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CONTRACTOR REPORT ARLCD-CR-77008

**NONLINEAR ANALYSIS OF FRAME STRUCTURES
SUBJECTED TO BLAST OVERPRESSURES**

WILLIAM STEA
GEORGE TSENG
DAVID KOSSOVER
AMMANN & WHITNEY
NEW YORK, NEW YORK

PAUL PRICE
JOSEPH CALTAGIRONE
ARRADCOM

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**US ARMY ARMAMENT RESEARCH AND DEVELOPMENT COMMAND
LARGE CALIBER
WEAPON SYSTEMS LABORATORY
DOVER, NEW JERSEY**

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cont. → equations and procedures necessary for its use. The report also contains example problems illustrating the use of DYNFA for the design of blast-resistant frame structures.

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SUMMARY

This report was developed in recognition of the need for expanded design information pertaining to structures subjected to blast environments produced by accidental explosions. It was prepared by the Manufacturing Technology Division of the Large Caliber Weapons Systems Laboratory, U.S. Army Armament Research and Development Command (AARADCOM). The purpose of the report is to provide facility designers with criteria and procedures for the design of the primary structural frames of steel buildings subjected to the effects of HE-type explosions. Integral with this purpose, the report presents a computer program titled "Dynamic Non-linear Frame Analysis" (referred to as DYNFA) for determining the response of frame structures subjected to time-dependent blast loadings. This report is intended to be used in conjunction with Picatinny Arsenal Technical Report 4837, "Design of Steel Structures to Resist the Effects of HE Explosions", the tri-service design manual, "Structures to Resist the Effects of Accidental Explosions" (TM 5-1300), and the AISC "Manual of Steel Construction".

The blast-resistant design of a frame structure is based on a dynamic analysis with the DYNFA program. The program implements a method of analysis which couples a lumped parameter representation of the structure with a numerical integration procedure to obtain a solution for the response. Inelastic behavior of the frame members and second order effects produced by the deflections of the frame are considered in the analysis performed by DYNFA. The response of the structure is expressed in terms of the deformations of the structure and the axial loads, bending moments and shears in each of the members. Inelastic behavior is measured on the basis of the end rotations of the frame members.

The report presents the basis for the DYNFA analysis, as well as methods and guidelines for applying lumped parameter modeling concepts to simulate frame structures and the blast environments to which they are subjected. A user's manual is provided which gives instructions for transmitting to DYNFA the input data describing the lumped parameter model of the structure. The manual describes the capabilities of the program, the required input data, the formats of the input data cards and the arrangement of the input data deck. A discussion on the utilization of the printed output of the program is also provided.

A preliminary frame design method is presented for use in conjunction with the DYNFA analysis. This method provides a means for making efficient and rational selections of initial

member sizes which are subsequently verified, and, if necessary, modified on the basis of the DYNFA analysis.

Methods are also presented for applying DYNFA to situations involving the design of columns subjected to bi-axial bending.

Criteria governing the design of blast-resistant frames are presented in terms of limits on the maximum deflections, member end rotations and levels of inelastic dynamic response.

Detailed procedures are presented for the blast-resistant design of frame structures. Procedures are given for two general design cases, namely: buildings subjected to normal shock waves and buildings subjected to quartering shock waves. For each general case, procedures are presented which correspond to the various phases of the frame design. The material presented includes procedures for the following: the preliminary frame design, the preparation of the analytical model of the structure and the related input data for DYNFA, and the utilization of the results of DYNFA in conjunction with the design criteria to verify the blast resistance of the frame. Each such procedure is illustrated by a numerical example.

SECTION 1

INTRODUCTION

1.1 Background

In the design of steel buildings to withstand the effects of high-explosive (HE) and other types of chemical explosions, standard structural members can be utilized for structures located in pressure ranges of 70.0 kPa (10.0 psi) or less. However, because of the transient nature and relatively high intensity of the applied loads, special procedures and criteria are required to fully define the response of the structure to the blast output.

Steel buildings consist of three general structural systems: (1) the walls and roof panels, (2) supporting members such as girts, purlins, diagonal bracing and other members which can be treated as individual elements, and (3) the main structural frame. The blast-resistant design of the first two systems, because of their relative simplicity, can be accomplished with analyses of single-degree-of-freedom systems (Ref 1, 2 and 3); whereas the design of the main frames will involve multi-degree-of-freedom system analyses. The complexities inherent in this type of analysis combined with the effects produced by the transient loads and the inelastic action of a frame require the aid of high-speed computers for a solution. Hence, a computer program titled "Dynamic Non-linear Frame Analysis" (hereinafter referred to as DYNFA) was developed whereby the response of general types of frame structures (such as those shown in Fig 1) subjected to arbitrary blast loads could be obtained. In addition, design criteria are presented which establish upper limits for the frame response as computed with DYNFA.

The U.S. Army, under the direction of the Project Manager for Production Base Modernization and Expansion, is currently engaged in a multi-billion dollar program to modernize and expand its ammunition production capability. In support of this program, the Manufacturing Technology Division of the Large Caliber Weapons Systems Laboratory, AARADCOM, with the assistance of Ammann & Whitney, Consulting Engineers, has, for the past several years, been engaged in a broad based program to improve explosive safety at these facilities. One segment of this program deals with the development of design criteria for explosion-resistant protective structures. Part of this effort involves the development of a comprehensive procedure for the design of blast-resistant steel frames. Computer program DYNFA and the related frame design criteria form the basis of this procedure.

The frame design procedure and the aforementioned computer program and related criteria are presented in this report.

1.2 Purpose and Scope

The basic purpose of this report is to provide ammunition facility designers with criteria and procedures for the design of the primary structural frames of steel buildings subjected to the effects of HE-type explosions.

Integral with the basic purpose stated above, this report also provides facility designers with instructions and guidelines for the use of DYNFA. Illustrative examples are also provided to assist facility designers in using the program.

This report has been prepared with the following general guidelines:

1. The material contained in this report supplements and, in some instances, modifies the criteria and procedures presented in Picatinny Arsenal Technical Report 4837 (Ref 1). Therefore, it is assumed that this report will be used in conjunction with TR 4837 by designers possessing a basic familiarity with the contents of that report.
2. As in the case of TR 4837, the blast loads used in the dynamic analysis are determined using the procedures and data contained in the Tri-Service Design Manual, TM 5-1300, "Structures to Resist the Effects of Accidental Explosions" (Ref 3).
3. Although the method of analysis presented herein is directed primarily towards structural steel frame buildings, it is also applicable to reinforced concrete frame structures. It should be noted, however, that the design criteria provided in this report is primarily applicable to steel structures and will require modification for the design of concrete structures.
4. The major emphasis has been placed on design of structures located some distance from the immediate blast area and, therefore, the material presented herein has limited application to structures located close-in to an explosion.

5. The detailed provisions for inelastic blast-resistant design of steel members shall be consistent with conventional static plastic design procedures as presented in the AISC "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings" (Ref 4) with the following exceptions: (1) the plastic strength of a section under the blast loading is determined using the appropriate dynamic yield stress for the material instead of the specified minimum yield stress, and (2) the load factors specified in Section 2.1 of Reference 4 are not applied when designing for blast loads.
6. In all cases, the static provision of the AISC Specification represents a minimum requirement for conventional dead and live loads. Moreover, it is presumed that designers using this report are familiar with static plastic design procedures for steel.
7. In general, the influence of conventional dead and live loads can be neglected in blast design or in the evaluation of the capacity of a blast-resistant structure. However, the effect of such loads upon the available capacity for blast resistance may be significant in the design of structures for relatively low overpressures; e.g., less than 7.0 kPa (1 psi) or in the evaluation of the blast resistance of a structure designed for conventional loads.

1.3 Format of the Report

This report is divided into sections and appendices devoted to individual topics related to the analysis and design of steel frame structures subjected to blast overpressures. For the purpose of clarity of presentation, most of the directly applicable material from TR 4837, TM 5-1300 and the AISC Specification and Manual is not repeated in this report. As far as possible, applicable equations, design charts and tables, and commentary material are included herein by reference. However, certain data provided in TR 4837 is repeated in Sections 2, 6 and 8 of this report for continuity. The following sections contain quantitative description of the frame analysis and related items:

Section 2 provides a procedure for the preliminary design of single-story frames subjected to blast loads. The procedure presented is a simplification of the preliminary frame design procedure presented in TR 4837.

Section 3 presents the basis for the dynamic analysis of frame structures as performed by DYNFA.

Sections 4 and 5 present methods and guidelines for analytically simulating frame structures and the blast loading environments to which they are subjected. The discussion is directed towards translating the various aspects of the physical problem into the input data for DYNFA.

Section 6 outlines the design criteria which is utilized to ascertain whether the computed response of a frame is within acceptable limits.

Section 7 presents procedures and criteria for utilizing DYNFA when the frame design includes bi-axial bending of the columns.

The last two sections are devoted entirely to the use of DYNFA. Section 8 describes the capabilities of the program together with a detailed description of the required input. Section 9 describes the printed output of the program and includes a discussion on the interpretation and utilization of the results to verify the adequacy of a frame.

Appendix A is devoted to illustrative examples on the use of DYNFA. In Appendix A, the methods, equations and guidelines presented in the body of the report, are implemented in several illustrative examples including one showing the design of frames with in-plane bending of the columns, and another illustrating a frame design which includes bi-axial bending of the columns.

Appendix B contains samples of the printed output of the program.

Appendices C through F contain supporting data for the report. The theory implemented in the program is presented in Appendix C, while Appendix D contains the program listing. Symbols and definitions of the terms used in the report (and program) are listed in Appendix E.

Since future standards of measurements in the United States will be based upon the SI Units (International System of Units), instead of the United States System of Units now in use, all measurements presented in this report will conform to those of the SI Units. However, for those individuals not completely familiar with the SI Units, the equivalent units in the U.S. System are provided in parentheses adjacent to the SI Unit, where appropriate. A list containing the units, symbols, and the United States System conversion factors for the SI Units is presented in Appendix F.

1.4 Components and Behavior of Steel Structures

General

In most cases, single-story rectangular structures will be encountered in the design of ammunition production and explosive manufacturing facilities. Depending upon the operation involved, the rectangular structure may be subdivided into one or more bays in either direction. The size (or sizes) of the bays will be determined by the layout of the column supports of the frame. In general, steel buildings may be rigidly framed either in one or both directions, as shown in Figure 2. Buildings which are framed in one direction are usually provided with lateral bracing in the other direction. Two-directional framing systems usually utilize common column supports, as illustrated in Figure 2b.

The rigid frame is the primary support component of a steel building (Fig 3), while the purlins, girts, roof and wall paneling form the secondary support system. As previously mentioned, the elements of the secondary system usually can be designed using single-degree-of-freedom system analyses (Ref 1, 2 and 3), while the design of the rigid frames will involve multi-degree-of-freedom system analyses.

Structural Behavior

The economy of facility design generally requires that steel structures be designed to perform in the inelastic response range when subjected to blast overpressures. In order to insure the structure's integrity throughout such severe conditions, the facility designer must be cognizant of the various possible failure modes and their inter-relationships. The limiting design values are dictated by the attainment of inelastic deflections and rotations without complete collapse. The amount of inelastic deformation is dependent not only upon the ductility characteristics of the material, but also upon the intended use of the structure following an incident. For the structure to maintain large plastic deformations, steps must be taken to prevent premature failure by either brittle fracture or instability (local or overall). Guidelines and criteria for dealing with these effects are presented in this report.

