIAL',12X,'MX',13X,'MZ')
      WRITE(3,199)
      DO 109 M=1,NFC
        IF(NY.NE.1) GO TO 7360
          WRITE(3,4329) M,INC(2,M), (CFBC(I,M),I=1,3)
          DO L=2,NLOAD
            WRITE(3,4330)L,(CFBC(I,NFC*(L-1)+M),I=1,3)
          END DO
7360      DO L=1,NLOAD
            IF(L.GT.1.OR.NY.EQ.1) GO TO 7376
            WRITE(3,4370)M,INC(2,M),L,(CFBFC(I,NFC*(L-1)+M),I=1,3)
            GO TO 7375
7376        WRITE(3,4340) L,(CFBFC(I,NFC*(L-1)+M),I=1,3)
7375        CONTINUE
          END DO
109   CONTINUE
C
77    CONTINUE
      RETURN
      END
C
C     DETERMINE LOADING OR UNLOADING AND PERFORM SOME ARRAY
C
      SUBROUTINE UNLOAD(IBCT,CS,AMO,RO,AN,SP,BENDF,ROTAT,BENDFAB,BENDFL,
     *                  NTYPE,ROTAT3,BENDF3,SIGN0,NLOADTM)
      IMPLICIT REAL*8 (A-H,O-Z)
      COMMON/GEN/ NTJ,NFB,NFC,NBAND,NEQ,NXM
      DIMENSION BENDF(2,NFB),ROTAT(2,NFB),BENDFAB(2,NFB),BENDFL(2,NFB),
     *          NTYPE(2,NFB),SIGN0(2,NFB),ROTAT3(2,NFB),BENDF3(2,NFB),
     *          CS(2,NFB),AMO(2,NFB),RO(2,NFB),AN(2,NFB),SP(2,NFB),
```

```
     *          IBCT(2,NFB)
C
      OPEN(2,FILE='FROTAT.OUT')
      DO  K=1,NFB
         DO I=1,2
           READ(2,71) ROTAT(I,K),ROTAT3(I,K)
         END DO
      END DO
71    FORMAT(2F15.5)
722   FORMAT(3F12.2,3F12.5)
      CLOSE(2)
771   OPEN(2,FILE='FMOMENT.OUT')
      REWIND(UNIT=2)

      DO 80 M=1,NFB
      DO 90 I=1,2
        READ(2,72) BENDF(I,M),BENDFAB(I,M),BENDF3(I,M),NTYPE(I,M)
90    CONTINUE
80    CONTINUE
72    FORMAT(3F15.2,3I5)
777   FORMAT(2F15.2)
      CLOSE(2)
      OPEN(2,FILE='BFAB2.OUT')
      DO M=1,NFB
        READ(2,777) (BENDFL(I,M),I=1,2)
      END DO
      CLOSE(2)
      DO 10 M=1,NFB
      DO 20 I=1,2
        II=IBCT(I,M)
        IF (II.EQ.1.OR.II.EQ.2) GO TO 20
        IF (NTYPE(I,M).EQ.1) THEN
          SIGN0(I,M)=BENDFL(I,M)*BENDF(I,M)
          IF(SIGN0(I,M).GE.0.0) GO TO 20
          NTYPE(I,M)=2
          GO TO 20
        END IF
        IF (NTYPE(I,M).EQ.3) THEN
          SIGN0(I,M)=BENDFL(I,M)*BENDF3(I,M)
          IF(SIGN0(I,M).GE.0.0) GO TO 20
          NTYPE(I,M)=4
          GO TO 20
        END IF
20    CONTINUE
10    CONTINUE
      DO 30 M=1,NFB
        DO 40 I=1,2
          JJ=NTYPE(I,M)
          II=IBCT(I,M)
          IF (II.EQ.1.OR.II.EQ.2) GO TO 40
          IF (JJ.EQ.1) RC=ROTAT(I,M)
          IF (JJ.EQ.3) RC=ROTAT3(I,M)
```

```
        IF (JJ.EQ.1) THEN
          SCO=AMO(I,M)/RO(I,M)*1000.
          SPP=ABS((1/RO(I,M)-SP(I,M)/(RO(I,M)*SCO))*RC)**AN(I,M)
          CS(I,M)=SCO*((1-SP(I,M)/SCO)/((1+SPP)**(1/AN(I,M)))+
     *           SP(I,M)/SCO)-SCO*SPP/((1+SPP)**(1+1/AN(I,M)))
        END IF
40    CONTINUE
30  CONTINUE
    RETURN
    END
C$INCLUDE BMF.FOR
```

```
C
C   THIS SUBROUTINE COMPUTE THE BEAM END FORCES.
C
    SUBROUTINE  BMF ( CN, INB, IBP, SC, JL, BFA, BFB)
    IMPLICIT REAL*8 (A-H,O-Z)
    COMMON  /GEN/ NTJ, NFB, NFC, NBAND, NEQ, NXM
    COMMON /SECT/ BP(4,20), CP(4,20)
    COMMON /MAT/ EM,GM
    COMMON /TEMPB1/NPRT,NFIL,IPDELT,NX13
    DIMENSION  CN(3,NTJ), INB(2,NFB), JL(3,NTJ), BFA(3,NFB),
   1 BFB(3,NFB), IBP(NFB), SC(2,NFB)
    REAL *8 JL
C
    FACTOR=1000.
C
4449 DO  30  M=1, NFB
    SA=SC(1,M)
    SB=SC(2,M)
    CALL  BELM(M, CN,INB, IBP, SA, SB, S1, S2, S3, S4, S5,
   áS22, S44, R11, R22, R23, R32, R33, I, J)
    F1 = S1*R11*(JL(1,J) - JL(1,I)) - R23*(S2*JL(2,J)+S22*JL(2,I))
   1 - R33*(S2*JL(3,J)+S22*JL(3,I))
    BFA(1,M) = BFA(1,M) - F1
    BFB(1,M) = BFB(1,M) + F1
    F1 = S3*R22*(JL(2,J) - JL(2,I)) + S3*R32*(JL(3,J) - JL(3,I))
    BFA(2,M) = BFA(2,M) - F1
    BFB(2,M) = BFB(2,M) + F1
    BFB(3,M) = BFB(3,M) - R11*S2*(JL(1,J) - JL(1,I))
   1 + R23*(S4*JL(2,J) + S5*JL(2,I)) + R33*( S4*JL(3,J) +
   1 S5*JL(3,I))
    BFA(3,M) = BFA(3,M) - R11*S22*(JL(1,J) -JL(1,I))
   1 + R23*(S5*JL(2,J) + S44*JL(2,I)) + R33*(S5*JL(3,J)
   1 + S44*JL(3,I))
30  CONTINUE
    PRINT 35
35  FORMAT(////5x,' *****   BEAM  END  FORCES  AND  MOMENTS   *****'//
   á17X,'****   A-END ****'//
   á5x,'BEAM NO.',2X,'JOINT',11X,'P2',14X,'M1',11X,'M3'/)
    DO  40  K = 1, NFB
    BFA(2,K)=BFA(2,K)/FACTOR
    BFA(3,K)=BFA(3,K)/FACTOR
    BFB(2,K)=BFB(2,K)/FACTOR
    BFB(3,K)=BFB(3,K)/FACTOR
    PRINT 45,K,INB(1,K),( BFA(J,K), J=1,3)
45  FORMAT(5x,I4,5X,I4,1X,3F15.2)
40  CONTINUE
    WRITE(*,36)
36  FORMAT(////5x,'***** BEAM END FORCES AND MOMENTS *****'//
   &17X,'***** B-END *****'//
   &5x,'BEAM NO.',2X,'JOINT',10X,'P2',14X,'M1',14X,'M3')
    DO 41 K=1,NFB
    PRINT 46, K,INB(2,K),(BFB(J,K),J=1,3)
```