1.5 Dynamic Analysis and Design

General

The design of steel buildings subjected to blast pressures involves performing dynamic analyses of a series of interior and

exterior frames spanning transversely and/or longitudinally through the buildings. For a given structure, the analysis of one representative interior and one exterior frame, spanning in each direction, generally can be used as a basis for designing all the frames throughout the building. However, in situations where large differentials in the blast loads occur and/or where large variations in frame spacings are prevalent, analyses of additional frames may be needed.

Basis of Analysis

The design of rigid frames, as treated in this report, is based upon the premise that the frames provide the entire resistance to lateral loads; hence, the sidesway resistance, afforded by the diaphragm action of the roof and wall panels, is neglected. In recent years, the use of diaphragm action has become more common in the design of conventional buildings for wind loads. Data and criteria pertaining to the strength and stiffness of cold-formed steel panels subjected to in-plane loads have been developed through extensive test programs (Ref 5). In applications involving conventional structures, the utilization of diaphragm action can produce substantial economies in design and construction. However, the wall and roof panels of blast-resistant buildings are normally designed to perform in the inelastic response range under the action of normal loads. The occurrence of inelastic strains in the material will decrease, to some extent, the in-plane shear capacity produced by the diaphragm action of the panels. In the absence of test data, the extent of the decrease of the in-plane shear capacity cannot be ascertained. It is quite possible that the diaphragms, in the inelastic condition, will have a very small in-plane shear capacity; therefore, until test data indicates otherwise, it is not considered appropriate to utilize diaphragm action in the design of blast-resistant structures.

In addition to diaphragm action, some resistance to the lateral movement of the exterior frames will be afforded by the presence of the wall girts spanning between the columns of the frames. The girts will tend to distribute the lateral loads among all of the columns; whereas in the frame analysis, the entire load is applied to the blastward column. Because of this, a somewhat conservative design for the exterior frames will be realized.

Each rigid frame is assumed to support a portion (or strip) of the building (Fig 4a). The supported strip is framed by the girts and purlins which span between adjoining frames. The portion of the building supported by a given frame extends to the full height of the structure and, for an interior frame, has a width equal to one-half of the sum of the distances between the

frame in question and the two adjacent frames. End frames are assumed to support a strip equal in width to one-half of the distance between the end frame and the first interior frame.

In the frame analysis, the loads acting on the contributory areas of the building are assumed to be concentrated at the centerline of the frames. This assumption neglects two effects which, if included in the analysis, would reduce the response of the frame; hence, the assumption introduces a measure of conservatism into the design of the frames. The effects referred to are: the energy absorption produced by the inelastic action of the secondary members (purlins, girts, etc.) and the rise time on the loading of the frame members due to the response of the secondary framing. However, the analysis does consider the time-lag effects associated with the blast wave sweeping across the structure.

In most cases, each frame can be analyzed independently; therefore, the scope of the analysis can be limited to two-dimensional rigid frames. However, when bi-axial bending of the building columns occurs, provision must be made in the analysis to account for the reduction in the column capacities caused by the simultaneous application of bending moments about both axes of the members.

In general, bi-axial bending occurs in some, if not all, of the columns of the building, depending on the direction of propagation of the blast wave. When the building is loaded in a quartering direction (Fig 4b), all of the columns experience bi-axial bending. When the shock front is parallel to the front wall of the building, only the exterior columns of the side walls undergo bi-axial bending. In these latter cases, the analyses of the frames perpendicular to the blast direction may be omitted. However, the response of the exterior columns to the side-on blast pressures is still required to design these members. This response may be estimated by a single-degree-of-freedom analysis of the member.

Design Procedure

The design of steel frames is basically a trial-and-error procedure. In order to determine the required member sizes, the response of the frame must be known. However, the response of the frame cannot be computed unless the sizes of the members have been determined; consequently, it is usually necessary to proceed with the analysis and design of the frame simultaneously.

The first step in the design of a frame is the selection of sizes of trial members. In situations involving structures loaded in the low-to-intermediate pressure range, the designs will involve

the use of standard structural shapes strengthened to provide the required blast protection. The adequacy of these members is determined by analyzing the frames using DYNFA. The results of the DYNFA analysis will indicate whether the inelastic deformations which occur during response are within the limiting design values specified in the body of this report.

The final design is determined by achieving an economical balance between stiffness and resistance such that the calculated response under the blast loading lies within the limiting values dictated by the operational requirements of the facility. In some cases, this may require several analyses of the frame with DYNFA. The number of analyses required is dependent upon the accuracy with which the trial members are selected. In order to limit the number of analyses required, a preliminary design method is provided in this report for the purpose of providing facility designers with the means for making efficient and rational selections of the initial member sizes. The method provided is a simplified version of the preliminary design procedure presented in Chapter 5 of Reference 1.

SECTION 2

PRELIMINARY FRAME DESIGN METHOD

2.1 Introduction

The objective of this section is to provide rational procedures for efficiently performing the preliminary design of the primary structural frames of steel buildings subjected to the effects of HE-type explosions. The preliminary frame design method comprises four tasks, which are:

1. Computation of the peak blast loads acting on the structure.
2. Selection of representative frames for analysis with DYNFA.
3. Computation of required plastic moment capacities for the members of the selected frames.
4. Selection of member sizes for all frames throughout the building.

The member sizes thus obtained are subsequently verified and, if necessary, revised on the basis of a series of rigorous dynamic analyses of the selected frames with DYNFA.

The blast loads used for the preliminary design are computed using the peak pressures obtained from Section 4 of Reference 3. A description of the procedures for computing these pressures is provided in Section 2.2. The determination of the blast loads for the preliminary design is not related to the discussion in Section 5, which is directed towards generating mathematical functions of pressure versus time for use with DYNFA.

For the general case of structures subjected to quartering shock waves, the quantities required are the peak loads acting on blastward walls as well as the roof. For structures subjected to normally directed shock waves, the quantities required are the peak loads on the blastward wall, the roof and the side wall. The effects of the loading on the leeward wall(s) is not considered for the preliminary design.

Following the computation of the blast loads, several frames are selected for analysis with DYNFA. For structures subjected to quartering shock waves, several representative frames spanning in

each direction are chosen. In such cases, the basis for the selection is a comparison of the blast loads (horizontal and vertical) acting on all frames positioned normal to each blastward wall. If the loads on all interior frames positioned normal to a given wall are nearly equal (within a few percent), analysis of only one such frame is required. In addition, if the loads on the exterior frames differ slightly, only one exterior frame is analyzed. Hence, in cases such as these, the frames selected for analysis are the blastward exterior frame and the interior frame adjacent to it. If the blast loads on successive frames differ greatly, several additional interior frames, as well as the leeward exterior frame, should be analyzed for economy of design. Such differences in the blast loading on successive frames are produced by: rapid decaying of shock front pressure as the blast wave traverses the structure; unequal spacing of frames; and variations in the secondary framing plan of the roof and walls from one framing bay to another. In most cases involving normal shock waves, DYNFA analyses are required for only two frames: a typical interior frame and an exterior frame. However, variations in the frame spacing or the secondary framing plan of the roof or walls may produce a requirement for analyses of additional frames.

Upon the completion of the first two tasks, the preliminary design proceeds with the computation of the required plastic moment capacities for the members of the selected frames. These quantities are computed on the basis of an analysis of each selected frame using the mechanism method as employed in static plastic design (Ref 6). Section 2.3 presents general expressions for computing the limiting bending capacities which correspond to the possible collapse mechanisms of single-story rigid and braced frames.

Each selected frame is analyzed independently; interactions between orthogonal frames caused by bi-axial bending of the columns are not considered. However, where bi-axial bending occurs, the flexural capacities about both axes of the member are required for the preliminary design. Bi-axial bending in the building columns is caused by the simultaneous action of the horizontal blast loads on two adjoining walls of the structure. Generally, to design for this condition, preliminary analyses of both longitudinal and transverse frames are necessary in order to compute the required flexural capacities about both axes of the columns. In cases involving normally directed shock waves, analyses of frames perpendicular to the direction of the blast can be eliminated since the sideways motions of these structures are prevented by the symmetric loading on the sides of the building. However, consideration must be given to the occurrence of bi-axial bending in the columns of the end frames parallel to the blast.

For the preliminary design of these columns, the flexural capacities required to resist the side-on pressures can be estimated by a simplified mechanism analysis of the individual members.

With the required flexural capacities determined on the basis of the mechanism analyses, the preliminary sizing of the frame members can be accomplished by applying the design criteria presented in Section 2.4.

The preliminary design procedure of this section is a simplification of the frame design method presented in Chapter 5 of Reference 1. The referenced material contains procedures for the preliminary selection of member sizes, as well as expressions for approximating the overall sideways response of the frame, and the individual responses of the exterior members of the frame. These latter procedures were included to provide facility designers with a means for evaluating the adequacy of the frame members selected on the basis of the preliminary design method. Based on experience with DYNFA, however, it has been determined that these latter procedures can be eliminated. Hence, the preliminary design procedure presented in this report is limited in scope to the initial selection of the member sizes for the main frames. In addition, the preliminary sizing of the frame members is simplified by the exclusion of the axial load and related beam column effects from the design criteria.

To facilitate the utilization of this report, the applicable equations and data presented in Chapter 5 of Reference 1 are reproduced in Sections 2.3 and 2.4 of this report. The reproduced material includes the expressions for the collapse mechanisms of single-story frames, the preliminary dynamic load factors for establishing the equivalent static loads for the various failure mechanisms considered and the moment capacity reduction equations for the effects of lateral torsional buckling.

Examples A.1 and A.2 of Appendix A demonstrate the use of the preliminary design method.

2.2 Computation of the Blast Pressures for Preliminary Design

The blast pressures utilized for the preliminary design of the main frames are:

1. The peak-reflected pressures acting on the blastward wall(s) of the structure, and
2. The peak pressure acting on the roof of the structure.

In situations involving quartering shock waves, the above quantities are computed at the corner of the building nearest the detonation and at the far corner of each blastward wall. The pressures thus obtained are then utilized to compare the blast loads acting on the frames positioned normal to each blastward wall. The peak pressures computed at the blastward corner of the building are utilized for the preliminary design of the blastward exterior frames and the interior frames adjacent to them. When analyses of additional frames are required because of large pressure differentials between the ends of the building, the peak reflected pressure utilized for the mechanism analysis of each such frame should be computed at the location of the frame and the roof pressure should be computed at a point halfway between the blastward and leeward ends of the frame. The occurrence of large pressure differentials between the ends of the building is prevalent in high pressure regions or situations involving relatively long structures.

For designing structures subjected to normal shock waves, the peak pressures are computed at the point on the blastward wall nearest the detonation, as shown in Figure 5.

The data for computing the blast pressures for the preliminary design are contained in Figures 4-5, 4-6, 4-11 and 4-12 of Reference 3. Figures 4-5, 4-11 and 4-12 contain curves of the incident and normal reflected pressures plotted as functions of the scaled distance $Z = R_A/W^{1/3}$, where R_A is the radial distance from the charge and W is the charge weight. The data in Figure 4-5 is applicable to TNT explosions in free air; while the data in Figures 4-11 and 4-12 applies to TNT surface explosions. Supplementary data for surface explosions is provided in Figure 6 of this report. To determine the value of a specific parameter, enter the appropriate figure with the scaled distance from the charge to the point of interest on the structure, and read the value from the appropriate curve.

For the preliminary design of frames located in the low-to-intermediate pressure level range (70.0 kPa or less), the peak pressure acting at a point on the roof can be taken as the peak positive incident pressure. This quantity can be read directly off of the appropriate curve in one of the aforementioned figures. For the design of structures located in higher pressure regions, the peak roof pressure should be taken as the sum of the peak incident pressure plus the drag pressure, which is computed as outlined in Section 4-14 of Reference 3.

When computing the reflected pressure at a point on a blastward wall, the angle of incidence between the direction of the blast wave and the wall must be considered. In cases where the

blast wave is normal to the wall, the peak reflected pressure can be read directly from the curves in Figure 4-5 or 4-12 of Reference 3. However, in situations involving quartering loads, supplementary data are required for the computation of the peak reflected pressure. These data are provided in Figure 4-6 of Reference 3. The referenced figure contains curves which relate the reflected pressure to both the incident pressure and the angle of incidence between the direction of the shock front and the wall.

To compute the reflected pressures in cases involving quartering loads, read from the appropriate curve in Figure 4-5, 4-11 or 4-12, the incident pressure at the point of interest on the structure. Next, compute the angle of incidence between the shock front and the blastward wall at the point of interest. Enter Figure 4-6 with these two quantities and read from the appropriate curve the value of the reflected pressure coefficient, which is defined as the ratio of the peak reflected to the peak incident pressure. Multiplication of the peak incident pressure by this coefficient will yield the peak reflected pressure.

2.3 Computation of Plastic Moment Capacities

General

The plastic moment capacities of the members are determined by establishing the governing failure mode for the frame. The design objective is to proportion the frame members such that the governing mechanism represents an economical solution.

Rigid Frames

General expressions for the possible collapse mechanisms of single-story rigid frames are presented in Table 1 for pinned and fixed-base frames subjected to combined vertical and horizontal loading.

For a particular frame within a framing system, the ratio of total horizontal-to-vertical peak loading (denoted by α) is dependent upon the following: the values of the peak pressures computed as discussed in Section 2.2, and the configuration of the secondary framing of the structure. Hence, the quantity α is determined as follows:

$$\alpha = q_h/q_v \quad (1)$$

where $q_v = p_v b_v =$ peak vertical load on rigid frame

$q_h = p_h b_h$ = peak horizontal load on rigid frame

p_v = peak pressure on roof

p_h = peak reflected pressure on
blastward wall

b_v = tributary width for vertical loading
on the roof girder

= tributary width of building supported
by frame when the purlins are normal
to the girder, or one-half of the sum
of the distances between the girder and
the two adjacent purlins, when the
purlins are parallel to the girder

b_h = tributary width for horizontal loading
on the exterior column of the blast-
ward wall

= tributary width of building supported
by frame when the girts are normal to
the column, or one-half of the sum of
the distances between the column and the
two adjacent girts, when the girts are
parallel to the column

The ratio, α , will usually lie in the range from about 1.8 to 2.5 when the direction of the blast wave propagation is perpendicular to the roof purlins. The value of α is much higher when the direction of the blast wave is parallel to the purlins, since in this case, only part of the vertical load is carried by the girders of the frame.

The following assumptions were made in developing the expressions given in Table 1:

1. The plastic bending capacity of the roof girder, M_p , is constant for all bays.
2. Both exterior columns have the same plastic bending capacity, CM_p . It was also assumed that the bending capacity of an exterior column is equal to or greater than the bending capacity of the girder (hence, $C \geq 1.0$).
3. All interior columns have the same plastic bending capacity, $C_1 M_p$.

In most cases, more economical designs of blast-resistant frames are realized when the failure mode corresponds to either a panel or combined mechanism (Mechanisms 3 through 6 in Table 1). It will normally be uneconomical to proportion a rigid frame such that the mode of failure for the structure corresponds to a simple beam mechanism (Mechanisms 1 and 2 in Table 1). Such a failure results from a localized failure of either the roof girder or the columns and therefore may occur while the remaining frame members remain elastic. Failure due to the formation of simple beam mechanisms is prevalent in laterally restrained frames where the lateral restraint is provided by a rigid support member connected to the frame or the application of equal, but opposite, horizontal loads to the exterior columns. The latter condition applies to frames which are positioned perpendicular to the direction of the blast wave. When the lateral restraint is provided by an elastic member (such as a diagonal brace), failure may occur due to other types of mechanisms (panel, combined, etc.) as discussed in subsequent sections.

The mechanism method is based on the upper-bound plastic limit theorem (Ref 6) which states that a load determined for an assumed mechanism will always be greater than or equal to the true plastic limit load for the structure. Thus, for a given frame with known member properties, the true plastic limit load is the minimum of the plastic limit loads computed for all of the possible collapse mechanisms, and the governing collapse mechanism is the one corresponding to the true plastic limit load. In design, the applied load is fixed while the member sizes are unknown. Applying the upper-bound theorem for design, the governing mechanism corresponds to the one requiring the largest plastic moment, M_p . For economical design, the designer should attempt to proportion the bending capacities of the columns and girders such that the governing mechanism is either a panel or combined mechanism. This is accomplished by several trial calculations in which the values of C and C_1 , and the corresponding maximum value of M_p computed for all mechanisms, are minimized in successive trials. Here, engineering judgement is required, since several sets of values of C and C_1 may yield similar values of M_p . Based on experience with the mechanism method, the following values of C and C_1 are recommended for initiating the trial calculation:

1. $C \geq 2.0$
2. $C_1 = 2.0$

After a few trials, it will become obvious which choices of C and C_1 should be utilized. In most cases, the bending capacity of an

exterior column will be greater than the bending capacity of an interior column; hence, when choosing values for C and C_1 , C should generally be greater than or equal to C_1 . However, in some situations, this may not achieve an economical solution; here, economy may dictate that C should be less than equal to C_1 .

Frames with Supplementary Bracing

The possible collapse mechanisms of single-story frames with diagonal tension bracing are given in Tables 2 and 3 for pinned-base frames with rigid and non-rigid girder to column connections. In each case, the ultimate capacity is expressed in terms of the equivalent static load and the member ultimate strength (either M or $A_b F_{dy}$). In these tables, the cross-sectional area of the tension brace is denoted by A_b , the parameter m is the number of braced bays, and F_{dy} is the dynamic yield stress for the bracing member. The dynamic yield stress is determined according to the provisions in Section 2.2 of Reference 1. The derivations of the expression given in Tables 2 and 3 are based on the same assumptions utilized in developing the expressions given in Table 1.

For non-rigid girder-to-column connections, the resistance functions for local mechanisms of the roof girder depend upon whether the girder is continuous over the columns or is framed between the columns. When the girder is continuous, the interior bay girder mechanisms are the same as those for a fixed beam; while the resistance of the exterior girder is the same as that of a fixed-pinned beam. In certain cases, it may be economical to provide a rigid connection at the exterior girder-to-column joint and non-rigid connections at interior girder-to-column joints.

For rigid frames with tension bracing, it is necessary to vary C , C_1 and A_b in order to achieve an economical design. When non-rigid girder-to-column connections are used, C and C_1 drop out of the resistance function for the sidesway mechanism and the area of the bracing can be calculated directly.

The preliminary design procedure for frames with supplementary diagonal braces is similar to the procedure described for rigid frames. For braced frames with rigid connections, however, the procedure is slightly more involved since it is necessary to assume a value for the brace area in addition to the assumptions for the coefficients C and C_1 . In selecting a trial value for A_b for frames with rigid connections, the minimum brace size will be controlled by limiting the slenderness ratio for the member, in order to prevent vibration or "slapping" during the response. This design condition can be expressed as:

$$r_b \geq L_b/300 \quad (2)$$

in which r_b is the minimum radius of gyration of the bracing member and L_b is its length between points of support. Even though a compression brace is not considered effective in providing resistance, the tension and compression braces should be connected together where they cross. In this manner, L_b for each brace may be taken equal to half of the total length.

In addition, in each particular application, there will be a limiting value of A_b beyond which there will be no substantial weight savings in the frame members since minimum sizes for the frame members are required based upon the maximum slenderness ratio requirements given in Section 4.2 of Reference 1. In general, values of A_b of about 6 to 25 square centimeters (1 to 2 square inches) will result in a substantial increase in the overall resistance for frames with rigid connections; hence, an assumed brace area in this range is recommended as a starting point. The determination of C and C_1 then follows the same procedure as outlined for rigid frames.

Dynamic Load Factors

For the purpose of preliminary design, it is necessary to make certain initial assumptions regarding the dynamic effect of the load on the deflection of the frame. These assumptions are required since the natural period of the system is initially unknown. To obtain initial estimates of the required mechanism resistance, the dynamic load factors of Table 4 may be used to obtain equivalent static loads for the indicated mechanisms. These factors are based, in part, on the data provided in Chapter 5 of Reference 1.

2.4 Preliminary Sizing of Frame Members

Member sizes are selected on the basis of the plastic section moduli. These quantities are computed using the required bending capacities determined from the mechanism analyses of several representative frames spanning in one or both directions of the building. The member sizes thus obtained must satisfy the provisions for the design of members subjected to combined axial load and bending moment that are set forth in Reference 1 and adapted, with some modifications, in this report.

The plastic section moduli for a member designed for bi-axial bending are computed with the following equations:

$$Z_x = (M_p)_x / F_{dy} \quad (3a)$$

$$Z_y = (M_p)_y / F_{dy} \quad (3b)$$

where Z_x, Z_y = the plastic section moduli for both axes of bending

$(M_p)_x, (M_p)_y$ = the required plastic bending capacities about both axes of bending

F_{dy} = the dynamic yield stress which is computed as described in Section 2.2 of Reference 1

Select a member whose x- and y-axes plastic section moduli are equal to or greater than Z_x and Z_y , respectively. The plastic section modulus for a member designed for uni-axial bending is computed with the equation given below:

$$Z_n = (M_p)_n / F_{dy} \quad (4)$$

where the subscript n refers to the axis of bending of the member.

The provision set forth in Reference 1 for the design of members subjected to combined axial load and bending moment comprise the following requirements:

1. Members subjected to combined axial load and bi-axial bending moments should be proportioned to satisfy the interaction formulas which consist of Equations 4.4 and 4.5 of Reference 1.
2. Members subjected to bending about their strong axis should be provided, where possible, with sufficient lateral bracing to prevent lateral-torsional buckling. Lateral bracing requirements are specified in Section 3.3.6 of Reference 1. Where the placement of lateral bracing is not possible, as in the case of the building columns, reductions in the moment capacities of members, unbraced in the weak direction, must be effected in order to account for the effects of lateral torsional buckling. These reductions are made on the basis of Equation 4.7 of Reference 1 which is repeated on the following page.

$$M_{mx} = [1.07 - (z/r_y)\sqrt{F_{dy}/262394}]M_{px} \leq M_{px} \quad (5a)$$

$$M_{my} = [1.07 - (z/r_x)\sqrt{F_{dy}/262394}]M_{py} \leq M_{py} \quad (5b)$$

where M_m = the ultimate bending capacity
in the absence of axial load

M_p = the design plastic bending capacity

$$= F_{dy}Z$$

Z = the plastic section modulus

F_{dy} = the dynamic yield stress. In the
above equations, the units for the
dynamic yield stress are kilopascals.

z = unbraced length of the member

r = radius of gyration

These reduced capacities are used in the interaction formulas
[Eq (4.4) and (4.5)] of Reference 1.

3. The member must satisfy the overall stability criteria as specified by the maximum allowable slenderness ratio (in the plane of bending) computed according to Equation 4.1 of Reference 1.

The overall stability criteria must be satisfied for both axes of bending when a member is designed for bi-axial bending.

4. The member must satisfy the local stability criteria specified in Section 3.3.4 of Reference 1.

These criteria are expressed in terms of the following quantities:

- a. Limiting width-thickness ratios for the flanges of I- and W-shapes (and similar built-up single-web shapes) that are subjected to compression involving plastic hinge rotation. To facilitate the utilization of the SI System of Units, these criteria are reproduced in Table 5 with the appropriate quantities expressed in SI units.