```
46  FORMAT(5x,I4,6X,I3,1X,3F15.2)
41  CONTINUE
44  FORMAT(2F15.2)
    RETURN
    END
C
C
C
C   THIS SUBROUTINE COMPUTES THE COLUMN END FORCES
C   AND MOMENTS FOR EACH COLUMN.  THE END FORCES AND MOMENTS
C   ARE PRINTED AS OUT-OF-PLANE AND IN-PLANE VECTORS SEPERATELY.
C
C
    SUBROUTINE CMF (CN, INC, ICP, JL, CFA, CFB, CIA, CIB, PP, NLOADTM,
   *            NITER)
    IMPLICIT REAL*8 (A-H,O-Z)
    COMMON  /IPT/ NST, NBAY1,NBAY3
    COMMON  /GEN/ NTJ, NFB, NFC, NBAND, NEQ, NXM
    COMMON /SECT/ BP(4,20), CP(4,20)
    COMMON /TEMPB1/NPRT,NFIL,IPDELT,NX13
    DIMENSION  CN(3,NTJ), INC(2,NFC), JL(3,NTJ), PP(NFC),
   1 CFA(3,NFC), CFB(3,NFC),CIA(3,NFC), CIB(3,NFC), ICP(NFC)
    REAL *8 JL
    INTEGER V,M,U,L
C
    OPEN(4,FILE='COLMOME.OUT')
    FACTOR=1000.
4453 DO  10  K = 1, NFC
    DO  20  KK = 1, 3
    CFA(KK,K) = 0.D0
    CFB(KK,K) = 0.D0
    CIA(KK,K) = 0.D0
    CIB(KK,K) = 0.D0
20  CONTINUE
10  CONTINUE
    MM = 1
    MN = NFC/NST
    DO  30  N = 1, NST
    M = ( N+1 )*NTJ/ (NST + 1)
    V = N*NTJ/(NST + 1)
    DO  40  J = MM, MN
    CALL  CELM ( J, CN,ICP, INC, A, B, C, D, E, F, G, H,
   1  P, Q, U, L, V, M, ZU, XU, ZL, XL, DL)
    PL=PP(J)/DL
    Q=Q+DL*DL*PL/30
    C=C-DL*DL*PL/7.5
    D=D-DL*PL/10
    F=F-1.2*PL
    F1 = A*(JL(1,U) - JL(1,L))
    CFB(1,J) = F1 + CFB(1,J)
    CFA(1,J) = - F1 + CFA(1,J)
    IF (NITER.EQ.1.AND.REAL(NLOADTM/2).NE.REAL(NLOADTM)/2) THEN
```

```
       PP(J)=PP(J)+CFA(1,J)
      END IF
      CFB(2,J) = CFB(2, J) + E*((JL(2,V) - JL(3,V)*XU) -
     1 (JL(2,M) - JL(3,M)*XL)) - B*JL(2,U) - P*JL(2,L)
      CFA(2,J) = CFA(2,J) + E*((JL(2,V)  - JL(3,V)*XU) -
     1 (JL(2,M) - JL(3,M)*XL)) - P*JL(2,U) - B*JL(2,L)
      CFB(3,J) = CFB(3,J) + D*((-JL(1,M) - JL(3,M)*ZL) -
     1 (-JL(1,V) - JL(3,V)*ZU)) + C*JL(3,U) + Q*JL(3,L)
      CFA(3,J) = CFA(3,J) + D*((-JL(1,M) - JL(3,M)*ZL) -
     1 (-JL(1,V) - JL(3,V)*ZU)) + Q*JL(3,U) + C*JL(3,L)
      F1 = F*( -JL(1,V) - JL(3,V)*ZU) - D*(JL(3,U) + JL(3,L))
     1  - F*(-JL(1,M) - JL(3,M)*ZL)
       CIB(1,J) = CIB(1,J) + F1
       CIA(1,J) = CIA(1,J) - F1
      F1 = G*( JL(2,V) - JL(3,V)*XU) + E*(-JL(2,U))
     1- G*(JL(2,M) - JL(3,M)*XL) + E*(-JL(2,L))
       CIB(2,J) = CIB(2,J) + F1
       CIA(2,J) = CIA(2,J) - F1
      F1 = H*(JL(3,V) - JL(3,M))
      CIB(3,J) = CIB(3,J) + F1
      CIA(3,J) = CIA(3,J) - F1
40    CONTINUE
      MM = MN + 1
      MN = (N+1)*NFC/NST
30    CONTINUE
      PRINT 35
35    FORMAT(////5x,'**** COLUMN OUT-OF-PLANE END FORCES AND MOMENTS'//
     $17X,'****  L-END  ****'//
     º5x,'COLUMN',4X,'JOINT',10X,'P1',13X,'M2',13X,'M3')
      DO  50  NN = 1, NFC
      CFA(2,NN)=CFA(2,NN)/FACTOR
      CFA(3,NN)=CFA(3,NN)/FACTOR
      CFB(2,NN)=CFB(2,NN)/FACTOR
      CFB(3,NN)=CFB(3,NN)/FACTOR
      PRINT 36,NN,INC(1,NN),(CFA(K,NN),K=1,3)
36    FORMAT(5x,I4,6X,I3,1X,3F15.2)
50    CONTINUE
      WRITE(*,34)
34    FORMAT(///5x,'*** COLUMN OUT-OF-PLANE END FORCE AND MOMENT *** '//
     &17X,'**** U-END **** '//
     &5x,'COLUMN',4X,'JOINT ',10X,'P1',13X,'M2',13X,'M3')
      DO 51 NN=1,NFC
      WRITE(*,37)NN,INC(2,NN),CFB(1,NN),CFB(2,NN),CFB(3,NN)
37    FORMAT(5x,I4,6X,I3,1X,3F15.2)
51    CONTINUE
      RETURN
      END
C
C     SUBROUTINE LOAD PROCESSES LOADING INFORMATION.
C     FIXED END FORCE VECTOR IS COMPUTED.
C     ENTRY STATEMENT IS USED FOR RELOAD THE STRUCTURE
C     FOR NONLINEAR  ANALYSIS.
```