- b. Limiting values (as given by Equations 3.12 and 3.13 of Reference 1) of the depth-thickness ratios for the webs of members subjected to plastic bending.

Based on experience with DYNFA, it has been determined that considerations related to the simultaneous effects of axial loads and bi-axial bending can be eliminated from the preliminary design. Hence, some portions of the above criteria can be simplified for the preliminary sizing of the frame members. First, the interaction formulas of Reference 1 (Item 1 above) can be replaced with the simplified equations given below:

$$(M_p)_x/M_{mx} \leq 1.1 \quad (6a)$$

$$(M_p)_y/M_{my} \leq 1.1 \quad (6b)$$

The design of members subjected to bending about one axis is governed by the uni-axial bending equation given below:

$$(M_p)_n/M_{mn} \leq 1.1 \quad (7)$$

where the subscript n refers to the axis of bending of the member.

The design requirements specified in Items 2 and 3 above remain unchanged and should be applied as described in Reference 1. The design requirements referred to in Item 4 also remain unchanged; however, the equations given in Reference 1 (Equations 3.12 and 3.13) which govern the minimum depth-thickness ratios of the webs of members subjected to plastic bending, are replaced by the following equation:

$$d/t_w \leq 29,060\sqrt{F_y} \quad (8)$$

Here, F_y is specified in kilopascals, d is the total depth of the member and t_w is the web thickness.

SECTION 3

BASIS OF DYNFA FRAME ANALYSIS

3.1 Introduction

This section presents the analytical techniques utilized by DYNFA to perform rigorous dynamic analyses of frame structures subjected to the blast loads from a high explosive detonation. The presentation here is primarily intended to provide facility designers with background material which describes the basis of the dynamic analysis as performed by DYNFA. As such, a complete understanding of most of this material, while desirable, is not mandatory for using DYNFA. However, it is necessary that the concept of the analytical model, as described in this section, be fully understood, as it provides the basis for the material related to the use of DYNFA.

The response of a frame to the blast loads is computed using an approach which couples a lumped parameter representation of the structure with a numerical computation procedure in which the equations of motion of the system are integrated directly using the linear acceleration method. Inelastic behavior of individual members is introduced into the analysis by the formation of concentrated plastic hinges whenever the combined axial load and bending moment capacity of a section are reached. The results of the analysis consist of the deformations of the structure and the axial loads, bending moments and shears in each of the members. End rotations of each member are computed and utilized to monitor the amount of plastic deformation occurring in the structure. The DYNFA computer program implements this analytical procedure. The details of the mathematical techniques utilized in the DYNFA analysis are provided in Appendix C.

A static analysis routine was incorporated into DYNFA in order to determine the initial conditions of the structure under the effects of dead and live loads. Normally, in single-story frame buildings, the magnitudes of the dead and live loads are usually small compared to the magnitude of the blast load. In multi-story structures, however, the dead and live loads could be significant.

3.2 Analytical Model

The essential step required for the rigorous dynamic analysis is the formulation of an analytical model of the structure. The model utilized to compute the response of frame structures subjected

to blast loads must satisfy three basic requirements. First, it must accurately reflect the configuration of the structure under consideration. Next, the elastic and inelastic behavior of the structure must be duplicated in the model. Third, the distribution of the mass of the structure, as well as the loads applied to it, must be reproduced in the model such that all of the desired responses are obtained in the analysis. Of equal importance is the requirement that the modeling technique employed in the analysis reduces the problem to a simpler form which readily lends itself to a solution.

In order to satisfy all of these requirements, the lumped parameter method for modeling complex structures is utilized in the analysis. This method greatly simplifies the analysis but still retains the capability for an accurate determination of the desired structural responses. For the problem at hand, the desired responses consist of the overall sidesway response of the frame and the individual responses of the exterior members of the frame.

The lumped parameter method enjoys extensive use in a wide range of applications throughout the industry. The principal reason for this is the ease with which it can be interfaced with high speed computers to obtain a solution. A lumped parameter model of a structure consists of an assemblage of massless structural elements interconnected at nodal points. An example of a typical model is shown in Figure 7. The mass of the structure, as well as the applied loads, are assumed to be concentrated at selected nodal points which are referred to as "mass points". Rotary inertia is not included in the analysis.

As discussed in Section 1, each frame of the building is analyzed individually using a two-dimensional representation of the structural system. In a two-dimensional model, the structure is limited to motions in one plane and therefore each nodal point has three degrees of freedom; namely, horizontal translation (D_H), vertical translation (D_V), and rotation (θ). These quantities are illustrated in Figure 7.

In the model, an individual member of the frame is represented by one or more structural elements, depending upon the detail required. The structural element is capable of transmitting axial loads as well as shears and bending moments.

3.3 Inelastic Behavior of Frame Members

The inelastic behavior of the frame members is treated in the analysis by:

1. Specifying the behavior of a member in the inelastic response range by defining the element's response (axial loads and bending moments) in terms of the element's elastic and plastic deformations. With the element behavior specified, an analytical technique is developed to simulate the desired inelastic behavior in the analysis.
2. An interaction equation which establishes the point at which inelastic behavior commences for each element.
3. An analytical method which accommodates the variations in the stiffness properties of the structural system that result from the inelastic behavior of the members.

Items 1 and 2 are discussed in this section and Item 3 is covered in the discussion pertaining to the formulation of the system stiffness matrix (Section 3.4).

Specification and Simulation of Inelastic Behavior of Frame Members

The specification of the inelastic behavior of the frame members is based on the assumption that a fully plastic section offers no additional resistance to the load (Ref 6). Hence, the desired behavior of the elements is represented by the bilinear hysteresis loop shown in Figure 8. The figure depicts the bending moment versus end rotation relationship (in the absence of axial load) for a typical element. A similar relationship exists between the axial load and axial deformations. The portion of the loop between points A and B represents the elastic behavior of the element when initially loaded. Once point B is reached, yielding of the element commences (plastic hinge forms) and continues until a maximum curvature at point C is reached. From point C to point D, the element unloads. The linear segment of the loop from C to D (and eventually to E) has the same slope as the initial loading segment (from A to B). The portion of the loop below the end rotation axis represents the load reversal (or rebound) in the element.

In single-degree-of-freedom analyses of individual members, the behavior described above can be readily accommodated by simply adjusting the resistance for the member such that a constant moment, and therefore a constant resistance, is designated when yielding commences. Ideally, it would be desirable if this same procedure could be made applicable to the multi-degree-of-freedom analysis of a frame structure by introducing plastic hinges at those sections in the structure which have yielded. However, the

use of this procedure may result in the formation of unstable members and/or substructures within the structural system. Such effects cannot be treated analytically as they produce mathematical anomalies in the solution. In order to avoid these problems, an alternate method is utilized to simulate the inelastic behavior. The method consists of subdividing the element stiffness into two components which are:

1. An elastoplastic component, and
2. An infinitely elastic component.

Such an element has been examined in References 7 through 9. The two components act in parallel and the total stiffness of the elements is the summation of the factored stiffnesses of the two components. The factors applied in DYNFA are 0.95 for the elastoplastic component and 0.05 for the elastic component. The composite behavior of the two components is illustrated in Figure 9a, where the behavior shown is similar to the hysteresis loop of Figure 8 with the exception that the magnitude of the moment varies in the inelastic portions of the response (Segments B-C and E-F). The individual behavior of the two components is illustrated in Figure 9b. As shown in the figure, the stiffness of the infinitely elastic component, k_{ie} , remains constant throughout the response; while the stiffness of the elastoplastic component, k_{ep} , depends upon the yield condition at the end of the element. From point A to point B in Figure 9a, the total stiffness of the element, k_c , equals the sum of the two component stiffnesses, k_{ie} and k_{ep} (point a to points b and b', Fig 9).

As yielding commences at point B (which corresponds to points b and b' in Fig 9b), the stiffness of the elastoplastic component becomes zero as illustrated by line bc (Fig 9b); while the stiffness of the elastic component remains constant at a value of $0.05k_c$, as illustrated by line b'c'. Hence, the total stiffness of the element reduces to a small percentage ($0.05k_c$) of its actual value, as illustrated by line B-C in Figure 9a. Appendix C contains a table of equations for the combined element stiffnesses for the various yield conditions at the ends of an element.

Interaction Equation

In blast-resistant design, structures must withstand the simultaneous application of horizontal and vertical loads of roughly equal magnitude; therefore, the members of the structure will be subjected to significant axial loads which limit their ultimate bending resistances. In addition, the axial loads will be time dependent and, for a given member, this time dependency will not

be related to the transient behavior of the member in its bending mode. In order to properly account for the axial load/bending moment interaction on the yielding of a member, a two-dimensional yield criterion is utilized to define inelastic action. A member is considered to have yielded at a section when the following relationship has been satisfied:

$$|P/P_c| + |M/M_m| \geq 1 \quad (9)$$

In the preceding equation, the values of P and M are the applied axial load and bending moment, respectively; while P_c and M_m are equal to the axial load and bending moment capacity, respectively. P_c is equal to either the axial load at yielding (P_p) when the member is in tension or the ultimate buckling load (P_u) when the member is in compression. The bending moment and axial load capacities are computed on the basis of the criteria specified in Chapter 4 of Reference 1 and reproduced in Section 4 of this report.

In the inelastic response range, the axial loads and bending moments of the modeling elements vary as illustrated in Figure 10. The occurrence of these variations stems from the utilization of the two-component approximation to the element behavior as described in the preceding section. Because of this, the peak plastic deformations may not be readily discerned from the time history of what may be considered a significant response parameter, such as the element bending moment or end rotation. Therefore, an algorithm is required to perform the following operations, namely: (1) to determine the maximum plastic deformations occurring at the yielded end of an element and, in conjunction with this, (2) to detect the point at which the element commences to unload elastically. The occurrence of these events is detected in the analysis with the use of the following relationship:

$$\frac{P_2(P_2 - P_1)}{|P_2|(P_c)} + \frac{M_2(M_2 - M_1)}{|M_2|(M_m)} < 0 \quad (10)$$

The maximum plastic deformation at the yielded end of an element is assumed to have occurred when the above relationship is satisfied. The parameters P, M, P_c and M_m in Equation (10) are defined in the preceding paragraphs. The subscripts 1 and 2 refer to the values of the parameters at two successive time stations in the response of the structure.

The above equation is utilized in the following manner. In the inelastic range, the element axial load and bending moment continue to increase at a relatively slow pace until a maximum

plastic deformation is attained. After this occurs, the element axial load/moment will decrease producing negative values for one or both of the ratios in Equation 10. When the sum of the two ratios becomes a negative quantity, the element is considered to rebound elastically.

3.4 Equation of Motion of System

The general equations of motion of the system are:

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{u\} = \{F(t)\} \quad (11)$$

where $\{u\}$, $\{\dot{u}\}$, $\{\ddot{u}\}$ = displacements, velocities and accelerations of the nodal points of the analytical model

$[M]$ = mass matrix of the system

$[C]$ = damping matrix of the system

$[K]$ = stiffness matrix of the system

$\{F(t)\}$ = matrix of the transient loads applied to the system

Mass Matrix

The mass matrix consists of discrete masses, the sum of which adds up to the total mass of the structure. These discrete masses are concentrated at selected nodal points in a manner which is consistent with the actual distribution of mass in the structure. A complete discussion of the techniques employed in assigning mass to the nodal points is provided in Section 4.

Damping Matrix

The damping matrix represents the internal energy absorption properties of the structure. For purposes of analysis, the damping matrix is usually assumed to be proportional to either the stiffness or mass matrices. In DYNFA, the damping matrix is taken as proportional to the mass matrix.

In most analyses involving steel structures, damping can be neglected; however, provisions were made in DYNFA for including this effect for special applications; e.g., reinforced concrete frames.

System Stiffness Matrix

The system stiffness matrix is an array of coefficients of the unknown displacements in a series of simultaneous equations which express the applied loads as functions of the deflections of the structure. It is generated by applying conventional methods of matrix structural analysis such as those utilized in Reference 11. Briefly, the system stiffness matrix is composed of the stiffness matrices of the individual elements of the model. For a beam element with three degrees of freedom at each end, the entries in the element stiffness matrix are the coefficients of the displacement variables in six simultaneous equations which relate forces and moments to the displacements and rotations at the ends of the element. Shear deformations are not included in the formulation. The system stiffness matrix is constructed by superimposing, in their proper position in the matrix (according to degree of freedom), the coefficients of the element stiffnesses.

In the analysis, the structure is assumed to respond linearly during a given time interval. However, the coefficients in the system stiffness matrix may be changed from one interval to the next, depending upon the yield conditions at the ends of the elements. Therefore, the non-linear response is obtained by sequencing the linear responses of the system (with varying stiffness coefficients) at successive time intervals.

Matrix of Applied Loads

The matrix of the applied loads contains the time history of the blast loads which are concentrated at the mass points on the exterior members of the structure. Included also are the unbalanced shears caused by the second order effects which occur as the structure responds to the blast loads. A description of these effects is provided in Section 3.5.

Numerical Solution

The solution to the equations of motion is obtained by a step-by-step integration procedure (Ref 8) in which the acceleration during a time interval is assumed to vary linearly.

3.5 Second Order Effects

Two important aspects of the problem which have been incorporated into the analytical procedure and computer program are: the P- Δ effect on the overall sidesway response of the frame, and the beam column effects on the responses of the individual members subjected to transverse loads between their supports.

P-Δ Effect

For a rigid frame structure subjected to lateral loads, the vertical loads combine with the sidesway deflection to produce additional bending moments in the members. For elastic structures, the deflections are relatively small; therefore, this effect, commonly called "the P-Δ effect", is generally of secondary significance in comparison with the flexural resistance of the structural members. Inelastic structures, however, may undergo relatively large sidesway deflections. These large deflections combined with the large vertical loads prevalent in blast design, produce bending moments which must be considered in the analysis. The P-Δ effect is included in the analysis primarily because of its influence on the sidesway response of the frame.

Beam Column Effect

When a member is subjected to both transverse and axial loads, the axial loads combine with the bending deflections to produce additional moments. In blast design, these "second order" bending moments cannot be neglected, as large deflections are expected when the member responds to the transverse load. The beam column effect is considered because of its influence on the individual bending responses of the exterior members of the frame.

Calculation of Second Order Effects

A rigorous treatment of these second order effects would significantly increase the complexity of the frame analysis. However, both of these effects can be approximated by introducing equivalent shears at both ends of each element (Ref 10). These shears form a couple which is equal in magnitude but opposite in direction to the secondary moments produced by the axial loads combined with the differential displacements between the ends of the element. Figure 11 illustrates the calculation of these equivalent shears. However, it should be noted that the procedure depicted in this figure is a simplification of the actual computation. The figure was included for illustrative purposes only. The actual computation of the equivalent shears, together with the mathematics involved in introducing these quantities into the matrix of the applied loads, are presented in Appendix C.

SECTION 4

MODELING TECHNIQUES

4.1 Introduction

The initial task in the analysis is the formulation of the analytical model of the frame under consideration. When constructing the model of the structure, the basic requirements of Section 3.2 must be satisfied in order to achieve the desired degree of accuracy and detail in the analysis.

Briefly, the requirements of Section 3.2 dictate that the model must accurately reflect the configuration, stiffness characteristics, and mass distribution of the structure. To assist facility designers in the preparation of analytical models which satisfy these requirements, modeling techniques and guidelines are presented in this section and in Section 5. The discussion is directed primarily towards the preparation of lumped parameter representations of frame structures for use in their analyses with DYNFA.

The lumped parameter representation of a frame structure is composed of the following: (1) an analytical model of the structure consisting of an assemblage of beam elements interconnected at nodal points, (2) a series of discrete masses assigned to selected nodal points in the model, (3) the section properties (area, moment of inertia) and capacities (axial load capacities, flexural capacity) of the members of the structure, and (4) idealizations of the transient loadings acting on the structure. The discussion in this section is directed towards the preparation of data related to the first three items. Procedures for generating loading functions for the analysis are presented in Section 5.

4.2 Development of Model

General

The systems under consideration in this report are the primary structural frames of rectangular steel buildings. For the analysis to produce the desired response quantities, (namely, displacements, member end rotations, column loads, etc.), the model of a frame must have the same configuration as the frame itself. As shown in Figure 7, the actual model is a line diagram of the structure under consideration. The diagram reflects the configuration of the structure and shows the placement of the nodal points in the model. All of the frame members are included in the model.

Models of braced frames include the diagonal braces as well as the columns and girders.

In the model, the members of the frame are represented by one or more beam elements. The section properties and capacities of the beam elements are identical to those of the frame members. However, as depicted in Figure 7, the beam elements are one-dimensional; that is, they have length but neither width nor depth. This limitation is generally not significant in analyses of steel frames, as the length-to-depth ratios of the frame members are generally quite large.

Since the modeling elements are one-dimensional, all of the significant dimensions (length, height, locations of nodal points, etc.) in the model must be related to the longitudinal axes of the frame members, which are located at the centroids of their cross-sections. The intersection of two members is generally taken as the intersection of their respective longitudinal axes.

A basic assumption in the analytical formulation of DYNFA is that structural continuity exists at all nodal points in the model, except at those nodes corresponding to support points. However, provision is made in DYNFA for modeling structural discontinuities, such as non-rigid girder to column connections. Modeling of such discontinuities is accomplished by pinning the ends of the appropriate elements (see Section 8.7). This provision is useful for modeling braced frames, where the ends of the diagonal braces are assumed to be pin-connected to the frame. Such a condition is modeled by pinning both ends of the element representing each diagonal brace.

Scope of Model

In general, the model used to compute the blast-induced response of frame structures includes the principal components (girders, columns and diagonal braces) of the main frame only. Except for their mass and load contribution, the secondary framing members of the roof and walls are not included in the model.

A basic assumption in the frame analysis is that the motions of the concrete foundation on the supporting soil have a negligible effect on the peak responses of the frame. Based on this, the foundation/soil systems are excluded from the model, and the nodal points, corresponding to the column foundation connections, are taken as completely fixed against horizontal and vertical translation. If these connections are capable of transmitting moments, the rotations of the column base nodal points are also fixed. A model such as this produces a conservative estimate of the frame's response.

To justify the exclusion of the foundation/soil systems, the foundation must have sufficient mass to react to the dynamic loads transmitted to the base of the frame. A foundation mass exceeding twice the mass of the steel superstructure will suffice for the task at hand. When an insufficiency of foundation mass exists, a significant interaction may occur between the sideways response of the frame and the rigid body response of the combined steel superstructure/foundation system. Since this interaction may appreciably reduce the frame response, it is advisable in these situations to extend the model such that the responses of the foundation/soil systems are included in the analysis. Figure 12 illustrates this modeling extension. The model contains elements representing both the foundation and the soil system. Not shown, however, are the discrete masses concentrated at selected nodal points on the foundation. Only the foundation mass is distributed among these nodal points. Soil mass is not included in the model. The reader is referred to Sections 5-7 and 5-8 of Reference 3 for guidance in computing the elastic properties of concrete foundations. Procedures and data for computing the stiffness properties of the soil are provided in Section 4 of Reference 12.

In some cases, additional supporting systems may be integral with the main framing. A typical example of this situation is shown in Figure 13. In this structure, the main frames are laterally supported by a massive concrete wall. The degree of support provided is determined by comparing the lateral stiffness of the wall with that of the frame. A wall whose lateral stiffness is at least ten times that of the frame is simulated analytically by restraining the nodal point at the girder-to-wall connection. On the other hand, a relatively flexible wall may have to be included in the model as its response would affect the frame response.

Placement of Nodal Points in the Model

Nodal points are usually positioned in the model at the following locations:

1. The intersection of two or more frame members or the connections of frame members to a supporting structure (such as the foundation).
2. Intermediate points on the exterior members to accommodate mass points.
3. Intermediate points on exterior members, other than mass points, where additional response information (deflect, bending moments, etc.) are required.

4. Intermediate points on members with gradual or abrupt variations in shape, or at the locations of structural discontinuities.

The first item requires no explanation, while the other three are discussed below.

Intermediate Nodal Points on Exterior Members

A structure has, in reality, an infinite number of normal modes of vibration (Ref 13) because of the uniform distribution of the structural mass along the individual members. In a frame analysis, however, the mass of the structure is concentrated at specified nodal points in the model (commonly referred to as "mass points"). Consequently, the model has a limited number of the structure's normal modes of vibration. The nature of these normal modes of vibration that are reproduced in the model is determined by the manner in which the mass is distributed among the mass points. The inclusion of specific modes in the model is dictated by the nature of the structural response to the applied loads. Under the action of the blast, the frame responds to the horizontal blast loads in a sidesway mode. At the same time, the frame responds in a series of local bending modes due to the transverse loads acting directly on the exterior members. These local bending modes are often characterized by pronounced bending of only one or two exterior members. In most cases, the primary response of an exterior member occurs in one of these local modes.

The fundamental sidesway mode of the frame can be produced by concentrating (also referred to as "lumping") all of the mass of the structure at the girder/column intersections. The modeling detail required to accomplish this is illustrated in Figure 14a. However, such a model lacks sufficient detail to produce the individual bending responses of the exterior members. To develop these responses, the distribution of mass in the model must be refined by lumping a portion of the structural mass at intermediate points on the exterior members. To accommodate this refinement, intermediate nodal points are generally placed along the exterior members, as shown in Figure 14b. In general, multiple intermediate mass points are utilized as a more accurate representation of the local bending modes of the frame will be reproduced in the model. The number of such points utilized depends, to some extent, on the secondary framing of the exterior surfaces. When the secondary members of a given exterior surface (wall or roof) are parallel to the frame, use three intermediate mass points within the span of each frame member supporting the surface. Locate these mass points at equal intervals along each member. In cases where the

secondary members are positioned normal to the frame, use either one of the following procedures:

1. When three or less secondary members are supported within the span of a frame member, locate the intermediate mass points at the connections of the secondary members to the frame proper.
2. When more than three secondary members are supported within the span of a frame member, locate three intermediate mass points at equal intervals along the member.

The utilization of two or three intermediate mass points on a member is sufficient to achieve the desired accuracy in the analysis. A greater number will greatly increase the computer costs without significantly improving the accuracy of the results. It is also recommended that, when possible, the same number of mass points be utilized on all exterior members in order to avoid gross discontinuities in the application of the blast loads in the analysis.

Intermediate Nodal Points Other Than Mass Points

In some cases, additional response data (displacements, rotations, bending moments, etc.) may be required at intermediate locations other than mass points on both interior and exterior members. In order to generate this data in the analysis, additional nodal points are placed at the locations in the model where these data are required. Additional nodal points are always required in those cases where there are no intermediate mass points located at the midspan of an exterior member. In such cases, the analysis will not generate data regarding the member's response at the midspan. Such data are required in order to produce a better approximation of the beam column effects in the analysis (see Section 3.5), as well as to ascertain whether the deflections and ductility ratios at the midspan are within the limits specified (Section 6) for the frame design; hence, a nodal point is required and should be placed at the midspan of each exterior member. The addition of this nodal point in no way implies that it must be assigned a concentrated mass. Such a situation occurs in Example A.3 in which the intermediate mass points on the exterior members are located at the intersections of the secondary members with the primary frame members.