```
C
C
      SUBROUTINE LOAD(CN, INB, IBP, CS, JD, AJL, BFA, BFB, AMLC,
     áAMLU, MU, MJ, MC,NLOADTM)
      IMPLICIT REAL*8 (A-H,O-Z)
      COMMON /GEN/NTJ, NFB, NFC, NBAND, NEQ, NXM
      COMMON /MAT/EM, GM
      COMMON /COM8/ BFAA(3,100),BFBB(3,100)
      COMMON /SECT/BP(4,20), CP(4,20)
      COMMON /ALOAD/NLJ, NLMC, NLMU, NLT1, NLT2, NLT3
      COMMON /TEMPB1/NPRT,NFIL,IPDELT,NX13
      DIMENSION CN(3,NTJ), INB(3,NFB), IBP(NFB), CS(2,NFB), JD(3,NTJ),
     áAJL(3,NLT1), BFA(3,NFB), BFB(3,NFB), AMLC(2,NLT2),
     áAMLU(NLT3), MU(NLT3), MC(NLT2), MJ(NLT1)
      REAL*8 JD,I3
      CHARACTER ESC*1
      ESC=CHAR(27)
C
      FACTOR=1000.
      IF(NLOADTM.EQ.1) THEN
        WRITE(3,35)
35      FORMAT(//5X,'**** LOAD COMBINATION AS INPUT ****')
      END IF
      CALL CLSC
      WRITE(*,'(//10X,''INPUT LOADING INFORMATION''\)')
      IF(REAL(NLOADTM/2).NE.REAL(NLOADTM)/2.AND.NLOADTM.GT.1) THEN
        NFIL=2
        NFIIL=2
        GO TO 888
      END IF
      WRITE(*,'(//10X,''DO YOU WANT NEW LOAD SYSTEM? YES or NO :'')')
      CALL YN(NFIIL)
C
C     NEW SCREEN
888   CALL CLSC
      IF(NLJ.EQ.0)GOTO 100
C
C
C     INPUT JOINT LOAD AND STORE IN A TEMPORARY
C     JOINT LOAD VECTOR.
C
      IF (NFIL.EQ.1.OR.NFIIL.EQ.1) GO TO 2000
      WRITE(*,'(//10X,''INPUT JOINT LOAD INFORMATION''\)')
      WRITE(*,'(/10X,''JOINT NO.'',5X,''FORCE'',4X,''FORCE OR MOMENT'',
     &2X,''FORCE OR MOMENT'')')
C
      OPEN(2,FILE='LOADOFJO.IN')
      DO 10 J=1,NLJ
      READ(2,*) N,P,T,B
      PRINT 1022,N,P,T,B
1022  FORMAT(12X,I3,6X,F8.2,4X,F10.2,8X,F10.2)
1002  FORMAT(7X,I3,6X,F8.2,4X,F10.2,8X,F10.2)
```

```
      MJ(J)=N
      AJL(1,J)=P
      AJL(2,J)=T
      AJL(3,J)=B
10    CONTINUE
      CLOSE(UNIT=2)
      GO TO 2092
C
2000 DO 2055 J=1,NLJ
        CALL SC
2050    WRITE(*,'(//10X,''LOADED JOINT NUMBER:'')')
        CALL RD(N,RL,55)
C
2051    WRITE(*,'(//10X,''FORCE:'')')
        CALL RD(INT,P,55)
C
2052    WRITE(*,'(//10X,''FORCE OR MOMENT:'')')
        CALL RD(INT,T,55)
C
2053    WRITE(*,'(//10X,''FORCE OR MOMENT:'')')
        CALL RD(INT,B,55)
C
        MJ(J)=N
        AJL(1,J)=P
        AJL(2,J)=T
        AJL(3,J)=B
2055 CONTINUE
2092 CONTINUE
C
C     EDIT ABOVE INPUT ?
C
      IF(REAL(NLOADTM/2).NE.REAL(NLOADTM)/2.AND.NLOADTM.GT.1)GO TO 2200
C
      WRITE(*,1115)
1115 FORMAT(//10X,'EDIT INPUT? YES or NO:')
      CALL YN(IY)
      IF (IY.NE.1) GO TO 2200
      CALL CLSC
      WRITE(*,1020)
1020 FORMAT(//10X,'TO MODIFY,INPUT,NEW VALUE')
      DO J=1,NLJ
        CALL SC
        WRITE(*,'(//10X,''LOADED JOINT NUMBER''/)')
        WRITE(*,'(10X,I5)') MJ(J)
        CALL ED(MJ(J),55)
C
        WRITE(*,'(//10X,''FORCE''/)')
        WRITE(*,'(10X,F10.2)') AJL(1,J)
        CALL EDR(AJL(1,J),55)
C
        WRITE(*,'(//10X,''FORCE OR MOMENT''/)')
        WRITE(*,'(10X,F10.2)') AJL(2,J)
```