Members with Varying Cross-Sections

In the analysis, the section properties and capacities of the elements are constant over their entire lengths. Therefore,

members with varying cross-sectional properties must be modeled with several elements and consequently, nodal points must be inserted between the ends of these members.

Two examples of members with varying cross-sectional properties are illustrated in Figure 15. The member shown in Figure 15a has a gradual taper and is symmetric about its centerline. Hence, it is modeled with four elements of equal length: two elements represent each half of the member. The member shown in Figure 15b has an abrupt variation in shape due to the presence of the haunch at the left end, and a structural discontinuity caused by the pin near the right end. In this case, three elements of unequal lengths are used and two intermediate nodal points are placed at the locations of the discontinuities. When abrupt discontinuities such as these occur on exterior members, intermediate mass points are first placed at equal intervals along the span in order to accommodate the desired distribution of the mass, and then additional nodal points are placed at the locations of the discontinuities. Hence, if three equally spaced intermediate mass points were desired on a member with two abrupt discontinuities, such as those shown in Figure 15b, a total of five intermediate nodal points would be placed in the span of the member: three to accommodate the mass points and two for the discontinuities.

4.3 Distribution of Structural Mass in Model

General

With the model completely defined in terms of beam elements and nodal points, the data preparation proceeds with the distribution of the mass among the nodal points of the model. Before the mass can be distributed, however, the locations of the dynamic degrees of freedom must be specified for the model.

Designation of Dynamic Degrees of Freedom

When the frame responds to the blast loads, horizontal and vertical inertial forces are developed by the accelerating mass of the structure. In order to reproduce these forces in the dynamic analysis, each discrete mass of the model must be assigned a degree of freedom (either horizontal or vertical) as well as a magnitude. Therefore, two sets of discrete masses are required: one for the horizontal degrees of freedom and one for the vertical degrees of freedom.

In the model, two distinct masses can be lumped at each mass point. Each lumped mass is associated with one of the translational degrees of freedom of the mass point. Degrees of freedom with assigned masses are generally referred to as "dynamic or

independent degrees of freedom"; whereas, those without mass are known as "dependent degrees of freedom".

The designation of the dynamic degrees of freedom for the model is governed by the nature of the desired responses. To develop the primary sideways response of the frame, each nodal point corresponding to a girders/column intersection is assigned horizontal and vertical dynamic degrees of freedom. In gabled frames, the nodal point at the peak in the roof is also assigned a pair of dynamic degrees of freedom.

Intermediate mass points on exterior members are allotted one dynamic degree of freedom. The direction of these degrees of freedom is always normal to the longitudinal axis of the member. On inclined members, with slopes exceeding ten degrees, each intermediate mass point is assigned two dynamic degrees of freedom.

The responses of the higher order extensional modes of the frame members should not be included in the analysis as they have a negligible effect on the frame response. These modes are developed by including intermediate dynamic degrees of freedom parallel to the longitudinal axes of the members. The inclusion of these dynamic degrees of freedom, although simplifying the mass distribution for the model, adds significantly to the computer costs incurred and could possibly cause numerical inaccuracies in the analysis.

Figure 16 shows the distribution of the dynamic degrees of freedom for two typical frame models, where the directions of the dynamic degrees of freedom are represented by arrows.

Computation of Nodal Masses

As discussed in Section 1, each frame is assumed to support a tributary strip of the building. In the analysis, the structural mass within this strip is distributed among the mass points of the model. Live loads are generally not included in the mass computation.

The computation of the nodal masses is accomplished in three stages. The first stage involves computing the masses of individual panels of the walls, roof or intermediate floors (if any) within the tributary strip. For this computation, a wall panel is defined as the area between successive floor levels; whereas a floor or roof panel is defined by the area between successive columns of the frame. In general, the mass of a panel comprises the mass of the following structural components within the panel:

the decking or siding, the secondary members and the primary frame member. The mass of the secondary framing is not included as part of the panel mass in cases involving a coarse secondary framing consisting of three or less members positioned normal to the frame. Instead, the mass of each secondary member is concentrated at the mass point on the model corresponding to its intersection with the frame. The masses of primary transverse framing members are also not included as part of the panel mass; instead, they are concentrated at the mass points on the model corresponding to the intersections of orthogonal frames with the frame being analyzed.

In the second stage, the mass of each panel is distributed as concentrated masses among the dynamic degrees of freedom on the primary frame member within the panel. General expressions for computing the values of the concentrated masses are provided in Figure 17 for wall panels, and Figure 19 for roof and floor panels. The parameters utilized in Figures 17 and 19 for computing the concentrated masses are illustrated in Figures 18 and 20, respectively.

As shown in Figures 17 and 19, two different distributions of the panel mass are given: one for dynamic degrees of freedom in the plane of the panel and another for those normal to the panel. The in-plane mass distribution is accomplished by equally dividing the panel mass among the in-plane dynamic degrees of freedom on the frame member. Based on the discussion of the previous section, two dynamic degrees of freedom are provided in the plane of the frame member, with each one placed at the member end. Hence, the distribution consists of simply concentrating one half of the panel mass at each end of the member.

The distribution of the panel mass among the dynamic degrees of freedom normal to the frame member is a more involved process. To accomplish this task, consideration must be given to the load paths followed in the transfer of normally directed forces from the secondary members of the panel to the frame proper. As the secondary framing plan establishes these load paths, the mass distribution is dependent upon the framing plan within the panel, as well as the number of dynamic degrees of freedom normal to the frame member. The expressions provided in the tables are based on these two considerations. It should be noted that these expressions are also based on the premise that the intermediate mass points are equally spaced along the member; consequently, when the mass points are unequally spaced, the equations in Figures 17 and 19 are not directly applicable. However, they may still be applied by using the techniques described in the discussion further on in this section.

Referring to Figures 17 and 19 and to the corresponding figures (Figures 18 and 20), when the secondary framing is parallel to the main frame, only a small portion of the total panel mass is distributed among the normally directed dynamic degrees of freedom at the intermediate mass points; the remaining panel mass is concentrated at the ends of the member. On the other hand, when the secondary framing is perpendicular to the main frame, the bulk of the total panel mass is distributed among the intermediate mass points.

In some cases, the expressions provided in Figures 17 and 19 may not be directly applicable for distributing the panel mass. One such case occurs when one end of an exterior frame member is a support point, as in the case of an exterior column of a single-story structure. To apply the data in Figures 17 or 19 in such a situation, consider the support node as a mass point and consider each restrained degree of freedom (degree of freedom restrained from moving) as a dynamic degree of freedom. Distribute the mass of the panel using the equations in Figures 17 or 19 and discard the masses assigned to the support node. In many cases, the framing plan within a panel will not correspond exactly to one of those shown in Figures 18 and 20. Nevertheless, the tabulated expressions of Figures 17 and 19 can still be applied. For example, if different framing plans are utilized on either side of the frame centerline, consider the panel as consisting of two subpanels whose longitudinal boundary is the frame member itself, and apply the appropriate expressions of Figure 17 or 19 to distribute the mass of each subpanel (note that the mass of each subpanel should include only one half the mass of the frame member). A similar situation occurs when the secondary framing within a panel contains a relatively large transverse member which transfers a major portion of the total panel load to the frame proper. Since the panel is essentially subdivided by the member, it can be considered as two subpanels whose common boundary is the transverse member for the purpose of computing the masses. Therefore, the mass of each subpanel can be distributed using the equations of Figures 17 and 19. The same technique can be utilized when the intermediate mass points are unequally spaced along a member due to the unequal spacing of secondary framing members within the panel. In this case, the panel is subdivided into several subpanels and the length of each subpanel is defined by the length of the frame member between successive mass points. Thus, the mass of the entire panel can be distributed by apportioning the mass of each subpanel according to the equations in Figures 17 and 19.

When analyzing an end frame, a portion of the mass of the end wall structure should be included in the mass associated with

the model of the frame. The procedures for accomplishing this are as follows:

1. When the girts on the end wall are positioned vertically, consider one half of the wall to be rotated upward 90 degrees, thereby becoming an extension of the roof, as shown in Figure 21. Thus, the mass of this portion of the side wall is taken as part of the roof mass.
2. When the girts of the end wall are positioned horizontally, consider the side wall to be partitioned as shown in Figure 22b and each such partition to be rotated 90 degrees, thereby forming extensions to the blastward and leeward walls, and adding a fictitious interior wall. The mass of these wall extensions are then included as part of the wall mass. The fictitious interior wall is considered as a separate wall panel and its mass is distributed using the appropriate equations of Figure 17.

The third stage of the mass computation consists of preparing a tabulation of the concentrated masses assigned to each mass point of the model. The following general guidelines are to be followed when preparing this tabulation:

1. The masses of interior columns or walls are divided equally among the mass points at the ends of the members.
2. Several masses assigned to one mass point are added to yield the total mass concentrated at the mass point. For mass points with two dynamic degrees of freedom, two summations are required: one for each dynamic degree of freedom at the nodal point.

4.4 Cross-Sectional Properties and Capacities of the Frame Members

General

In order to simulate the elastic and inelastic behavior of the frame, certain physical properties and capacities of the frame members are assigned to the elements of the model. The assigned quantities are:

1. Cross-sectional area
2. Moment of inertia about an axis normal to the plane of the frame
3. Ultimate dynamic load capacity in axial tension
4. Ultimate dynamic load capacity in axial compression
5. Ultimate bending capacity in the absence of axial load.

In most of the structures encountered, standard steel shapes are utilized. Sufficient information is available in Reference 14 to determine the physical properties and capacities listed above.

Tapered members are generally modeled with several elements of equal length and the physical properties of each such element are computed at the midpoint of the corresponding segment of the member. For the pin-connected tension braces of diagonally-braced frames, assigned values of zero for the moment of inertia (Item 2) and the ultimate bending capacity (Item 5) are used in the analysis; and a fictitiously small value is used for the ultimate compressive capacity (Item 4).

Dynamic Tensile Strength

In order to determine the ultimate capacities of the frame members, the appropriate dynamic yield stress for the material must be specified. Generally, the dynamic yield stress is taken as:

$$F_{dy} = cF_y \quad (12)$$

where F_{dy} = dynamic yield stress of material
 F_y = static yield stress of material
 c = the dynamic increase factor

The dynamic increase occurs because of the rapid rate of strain experienced by the material due to the sudden onset of the blast load. Table 2.1 of Reference 1 contains recommended values of the dynamic increase factor for most applications involving structural steel members.

Ultimate Tensile Capacity

The ultimate dynamic load capacity in axial tension is computed as follows:

$$P_p = F_{dy}A \quad (13)$$

where P_p = ultimate tensile load capacity of the member
 A = cross-sectional area of the member.

Ultimate Compressive Capacity

The ultimate dynamic load capacity in axial compression is computed with the following equation:

$$P_u = 1.7F_aA \quad (14)$$

where P_u = the ultimate compressive load capacity of the member
 F_a = the maximum compressive stress permitted in the absence of bending. This quantity is computed using Equation 4.3 of Reference 1.

Ultimate Bending Capacity

The ultimate bending capacities for the frame members are specified as follows:

$$M_{mx} = [1.07 - (z/r_y)\sqrt{F_{dy}/262,394}]M_{px} \leq M_{px} \quad (5a)$$

and

$$M_{my} = [1.07 - (z/r_x)\sqrt{F_{dy}/262,394}]M_{py} \leq M_{py} \quad (5b)$$

The parameters in these equations are defined in Section 2.4.

Reductions of the member bending capacities to account for second order effects are not required, as these phenomena are simulated in the analysis.

SECTION 5

COMPUTATION OF BLAST LOADINGS

5.1 Introduction

Basic to protective design is the capability of calculating efficiently and accurately the anticipated output of an accidental explosion. As applied to the frame analysis, this requirement consists of translating the various blast loading parameters (charge weight, distance, pressure, time, area, etc.) into simplified mathematical functions, of force versus time, which are suitable for use in multi-degree-of-freedom system analyses. The objective of this section is to present the methods to accomplish this task.

The development of loading functions for the frame analysis proceeds in two stages. In the first stage, the exposed area is subdivided into a series of tributary loading areas; each area is associated with a given mass point on one of the exterior members of the frame. With the completion of this initial task, the data preparation then proceeds to the development of the pressure-time histories for the analysis. These pressure-time histories are modified versions of the pressure waveforms determined using the methods and data provided in Section 4 of Reference 3.

The methods and guidelines for accomplishing each of the above tasks are presented in the remainder of this section. Included also is a section containing a discussion of two special effects which must be considered when generating the loading functions in order to avoid introducing spurious effects or numerical errors into the dynamic analysis.

5.2 Computation of Tributary Loading Areas

As discussed in Section 1, each frame is assumed to resist the load on a tributary strip of the building. To distribute the applied loads, the total exposed surface area of the supported strip is subdivided into tributary areas; and each tributary area is assigned to a mass point on an exterior member. In the analysis, the total blast load on a tributary area is taken as acting at its associated mass point. A loaded mass point always lies within the boundaries of its assigned tributary area.

The dimensions of the tributary areas are established on the basis of the framing plan of the exposed areas. Figures 23 and 24 show the dimensions of the tributary areas for panels with the two

most common types of framing plans. The data provided is generally applicable to roof panels as well as wall panels. The term "panel" is used, in the context of this section, in the same manner as described in Section 4.3. It should be noted that the panel boundaries designated for distributing the tributary areas must always correspond to those established for the purpose of distributing the panel mass among the normally directed dynamic degrees of freedom (see Section 4.3).

When sloped surfaces (such as the roof of a gabled structure) are encountered, the loading areas utilized in the analysis comprise the projections of the area onto horizontal and vertical planes.

The boundaries of the tributary areas are determined in the following manner. First, the total panel area is divided into two parts: one part being the area directly supported by the frame member, and the other being the area supported by primary transverse framing members. The directly supported area is divided among the mass points on the member; while the area supported by transverse members is divided equally among the mass points at the ends of the member. The area distributions illustrated in Figures 23 and 24 are based on this procedure.

The utilization of the data provided (Fig 23 and 24) is illustrated in Figure 25 which depicts the subdivision of the exposed area for a two-bay frame. In this figure, the tributary areas for the roof and wall panels are apportioned on the basis of the data provided in Figure 24, which indicates that the secondary framing members are normal to the plane of the frame. However, the guidelines provided in the subsequent discussion apply as well to cases where the secondary framing corresponds to that shown in Figure 23.

Inspection of Figure 25 reveals that mass points on the common boundary of two panels are assigned tributary areas from each of the intersecting panels. In the case of the mass point at the exterior column/girder intersection, the intersecting panels are orthogonal, and therefore two tributary areas are assigned: one in the plane of the wall for the horizontal loading and one in the plane of the roof for vertical loadings. When the intersecting panels are coplanar, as occurs at the connections of the girders to the interior columns, the respective areas of each panel are combined to form a single tributary area which is assigned to the mass point at the column.

5.3 Construction of Pressure-Time Histories for Frame Analysis

General

With the boundaries of the tributary areas established, the development of the loading for the frame analysis proceeds with the construction of the pressure-time histories. The pressure-time histories utilized in the frame analysis are generally multi-linear functions characterized by: a peak pressure; a rise time (time required for pressure to build up to a peak value); and a decay time (time interval over which pressure decays from a peak value to zero).

The construction of these pressure-time histories involves combining the procedures and data in Chapter 4 of Reference 3 with the lumped parameter modeling concepts applied in the frame analysis. This task is accomplished in the following order: first, the procedures of Reference 3 are applied to generate pressure-time histories at the locations of the mass points on each exposed surface [blastward wall(s), roof, leeward wall(s), etc.] of the structure. Then these waveforms are phased, in the time domain, to simulate the effect of the wave traversing the structure and, finally, the waveforms are modified to account for the time lag effects and the non-uniformity of the loading that occur as the wave traverses the individual tributary areas.

Computation of Pressure-Time Histories at Mass Points on Structure

The pressure-time history for a mass point on a blastward wall is computed using the procedures outlined in Section 4-14a of Reference 3. The referenced material provides all of the necessary guidance and data required to accomplish this task. Application of these methods will yield either of the reflected pressure waveforms depicted in Figure 26. The waveform in Figure 26a will be apparent in most cases. However, at high pressure levels, such a representation of the loading may be inaccurate due to the extremely short pressure pulse durations involved. In such cases, the reflected pressure waveform will conform to the one shown in Figure 26b.

A typical pressure waveform for a mass point on the roof, side wall or leeward wall is illustrated in Figure 27. The necessary data for constructing this waveform is contained in Figure 4-5, 4-11 or 4-12, depending upon whether the explosion is above or on the ground, and in Figure 4-66 of Reference 3. Briefly, the peak pressure on these surfaces is taken as the sum of the incident (P_{s0}) and drag pressures ($C_D q_0$). The incident pressure is computed using the data provided in Figure 4-66 together with the drag coefficients supplied in Section 4-14b. The duration, t_{of} , is expressed as twice the positive incident impulse divided by the peak

incident pressure, $2i_s/P_{s0}$. The positive incident impulse is obtained from either Figure 4-5, 4-11 or 4-12 in Reference 3.

Example A.3 illustrates the procedures for computing both the reflected and incident/drag pressure-time histories.

The minimum data required for the analysis of a given frame consists of the following: the reflected pressure-time history at the blastward wall, and the combined incident/drag pressure histories together with the shock front velocities at both the blastward and leeward ends of the roof. In cases involving quartering shock waves, these data are computed at the location of the frame line. The reflected pressure-time history is utilized to generate the loading function on the exterior blastward column. In cases involving normal shock waves, the loading function for the exterior blastward column corresponds to the reflected pressure-time history determined using the procedures of Reference 3.

To generate the loading functions for the roof girder and leeward column, the combined incident/drag pressure-time histories are first utilized to determine the pressure-time histories at all of the mass points on these members. For the mass points on the roof girder, this task is accomplished by linearly interpolating for both the peak pressure and duration using the data (incident and drag pressure durations) computed at each end of the roof. This interpolation is performed on the basis of the distance from the mass point in question to the blastward end of the roof, as shown below:

$$(P_{pk})_i = (P_{pk})_B - [(P_{pk})_B - (P_{pk})_L](z_{ir})/L_R \quad (15)$$

$$(t_{dr})_i = (t_{dr})_B - [(t_{dr})_B - (t_{dr})_L](z_{ir})/L_R \quad (16)$$

where $(P_{pk})_i$, $(P_{pk})_B$, and $(P_{pk})_L$ = the peak pressures acting at the mass point i , and at the blastward and leeward ends of the roof, respectively

$(t_{dr})_i$, $(t_{dr})_B$, and $(t_{dr})_L$ = the durations of the pressure at the mass point i and at the blastward and leeward ends of the roof

L_R = the length of the roof

z_{ir} = the distance from the blastward end of the roof to the mass point i

To determine the pressure-time history for a mass point on the leeward wall, extrapolate the pressure data computed for the roof. Such an approach is utilized when the leeward wall is taken as an extension of the roof (Section 5.3); when the leeward wall is considered as an extension of the side wall, no such extrapolation is required, as all mass points on the leeward wall are equidistant from the blastward end of the frame. Here, the loading for the entire wall is based on the pressure-time history at the leeward end of the roof.

The extrapolation of the pressure data for the leeward wall is accomplished on the basis of the vertical distance from the roof to the mass point in question, as shown below:

$$(P_{pk})_i = (P_{pk})_L - [(P_{pk})_B - (P_{pk})_L](z_{iz})/L_R \quad (17)$$

$$(t_{dr})_i = (t_{dr})_L - [(t_{dr})_B - (t_{dr})_L](z_{iz})/L_R \quad (18)$$

The parameters in these equations are defined above with the exception of z_{iz} , which is the vertical distance from the roof to the mass point in question on the leeward wall.

Interpolation and extrapolation for the shock front velocities at the mass points on the roof and leeward walls are accomplished in a similar manner. In cases involving structures located in low-to-intermediate pressure ranges, the shock front velocity will be constant across the structure. Therefore, there will generally be no requirement for interpolating or extrapolating for these quantities at the mass points on the roof and leeward wall.

In situations involving very long structures or structures located in higher pressure regions, a more accurate interpolation for the peak roof pressures, load durations and shock front velocities can be achieved by determining the combined incident/drag pressure-time histories and shock front velocities at one or more locations along the roof girder, such as the middle of the roof or at every column/girder intersection. The data computed at two such locations is then utilized to interpolate for the pressures, durations and velocities at intervening mass points. Such an approach is also applicable for computing the pressure-time histories and shock front velocities at points on the leeward column when the leeward wall is taken as an extension of the roof. Here, pressure-time histories and velocities are computed at each floor level (including the foundation) and the interpolation is performed for the mass points between successive floor levels.

Phasing of Pressure-Time Histories for the Frame Analysis

The waveforms are phased on a time scale in order to simulate the effect of the blast wave traversing the structure. This phasing is accomplished by specifying a time lag before commencing the loading at each mass point. This quantity represents the time between the initial impingement of the blast wave on the frame and the arrival of the wave at the leading edge of the tributary area associated with a given mass point. This quantity, generally referred to as the arrival or travel time, is computed as follows:

$$t_a = D/U_{AVG} \quad (19)$$

where

t_a = the arrival time

D = the distance from the blastward surface of the frame to the leading edge of the tributary area. This distance is always measured parallel to the direction of the blast wave.

U_{AVG} = the average shock front velocity between the blastward end of the frame and the mass point

Modification of Pressure-Time Histories for the Frame Analysis

Two modifications of the phased pressure waveforms are needed which will account for the time lag effects and non-uniformity in the loading on the individual tributary areas. A typical modified waveform is depicted in Figure 28.

The first modification consists of applying a rise time, t_{rt} , to the loading. This quantity represents the time required for the blast wave to traverse the tributary area, and is computed using the following equation:

$$t_{rt} = a/U \quad (20)$$

where

t_{rt} = the rise time which is the time required for the blast wave to traverse the tributary area

a = the maximum dimension of the tributary area measured parallel to the direction of propagation of the shock front

U = the shock front velocity at the mass point

The second modification is effected to simulate the non-uniformity of the loading on the tributary area. As the wave moves across the area, an unequal pressure distribution will exist. To account for this effect, an average pressure, $(P_{pk})_{AVG}$, is utilized in the analysis.

For the usual case of the linearly decaying loading function (see Fig 28), the average pressure acting on the area is computed using the following expression:

$$(P_{pk})_{AVG} = P_{pk} [1 - (a/2Ut_{dr})] \quad (21)$$

where $(P_{pk})_{AVG}$ = the average peak pressure acting on the tributary area

P_{pk} = the peak pressure (the peak reflected pressure or the sum of the incident and drag pressures, whichever is the case) initially acting at a given mass point

t_{dr} = the duration of the pressure waveform (whether incident or reflected) computed using the methods of Section 4-14 of Reference 3

= t_{of} for the incident pressure-time history

= t_r for the reflected pressure-time history with a linear decay (Fig 26b).