```
        CALL EDR(AJL(2,J),55)
C
        WRITE(*,'(//10X,''FORCE OR MOMENT''/)')
        WRITE(*,'(10X,F10.2)') AJL(3,J)
        CALL EDR(AJL(3,J),55)
      END DO
2200 OPEN(2,FILE='LOADOFJO.IN')
      IF(REAL(NLOADTM)/2.EQ.REAL(NLOADTM/2)) GO TO 2233
      WRITE(3,646) (NLOADTM+1)/2
646  FORMAT(/5X,'LOAD CASE',2X,I3)
      WRITE(3,666)
666  FORMAT(/5X,'JOINT LOADING')
      WRITE(3,1001)
1001 FORMAT(/5X,'JOINT NO.',5X,'FORCE',5X,'FORCE OR MOMENT',3X,
     *        'FORCE OR MOMENT')
2233 DO 2062 J=1,NLJ
      IF(REAL(NLOADTM)/2.EQ.REAL(NLOADTM/2)) GO TO 2244
      WRITE(3,1002) MJ(J),AJL(1,J),AJL(2,J),AJL(3,J)
2244 WRITE(2,*) MJ(J),AJL(1,J),AJL(2,J),AJL(3,J)
2062 CONTINUE
      CLOSE(UNIT=2)
2065 CONTINUE
C
100  IF(NLMC.EQ.0) GOTO 200
C
C
C    INPUT  CONCENTRATED GRAVITY LOAD ON BEAM
C    AND STORE IN A TEMPORARY BEAM LOAD VECTOR
C
      IF (NFIL.EQ.1.OR.NFIIL.EQ.1) GOTO 2070
      OPEN(UNIT=2,FILE='CONCENTR.IN')
      PRINT 1003
1003 FORMAT(//10X,' CONCENTRATED GRAVITY LOAD ON BEAM'/10X,
     á  ' BEAM NO.',5X,'LOAD','(kN)',5X,'DISTANCE FROM A-END','(m)')
C
      DO 20 M=1, NLMC
        READ(2,*) N, P, AL
        PRINT 1004,N,P,AL
1004    FORMAT(15X,I5,5X,F10.2,16X,F10.2)
        MC(M)=N
        AMLC(1,M)=P
        AMLC(2,M)=AL*FACTOR
20    CONTINUE
      CLOSE(UNIT=2)
      GO TO 2085
2070 DO 2079 M=1,NLMC
2075   WRITE(*,'(//10X,''BEAM NO.:'')')
        CALL RD(N,RL,55)
C
2076   WRITE(*,'(//10X,''LOAD:'')')
        CALL RD(INT,P,55)
C
```

```
2077  WRITE(*,'(//10X,''DISTANCE FROM A-END:'')')
      CALL RD(INT,AL,55)
C
      MC(M)=N
      AMLC(1,M)=P
      AMLC(2,M)=AL*FACTOR
2079 CONTINUE
2080 CONTINUE
C
    WRITE(*,1115)
    CALL YN(IY)
    IF (IY.NE.1) GO TO 2085
2081 WRITE(*,'(//10X,''CONCENTRATED LOAD TO EDITED'')')
    CALL CLSC
    WRITE(*,1020)
    DO M=1,NLMC
      CALL SC
      WRITE(*,'(//10X,''BEAM NO.:'')')
      WRITE(*,'(10X,I5)') MC(M)
      CALL ED(MC(M),55)
C
      WRITE(*,'(//10X,''LOAD:'')')
      WRITE(*,'(10X,F10.2)') AMLC(1,M)
      CALL EDR(AMLC(1,M),55)
C
      WRITE(*,'(//10X,''DISTANCE FROM A-END:'')')
      WRITE(*,'(10X,F10.2)') AMLC(2,M)
      CALL EDR(AMLC(2,M),55)
    END DO
C
2085 OPEN(UNIT=2,FILE='CONCENTR.IN')
    WRITE(3,1003)
    DO 2086 M=1,NLMC
    WRITE(2,*) MC(M),AMLC(1,M),AMLC(2,M)/FACTOR
    WRITE(3,1004) MC(M),AMLC(1,M),AMLC(2,M)/FACTOR
2086 CONTINUE
2087 CONTINUE
C
200  IF(NLMU.EQ.0) GOTO 300
C
C
C   INPUT UNIFORMLY DISTRIBUTED LOAD ON BEAM
C   AND STORE IN TEMPORARY BEAM LOAD VECTOR
C
    FACTOR=1000.
    IF (NFIL.EQ.1.OR.NFIIL.EQ.1) GOTO 3000
    OPEN(UNIT=2,FILE='UNIFORML.IN',STATUS='OLD')
    PRINT 1005
1005 FORMAT(//10X,'UNIFORMLY DISTRIBUTED LOAD ON BEAM'/
   á 10X,'BEAM NO.',7X,'LOAD')
C
    DO 30 M=1, NLMU
```

```
        READ(2,*) N, P
        PRINT 1006,N, P
1006    FORMAT(9X,I5,5X,F10.2)
        MU(M)=N
        AMLU(M)=P/FACTOR
30    CONTINUE
      GOTO 3040
3000 CALL CLSC
      DO 3030 M=1,NLMU
        CALL SC
        WRITE(*,'(//10X,''BEAM NUMBER'')')
        CALL RD(N,RL,55)
C
        WRITE(*,'(//10X,''LOAD'')')
        CALL RD(INT,P,55)
C
        MU(M)=N
        AMLU(M)=P/FACTOR
3030 CONTINUE
1000 WRITE(*,'(//10X,''EDIT INPUT ? YES or NO:'')')
      CALL YN(IY)
      IF (IY.NE.1) GOTO 3040
      CALL SC
      WRITE(*,'(10X,''UNIFORM LOAD TO BE EDITED'')')
      WRITE(*,1020)
      DO M=1,NLMU
        WRITE(*,'(//10X,''BEAM NUMBER'')')
        WRITE(*,'(10X,I5)') MU(M)
        CALL ED(MU(M),55)
C
        WRITE(*,'(//10X,''LOAD'')')
        WRITE(*,'(10X,F10.2)') AMLU(M)*FACTOR
        CALL EDR(AML,55)
        AMLU(M)=AML/FACTOR
      END DO
C
3040 OPEN(UNIT=2,FILE='UNIFORML.IN',STATUS='UNKNOWN')
      WRITE(3,656)
      WRITE(3,1005)
656   FORMAT(/5X,'LOAD CASE   1')
      DO 3045 M=1,NLMU
        WRITE(2,*) MU(M),AMLU(M)*FACTOR
        WRITE(3,1006) MU(M),AMLU(M)*FACTOR
3045 CONTINUE
3050 CONTINUE
      CLOSE(2)
C
300   CONTINUE
      ENTRY RELOAD(CN,INB,IBP,CS,JD,AJL,BFA,BFB,AMLC,AMLU,MU,MJ,MC)
C
C     ZERO BEAM END FORCE VECTOR
C
```

```
      DO 40 M=1, NFB
      DO 40 MM=1, 3
      BFA(MM,M)=0.0
      BFAA(MM,M)=0.0
      BFB(MM,M)=0.0
      BFBB(MM,M)=0.0
40    CONTINUE
C
C      ZERO JOINT LOAD VECTOR
C
      DO 50 J=1,NTJ
      DO 50 JJ=1,3
      JD(JJ,J)=0.0
50    CONTINUE
C
      IF(NLJ.EQ.0) GOTO 400
C
C     APPLY INPUT JOINT LOAD TO JOINT LOAD VECTOR
C
      DO 60 J=1, NLJ
      K=MJ(J)
      JD(1,K)=AJL(1,J)
      JD(2,K)=AJL(2,J)
      JD(3,K)=AJL(3,J)
60    CONTINUE
C
400   IF (NLMC.EQ.0) GOTO 500
C
C     COMPUTE FIXED END FORCE DUE TO CONCENTRATED GRAVITY LOAD,
C     AND STORE IN FIXED END FORCE VECTOR AND ADD THE NEGATIVE
C     TO THE JOINT LOAD VECTOR.
C
      DO 70 M=1,NLMC
      K=MC(M)
      P=AMLC(1,M)
      AL=AMLC(2,M)
      SA=CS(1,K)
      SB=CS(2,K)
      CALL BELM(K,CN,INB,IBP,SA,SB,S1,S2,S3,S4,S5,S22,S44,R11,R22,
     áR23,R32,R33,I,J)
      IR2=R22
      IR3=R23
      IF(IR2.NE.0) DL=DABS(CN(1,I)-CN(1,J))
      IF(IR3.NE.0) DL=DABS(CN(3,I)-CN(3,J))
      BL=DL-AL
      MP=IBP(K)
      I3=BP(3,MP)
      DB1=(P*AL**3)/(3.*EM*I3)+(P*AL**2*BL)/(2.*EM*I3)+(P*DL*AL)/SA
      DB3=P*AL**2/(2.*EM*I3)+(P*AL)/SA
      PBF1=S2*DB3-S1*DB1
      PBF3=S2*DB1-S4*DB3
      PAF1=-(P+PBF1)
```

```
      PAF3=-(P*AL+DL*PBF1+PBF3)
      BFA(1,K)=BFA(1,K)+PAF1
      BFAA(1,K)=BFAA(1,K)+BFA(1,K)
      BFB(1,K)=BFB(1,K)+PBF1
      BFBB(1,K)=BFBB(1,K)+BFB(1,K)
      BFA(3,K)=BFA(3,K)+PAF3
      BFAA(3,K)=BFAA(3,K)+BFA(3,K)
      BFB(3,K)=BFB(3,K)+PBF3
      BFBB(3,K)=BFBB(3,K)+BFB(3,K)
      JD(1,I)=JD(1,I)-PAF1
      JD(1,J)=JD(1,J)-PBF1
      IF(IR3.NE.0)GOTO 65
      JD(3,I)=JD(3,I)-(PAF3*R33)
      JD(3,J)=JD(3,J)-(PBF3*R33)
      GOTO 70
65    CONTINUE
      JD(2,I)=JD(2,I)-(PAF3*R23)
      JD(2,J)=JD(2,J)-(PBF3*R23)
70    CONTINUE
C
500   IF(NLMU.EQ.0)RETURN
C
C     COMPUTE FIXED END FORCE DUE TO UNIFORMLY DISTRIBUTED
C     LOAD VECTOR, AND STORE IN THE BEAM END FORCE VECTOR.THE
C     NEGATIVE OF THE FIXED END FORCE VECTOR IS ADDED TO THE
C     JOINT LOAD VECTOR
C
      DO 80 M=1, NLMU
      K=MU(M)
      SA=CS(1,K)
      SB=CS(2,K)
      W=AMLU(M)
      MP=IBP(K)
      I3=BP(3,MP)
      CALL BELM(K,CN,INB,IBP,SA,SB,S1,S2,S3,S4,S5,S22,S44,R11,R22,
     áR23,R32,R33,I,J)
      IR2=R22
      IR3=R32
      IF(IR2.NE.0)DL=DABS(CN(1,I)-CN(1,J))
      IF(IR3.NE.0)DL=DABS(CN(3,I)-CN(3,J))
      DB1=(W*DL**4)/(8.*EM*I3)+(W*DL**3)/(2.*SA)
      DB3=(W*DL**3)/(6.*EM*I3)+(W*DL**2)/(2.*SA)
      WFB1=S2*DB3-S1*DB1
      WFB3=S2*DB1-S4*DB3
      WFA1=-(W*DL+WFB1)
      WFA3=-(W*DL**2/2.+DL*WFB1+WFB3)
      BFA(1,K)=BFA(1,K)+WFA1
      BFAA(1,K)=BFAA(1,K)+BFA(1,K)
      BFB(1,K)=BFB(1,K)+WFB1
      BFBB(1,K)=BFBB(1,K)+BFB(1,K)
      BFA(3,K)=BFA(3,K)+WFA3
      BFAA(3,K)=BFAA(3,K)+BFA(3,K)
```

```
      BFB(3,K)=BFB(3,K)+WFB3
      BFBB(3,K)=BFBB(3,K)+BFB(3,K)
      JD(1,I)=JD(1,I)-WFA1
      JD(1,J)=JD(1,J)-WFB1
      IF(IR3.NE.0) GOTO 75
      JD(3,I)=JD(3,I)-(WFA3*R33)
      JD(3,J)=JD(3,J)-(WFB3*R33)
      GOTO 80
 75   CONTINUE
      JD(2,I)=JD(2,I)-(WFA3*R23)
      JD(2,J)=JD(2,J)-(WFB3*R23)
 80   CONTINUE
      RETURN
      END
C
C
C
C     THIS SUBROUTINE TAKE IN THE BRACING ELEMENTS
C     END INCIDENCE TO THE JOINTS.
C
C
      SUBROUTINE INTRS( INT,ITP,NLOADTM)
      IMPLICIT  REAL*8 (A-H,O-Z)
      COMMON /TBRACE/ NFT, NBF
      COMMON /SECT2/ TP(20)
      DIMENSION  INT(2,NFT), ITP(NFT)
C
C     INPUT BRACING INCIDENCE
C
      IF(NLOADTM.GT.1) GO TO 1010
      CALL CLSC
      WRITE(*,'(//10X,''NEW BRACING SYSTEM? YES or NO :'')')
      CALL YN(NFIL)
C
C     NEW SCREEN
C
      CALL CLSC
      IF (NFIL.EQ.1) GO TO 2000
1010  OPEN(2,FILE='BRAINCID.IN')
      DO  M=1, NFT
         READ(2,*)N,INT(1,M),INT(2,M)
100   END DO
      CLOSE(2)
      GO TO 2300
2000  DO 2055 M=1,NFT
        CALL SC
2050    WRITE(*,'(//10X,''BRACING MEMBER:'')')
        CALL RD(M,RL,55)
C
        WRITE(*,'(//10X,''JOINT OF LOWER END CONNECTION:'')')
        CALL RD(NL,RL,55)
C
```

```
        WRITE(*,'(//10X,''JOINT OF UP END CONNECTION:'')')
        CALL RD(NU,RL,55)
C
        INT(1,M)=NL
        INT(2,M)=NU
2055    CONTINUE
C
C    EDIT ABOVE INPUT ?
C
      WRITE(*,1115)
1115 FORMAT(//10X,'EDIT INPUT? YES or NO:')
      CALL YN(IY)
      IF (IY.NE.1) GO TO 2300
      CALL CLSC
      WRITE(*,1020)
1020 FORMAT(//10X,'TO MODIFY,INPUT,NEW VALUE')
      DO 2065 M=1,NFT
        CALL SC
        WRITE(*,'(//10X,''BRACING NUMBER:'')')
        WRITE(*,'(10X,I5)') M
        CALL ED(M,55)
C
        WRITE(*,'(//10X,''JOINT OF LOWER END CONNECTION:'')')
        WRITE(*,'(10X,I5)') INT(1,M)
        CALL ED(INT(1,M),55)
C
        WRITE(*,'(//10X,''JOINT OF UP END CONNECTION:'')')
        WRITE(*,'(10X,I5)') INT(2,M)
        CALL ED(INT(2,M),55)
2065 CONTINUE
2300 OPEN(2,FILE='BRAINCID.IN')
      WRITE(3,1001)
1001 FORMAT(/'BRACING NMBER',8X,'LOWER END',2X,'UP END')
      DO 2062 M=1,NFT
        WRITE(3,1002) M,INT(1,M),INT(2,M)
        WRITE(2,*) M,INT(1,M),INT(2,M)
1002 FORMAT(//4X,I5,12X,I5,5X,I5)
2062 CONTINUE
C
C    INPUT BRACING PROPERTIES
C
      IF(NLOADTM.GT.1) GO TO 1040
      WRITE(*,'(//10X,'' NEW BRACING PROPERTIES? YES or NO :'')')
      CALL YN(NFIL)
C
C     NEW SCREEN
C
      IF (NFIL.EQ.1) GO TO 2800
1040 OPEN(2,FILE='BRAPROP.IN')
          READ (2,*) NTP
          DO  105 N=1, NTP
             READ (2,*)  TP(N)
```

```
105     CONTINUE
      CLOSE(2)
      GO TO 2850
2800 CALL SC
      WRITE(*,'(//10X,''BRACING MEMBER:'')')
      CALL RD(NTP,RL,55)
      WRITE(*,'(//10X,''AREA OF X-SECTION:'')')
      DO  108 N=1, NTP
         CALL RD(IN,RA,55)
         TP(N)=RA
108  CONTINUE
C
C    EDIT ABOVE INPUT ?
C
      WRITE(*,1115)
      CALL YN(IY)
      IF (IY.NE.1) GO TO 2850
      CALL CLSC
      WRITE(*,1020)
      CALL SC
      WRITE(*,'(//10X,''BRACING MEMBER:'')')
      WRITE(*,'(10X,I5)') NTP
      CALL ED(NTP,55)
      DO  N=1, NTP
         WRITE(*,'(//10X,''AREA OF X-SECTION:'')')
         WRITE(*,'(10X,F10.3)') TP(N)
         CALL ED(TP(N),55)
      END DO
2850 OPEN(2,FILE='BRAPROP.IN')
      WRITE(2,*) NTP
      DO N=1,NTP
        WRITE(2,*) TP(N)
      END DO
      CLOSE(2)
      WRITE(3,102) NTP
102  FORMAT(/'  NUMBER OF BRACING TYPE-',I3//
     º'BRACING TYPE',5X,'X-SECTION AREA')
      DO N=1,NTP
        WRITE(3,103) N, TP(N)
      END DO
103  FORMAT(9X,I3,5X,F11.2)
104  CONTINUE
C
C    INPUT BRACING PROPERTIES ID NO..
C
      IF(NLOADTM.GT.1) GO TO 1030
      WRITE(*,'(//10X,''NEW BRACING PROPERTIES ID ? YES or NO:'')')
      CALL YN(NFIL)
C
C    NEW SCREEN
C
      CALL CLSC
```

```
      IF (NFIL.EQ.1) GO TO 2900
 1030 OPEN(2,FILE='BRAIDNO.IN')
      READ (2,*) (ITP(N), N=1,NFT)
      CLOSE(2)
      GO TO 2950
 2900 CALL SC
      DO N=1,NFT
        WRITE(*,'(//10X,''BRACING ID NO.:'')')
        CALL RD(NID,RL,55)
        ITP(N)=NID
      END DO
C
C     EDIT ABOVE INPUT ?
C
      WRITE(*,1115)
      CALL YN(IY)
      IF (IY.NE.1) GO TO 2950
      CALL CLSC
      WRITE(*,1020)
      CALL SC
      DO N=1,NFT
        WRITE(*,'(//10X,''BRACING PROPERTY ID.:'')')
        WRITE(*,'(10X,I5)') ITP(N)
        CALL ED(ITP(N),55)
      END DO
 2950 OPEN(2,FILE='BRAIDNO.IN')
      WRITE (2,*) (ITP(N), N=1,NFT)
      CLOSE(2)
      RETURN
      END
C
C
C
C     THIS SUBROUTINE PERFORMS AN ASSEMBLY OF THE STIFFNESS
C     COEFFICIENTS OF EACH INDIVIDUAL BRACING ELEMENT
C     INTO THE STRUCTURE STIFFNESS MATRIX.
C
C
      SUBROUTINE  BRACE( CN, INT, SS, ITP )
      IMPLICIT  REAL*8 (A-H,O-Z)
      COMMON /MAT/ EM, GM
      COMMON /GEN/ NTJ, NFB, NFC, NBAND, NEQ, NXM
      COMMON /TBRACE/ NFT, NBF
      COMMON /SECT2/ TP(20)
      COMMON /IPT/ NST, NBAY1, NBAY3
      INTEGER V,M,U,L
      DIMENSION  CN(3,NTJ), INT(2,NFT), SS(NXM), ITP(NFT)
C
C
      KK(L1,M1,L3,M3)=(3*L1-3*M1+L3-M3)*NEQ + 3*M1 + M3 - 3
      J=0
      DO 50 JJ=1, NBF
```

```
      DO 100 N=1,NST
      M=(N+1)*NTJ/(NST+1)
      V=N*NTJ/(NST+1)
      DO 200 NN=1,2
      J=J+1
C
C     KODE IS USED TO KEEP TRACK OF THE DEGREE OF FREEDOM
C     OF BRACING ELEMENTS IN TWO POSSIBLE ORIENTATION OF
C     BRACING IN A RECTANGULAR BUILDING
C
      CALL TELM(J,CN,ITP,INT,S1,S2,S3,U,L,KODE,COSA,SINA,AEL)
      K=KK(U,U,1,1)
      SS(K)=SS(K) + S2
      K=KK(V,U,KODE,1)
      SS(K)=SS(K) + S3
      K=KK(V,V,KODE,KODE)
      SS(K)=SS(K) + S1
      K=KK(L,U,1,1)
      SS(K)=SS(K) - S2
      K=KK(L,V,1,KODE)
      SS(K)=SS(K) - S3
      K=KK(L,L,1,1)
      SS(K)=SS(K) + S2
      K=KK(M,U,KODE,1)
      SS(K)=SS(K) - S3
      K=KK(M,V,KODE,KODE)
      SS(K)=SS(K) - S1
      K=KK(M,L,KODE,1)
      SS(K)=SS(K) + S3
      K=KK(M,M,KODE,KODE)
      SS(K)=SS(K) + S1
200   CONTINUE
100   CONTINUE
50    CONTINUE
      RETURN
      END
C
C
C
C     THIS SUBROUTINE COMPUTES THE STIFFNESS COEFICIENTS
C     OF EACH BRACING ELEMENT.
C
C
      SUBROUTINE  TELM(M, CN, ITP, INT, S1, S2, S3, U, L, KODE,COSA,
     *                 SINA,AEL)
      IMPLICIT REAL*8 (A-H,O-Z)
      COMMON /GEN/ NTJ, NFB, NFC, NBAND, NEQ, NXM
      COMMON /MAT/ EM, GM
      COMMON /TBRACE/ NFT, NBF
      COMMON /SECT2/ TP(20)
      INTEGER U, L
      DIMENSION CN(3,NTJ), ITP(NFT), INT(2,NFT)
```

```
C
C
      NCASE=1
      GOTO 100
      ENTRY BFORCE(M,COSA,SINA,AEL,KODE,U,L,ITP,INT,S1,S2,S3,CN)
      NCASE=2
100   MP=ITP(M)
      A=TP(MP)
      L=INT(1,M)
      U=INT(2,M)
      X1=CN(1,L)
      Y1=CN(2,L)
      Z1=CN(3,L)
      X2=CN(1,U)
      Y2=CN(2,U)
      Z2=CN(3,U)
      KODE=1
      IF(DABS(X1-X2) .LT. 0.0001 ) KODE=2
      GOTO(150,160), KODE
150   DL=DSQRT((X2-X1)**2+(Y2-Y1)**2)
      AL=X2-X1
      GOTO 170
160   DL=DSQRT((Z2-Z1)**2+(Y2-Y1)**2)
      AL=Z2-Z1
170   COSA=AL/DL
      SINA=(Y2-Y1)/DL
      AEL=A*EM/DL
      GOTO( 200, 300) , NCASE
200   S1=COSA*AEL*COSA
      S2=SINA*AEL*SINA
      S3=SINA*AEL*COSA
300   RETURN
      END
C
C
C
C     THIS SUBROUTINE COMPUTES THE BRACING END FORCES.
C
C
      SUBROUTINE  FBRACE( CN, INT, ITP, JD, TFA, TFB ,COSA, SINA)
      IMPLICIT REAL*8 ( A-H,O-Z)
      COMMON  /IPT/ NST, NBAY1, NBAY3
      COMMON  /GEN/ NTJ, NFB, NFC, NBAND, NEQ, NXM
      COMMON  /TBRACE/ NFT, NBF
      COMMON  /SECT2/ TP(20)
      DIMENSION  CN(3,NTJ), INT(2,NFT), JD(3,NTJ), TFA(NFT),
     áTFB(NFT), ITP(NFT)
      REAL *8 JD
      INTEGER V,M,U,L
C
C
      WRITE(*,5)
```

```
5    FORMAT(///' ****  BRACING ELEMENT END FORCES'//
     á12X,'LOWER END',23X,'UPPER END'/
     á'BRACING',5X,'JOINT',5X,'AXIAL FORCE',5X,'JOINT',4X,'AXIAL FORCE')
      DO  10  K=1, NFT
      TFA(K)=0.D0
      TFB(K)=0.D0
10    CONTINUE
      MT=0
      DO 20 J=1, NBF
      DO 30 N=1, NST
      M=(N+1)*NTJ/(NST+1)
      V=N*NTJ/(NST+1)
      DO 40 NT=1, 2
      MT=MT+1
      CALL BFORCE( MT,COSA,SINA,AEL,KODE,U,L,ITP,INT,S1,S2,S3,CN)
      ZU=CN(3,U)-CN(3,V)
      XU=CN(1,U)-CN(1,V)
      ZL=CN(3,L)-CN(3,M)
      XL=CN(1,L)-CN(1,M)
      GOTO (100, 200) , KODE
100   DSPV=JD(1,V)+ZU*JD(3,V)
      DSPM=JD(1,M)+ZL*JD(3,M)
      GOTO 300
200   DSPV=JD(2,V)-XU*JD(3,V)
      DSPM=JD(2,M)-XL*JD(3,M)
300   DSPU=JD(1,U)
      DSPL=JD(1,L)
      F1=AEL*(COSA*(DSPV-DSPM)+SINA*(DSPU-DSPL))
      TFB(MT)=F1
      TFA(MT)=-F1
      PRINT 400, MT, INT(1,MT),TFA(MT),INT(2,MT),TFB(MT)
400   FORMAT( 3X,I2,8X,I4,4X,F12.2,5X,I4,6X,F12.2)
40    CONTINUE
30    CONTINUE
20    CONTINUE
      RETURN
      END
C
C
C     THIS SUBROUTINE PERFORMS THE P-DELTA EFFECT CHECK
C     ACCORDING TO THE LOADCONTROL NEWTON-RAPHSON METHOD
C
C
      SUBROUTINE  PDELTA( CN,INB,INC,IBP,ICP,SC,JL,AJL,PP,NPD,NPDT)
      IMPLICIT REAL*8 (A-H,O-Z)
      COMMON  /IPT/ NST, NBAY1,NBAY3
      COMMON  /GEN/ NTJ, NFB, NFC, NBAND, NEQ, NXM
      COMMON /SECT/ BP(4,20), CP(4,20)
      COMMON /MAT/ EM,GM
      COMMON /ALOAD/NLJ, NLMC, NLMU, NLT1, NLT2, NLT3
      COMMON /COM8/ BFAA(3,100),BFBB(3,100)
      COMMON /TEMPB1/NPRT,NFIL,IPDELT,NX13
```

```
      DIMENSION  CN(3,NTJ), INB(2,NFB), JL(3,NTJ), BFA(3,90),BFB(3,90)
     *,IBP(NFB), SC(2,NFB),INC(2,NFC),AJL(3,NLT1),CCFA(3,100),
     * CCFB(3,100),CCIA(3,100),CCIB(3,100),ICP(NFC),FFORIN(3,100),
     * QINC(3,100),PP(NFC),AAJL(3,100)
      REAL *8 JL
      INTEGER V,M,U,L
C
      DO I=1,NLT1
        DO J=1,3
          AAJL(J,I)=0.D0
        END DO
      END DO
      RMAX=0.D0
      NPD=0.D0
      IF (NPDT.GT.1) GO TO6
      DO I=1,NTJ
        DO J=1,3
          FFORIN(J,I)=0.D0
        END DO
      END DO
6     CONTINUE
      DO I=1,NTJ
        DO J=1,3
          QINC(J,I)=0.D0
        END DO
      END DO
C
C     CALCULATE BEAM END FORCES
C
      DO  60  M=1, NFB
      SA=SC(1,M)
      SB=SC(2,M)
      CALL  BELM(M, CN,INB, IBP, SA, SB, S1, S2, S3, S4, S5,
     &S22, S44, R11, R22, R23, R32, R33, I, J)
      F1 = S1*R11*(JL(1,J) - JL(1,I)) - R23*(S2*JL(2,J)+S22*JL(2,I))
     1 - R33*(S2*JL(3,J)+S22*JL(3,I))
      BFA(1,M) = BFAA(1,M) - F1
      BFB(1,M) = BFBB(1,M) + F1
      F1 = S3*R22*(JL(2,J) - JL(2,I)) + S3*R32*(JL(3,J) - JL(3,I))
      BFA(2,M) = BFAA(2,M) - F1
      BFB(2,M) = BFBB(2,M) + F1
      BFB(3,M) = BFBB(3,M) - R11*S2*(JL(1,J) - JL(1,I))
     1 + R23*(S4*JL(2,J) + S5*JL(2,I)) + R33*( S4*JL(3,J) +
     1 S5*JL(3,I))
      BFA(3,M) = BFAA(3,M) - R11*S22*(JL(1,J) -JL(1,I))
     1 + R23*(S5*JL(2,J) + S44*JL(2,I)) + R33*(S5*JL(3,J)
     1 + S44*JL(3,I))
        QINC(2,I)=QINC(2,I)+BFA(1,M)
        QINC(2,J)=QINC(2,J)+BFB(1,M)
        QINC(3,I)=QINC(3,I)+BFA(3,M)*R22
        QINC(3,J)=QINC(3,J)+BFB(3,M)*R22
60    CONTINUE
```

```
C
C   CALCULATE COLUMN END FORCES
C
      DO 10 K=1,NFC
        DO 20  KK = 1, 3
          CCFA(KK,K) = 0.D0
          CCFB(KK,K) = 0.D0
          CCIA(KK,K) = 0.D0
          CCIB(KK,K) = 0.D0
20      CONTINUE
10    CONTINUE
      MM = 1
      MN = NFC/NST
      DO 30  N = 1, NST
        M = ( N+1 )*NTJ/ (NST + 1)
        V = N*NTJ/(NST + 1)
        DO 40  J = MM, MN
          CALL  CELM ( J, CN,ICP, INC, A, B, C, D, E, F, G, H,
     1              P, Q, U, L, V, M, ZU, XU, ZL, XL ,DL)
          PL=PP(J)/DL
          D=D-DL*PL/10
          C=C-DL*DL*PL/7.5
          Q=Q+DL*DL*PL/30
          F=F-PL*1.2
          F1 = A*(JL(1,U) - JL(1,L))
          CCFB(1,J) = F1 + CCFB(1,J)
          CCFA(1,J) = - F1 + CCFA(1,J)
          CCFB(3,J) = CCFB(3,J) + D*((-JL(1,M) - JL(3,M)*ZL) -
     1              (-JL(1,V) - JL(3,V)*ZU)) + C*JL(3,U) + Q*JL(3,L)
          CCFA(3,J) = CCFA(3,J) + D*((-JL(1,M) - JL(3,M)*ZL) -
     1              (-JL(1,V) - JL(3,V)*ZU)) + Q*JL(3,U) + C*JL(3,L)
          F1 = F*( -JL(1,V) - JL(3,V)*ZU) - D*(JL(3,U) + JL(3,L))
     1        - F*(-JL(1,M) - JL(3,M)*ZL)
          CCIB(1,J) = CCIB(1,J) + F1
          CCIA(1,J) = CCIA(1,J) - F1
C
          K=INC(1,J)
          L=INC(2,J)
          QINC(1,K)=QINC(1,K)+CCIA(1,J)
          QINC(1,L)=QINC(1,L)+CCIB(1,J)
          AAJL(1,V)=AAJL(1,V)-CCIB(1,J)
          IF (M.LE.NLT1) THEN
            AAJL(1,M)=AAJL(1,M)-CCIA(1,J)
          END IF
          QINC(2,K)=QINC(2,K)+CCFA(1,J)
          QINC(2,L)=QINC(2,L)+CCFB(1,J)
          QINC(3,K)=QINC(3,K)+CCFA(3,J)
          QINC(3,L)=QINC(3,L)+CCFB(3,J)
40      CONTINUE
        MM = MN + 1
        MN = (N+1)*NFC/NST
30    CONTINUE
```

```
C
      NSTT=NST+1
      NODE=1
      NCPF=NFC/NST
      DO 70 N=1,NST
        V=N*NTJ/NSTT
        DO 80 J=NODE,NCPF
          K=INC(1,J)
          L=INC(2,J)
          AAJL(3,L)=AJL(3,L)+(AJL(3,L)-QINC(3,L))
          IF (K.LE.NLT1) THEN
            AAJL(1,K)=AJL(1,K)+(AJL(1,K)-QINC(2,K))
          END IF
80      CONTINUE
        ERROR=DABS((AJL(1,V)-AAJL(1,V))/AJL(1,V))
        IF (ERROR.GT.RMAX) THEN
          RMAX=ERROR
        END IF
        AAJL(1,V)=AJL(1,V)+(AJL(1,V)-AAJL(1,V))
        NODE=NCPF+1
        NCPF=(N+1)*NFC/NST
70    CONTINUE
      PRINT *,RMAX
      DO J=1,NTJ
        DO I=1,3
          FFORIN(I,J)=QINC(I,J)
        END DO
      END DO
      DO I=1,NLJ
        AJL(1,I)=AAJL(1,I)
      END DO
      IF (RMAX.GT.0.001) NPD=NPD+1
      RETURN
      END
```