For the particular case of the reflected pressure waveform with the bilinear decay (Fig 26a), the parameter t_{dr} is replaced in Equation 21 by a fictitious duration for the reflected pressure, t_R , which is computed using the following equation:

$$t_R = t_c / [1 - P_s/P_r] \quad (22)$$

where t_c = the clearing time required to relieve the reflected pressure. This quantity is computed as outlined in Section 4-14 of Reference 3.

P_s = the value of the pressure at time t_c (Fig 26a)

P_r = the peak reflected pressure (Fig 26a)

The modified reflected pressure waveform with a bilinear decay is illustrated in Figure 29.

Calculation of Arrival Time, Rise Time and Average Pressure for Tributary Areas on the Varicus Surfaces

To compute the blast loading parameters, t_a , t_{rt} and $(P_{pk})_{AVG}$, the quantities a and D must be known. Examples of a and D are shown in Figures 30 and 31 for tributary areas on blastward walls; in Figure 32 for tributary areas on the roof; and in Figures 33 through 35 for tributary areas on leeward walls. Illustrations showing both normal and quartering shock waves are provided. The figures are not meant to specify the expressions for computing the values of a and D ; instead, they are included to illustrate the manner in which these quantities are determined, and therefore provide guidance for accomplishing this task.

Three illustrations related to the leeward walls are supplied because of the complex nature of the loading on these surfaces. Briefly, the loading on the leeward wall of the structure consists of secondary waves formed by the expanding pressure front as it moves past the rear edges of the building. These waves spill over from the roof and adjacent walls. For the purpose of computing the pressure-time histories for the analysis, the leeward wall is assumed to be an extension of either the roof or adjacent wall. If the spillover from the roof produces a more rapid loading of the leeward wall, the leeward wall is taken as an extension of the roof and the values for a and D are determined as illustrated in Figure 33. On the other hand, if the spillover from the adjacent wall produces a more rapid loading, Figures 34 and 35 are applicable.

In most cases involving single-story buildings, the spillover from the roof produces the more rapid loading on the portion of the leeward wall supported by the interior frames; while the spillover from the adjacent walls produces the same effect for the portion of the leeward wall supported by the end frames. In situations involving multi-story buildings, the tributary areas at the upper stories may be initially loaded by the spillover from the roof; whereas those areas at the base will initially be engulfed by the spillover from adjacent walls.

Calculation of Blast Loading Parameters t_{pk} , t_c' , t_T

The time, t_{pk} , at which pressure $(P_{pk})_{AVG}$ occurs is computed using the following equation:

$$t_{pk} = t_a + t_{rt} \quad (23)$$

The fictitious clearing time, t_c' , for the modified reflected pressure-time history with a bilinear decay is computed using the equation below:

$$t_c' = t_a + t_{rt}/2 + t_c \quad (24)$$

The duration, t_{DI} , of the pressure-time history is computed as follows:

$$t_{DI} = t_{rt}/2 + t_{dr} \quad (25)$$

and the time, t_T , at which the pressure decays to zero is:

$$t_T = t_a + t_{DI} \quad (26)$$

The parameters, t_a , t_{rt} , t_c and t_{dr} are defined in the preceding sections.

5.4 Special Considerations

An area of special concern involves the use of pressure pulses with zero rise times; a situation common in analyses of frames for normal shock waves. Since a portion of the front wall area is assigned to the mass point at the exterior girder/column connection, some of the horizontal load will be applied directly along the longitudinal axis of the roof girder. When this occurs, an artificial magnification of the axial load in the girder may result due to the response of the member in its extensional mode. This magnification, combined with the spurious extensional mode vibrations of the member, will warp the axial load/bending moment interaction, which is the basis for determining when plastic action occurs. As discussed in Section 4.2, one method of avoiding this problem is to use multiple intermediate mass points (up to a maximum of three) on the exterior columns. This will reduce the magnitude of the axial load applied directly to the girder. If this remedy cannot be applied, another means of avoiding the problem is to alter the pressure waveform. This alteration can be accomplished by computing the period of the exterior column taken as a single-degree-of-freedom system. Using one quarter

of this period in place of the rise time, t_{rt} , in the equations of the preceding sections, a waveform corresponding to either of those shown in Figures 28 and 29 (as the case may be) can be constructed and utilized in the analysis.

Problems can also occur when the duration of the blast loading is small compared to the frequencies of the analytical model. This problem will arise in applications of DYNFA to situations involving close-in explosions. Although such applications of DYNFA will be rare, facility designers should be aware of the problem. In these situations, the integration time interval must be short enough to insure that the total impulse is included in the analysis. The use of a maximum time interval equal to 1/20th of the loading duration will achieve the desired result. Too large an interval will produce a stunted representation of the loading in the analysis which, in turn, will produce a lower calculated response of the frame than that which actually occurs.

SECTION 6

DESIGN CRITERIA

6.1 Introduction

In order to restrict the amount of damage a structure will experience when resisting the effects of an accidental explosion, limiting values must be assigned to the appropriate response quantities. For members which can be analyzed as single degree-of-freedom systems (beams, floor and wall panels, joists, etc.), the appropriate quantities are the maximum ductility ratio (defined as the ratio of the maximum deflection to the equivalent elastic deflection) and the maximum end support rotation. Both of these quantities are computed as functions of the deflection at the midspan of the member. A detailed discussion of these criteria is presented in Chapter 2 of Reference 1.

Frame structures, on the other hand, must be analyzed as multi-degree of freedom systems and therefore the peak response of these structures cannot be determined on the basis of a single response quantity. This fact, combined with the wide range and time-varying nature of end conditions of the individual frame members, makes the measurement of ductility ratios and support rotations more complex. Consequently, other criteria are required to measure the complex responses of a frame structure. The deformation criteria in Section 2.3.2 of Reference 1 establishes acceptable limits for certain quantities related to the frame response. However, these response quantities do not, in some cases, provide a clear indication of the amount of inelastic action occurring. Hence, some members may be on the verge of collapsing due to the occurrence of excessive plastic strains while the analysis indicates that the deflections of the frame are within the deformation criteria. Therefore, other response quantities must be monitored in order to provide a more precise measurement of the plastic deformations. In addition, limits for these quantities must be specified in order to restrict the amount of damage suffered by the structure. The designation of appropriate response quantities for measuring the plastic deformations, together with the limiting values specified for these quantities, provide the basis for the ductility criteria for frame structures. These ductility criteria and the applicable deformation criteria in Reference 1 govern the design of blast-resistant frame structures.

The remainder of this section is devoted to the frame design criteria. Consistent with the standards established in

Reference 1, the criteria are specified for two different levels of damage. Structures in the first category are designated as "reusable" structures; while those in the second category are referred to as "non-reusable" structures.

The first part of the frame design criteria consists of the applicable portions of the deformation criteria given in Reference 1. These criteria establish limits on the following response quantities: (1) the sideways deflection, δ , of each story, and (2) the end rotation, θ , (also referred to as the "chordal angle") of the individual members with reference to a chord joining the member ends. These quantities are illustrated in Figure 36. They are computed using the appropriate portions of the response history printed by DYNFA (Section 9.5).

As specified in Reference 1, the limiting values for the aforementioned response quantities are:

1. Reusable structures:

For sideways, maximum $\delta/H = 1/50$

For individual frame members, $\theta_{\max} = 1^\circ$.

2. Non-reusable structures:

For sideways, maximum $\delta/H = 1/25$

For individual frame members, $\theta_{\max} = 2^\circ$.

The second part of the frame design criteria, consisting of the limiting ductility ratios, is presented in the remainder of this section.

A summary of the frame design criteria is provided at the end of this section.

6.2 Measurement of Ductility Ratio

The response quantity used to measure the ductility ratio is the element end rotation, γ (Fig 37 and 38), which is the angle between the longitudinal axis of an element in its initial position, and a tangent to the elastic curve of the element. In contrast, the member end rotation, θ (Fig 36), is measured between one chord joining both ends of a member and another chord joining one end with a point at the midspan of the deformed member.

In the elastic response range, the element end rotation, γ (Fig 37), consists of the two components listed below:

1. A rigid body component, α , which is computed as follows:

$$\alpha = (y_B - y_A)/L \quad (27)$$

where $(y_B - y_A)$ = the differential displacement between the ends of the element

L = the length of the element

2. An elastic component, ϕ , which is the difference between the total element end rotation, γ , and the rigid body rotation, α :

$$\phi = \gamma - \alpha \quad (28)$$

The elastic components, ϕ_A and ϕ_B , at points ends A and B (Fig 36), are a direct measure of the bending moments at the ends of the element.

In the inelastic response range, the element end rotation consists of three components (Fig 37), namely: a rigid body rotation, α , which varies throughout this period as the element continues to deform; the elastic component, ϕ_{max} , measured at the onset of yielding at the end of the element; and a plastic component, β , which commences with initial yielding at the end of the element and increases until a peak plastic rotation, β_{max} , is realized. The elastic component of the rotation remains constant at a value of ϕ_{max} throughout the yielding process until a peak plastic rotation, β_{max} , is achieved. At this time, the element unloads and the elastic rotation varies accordingly. Appendix C presents the equations for the plastic component of the element end rotation for all of the possible combinations of yield conditions.

The computation of the various components of the element end rotations is performed by DYNFA as part of the frame analysis. The program also records the maximum positive and negative plastic and elastic components of the rotations at both ends of each element and uses these values to compute the maximum ductility ratios for each element using the following equation:

$$\mu = 1.0 + \beta_{max}/\phi_{max} \quad (29)$$

6.3 Ductility Ratios for Members Subjected to Bending Alone or to Simultaneous Bending and Axial Tension

Since the members of the frame are subjected to axial loads as well as bending moments, consideration must be given to the impact of the axial load on the ability of the member to sustain plastic deformations. In cases of steel members subjected to bending alone or to simultaneous bending and axial tension, ductility ratios between 13 and 20 were measured at collapse (Ref 16). In order to avoid a failure, however, the maximum ductility ratios have been specified as 3 for reusable structures and 6 for non-reusable structures. These quantities were derived by placing factors of safety (approximately 4 for reusable structures and 2 for non-reusable structures) on the minimum ductility ratio measured at collapse. In assigning these safety factors, the primary consideration in the case of reusable structures was to limit the amount of damage; whereas in the case of non-reusable structures, the primary consideration was to provide an ample margin of safety against collapse.

6.4 Ductility Criteria for Beam Columns

Failure Mode of Beam Columns

Beam columns may suffer an instability failure at ductility ratios below those measured for members subjected to simultaneous bending and axial tension. Consider the typical load-deformation response, depicted in Figure 39, for a laterally loaded beam column. Note that the load-carrying ability of the member dissipates after the peak load has been achieved. As the unloading progresses, an instability failure occurs due to the formation of plastic hinges on the member. Hence, the effects of stability must be considered for beam columns, because in some situations, plastic deformations corresponding to ductility ratios of 3 and 6 may not be attainable without a failure occurring.

Basis of Criteria

In order to incorporate the effects of beam column instability into the failure criteria, the data and methods of Galambos and others (References 16 and 17) were used to develop limiting ductility ratios for beam columns. In this report and in References 16 and 17, the ultimate ductility ratios are expressed in terms of the rotation capacity of the member end. The rotation capacity, R_c , is defined as the ratio of the maximum plastic rotation at failure, β_{max} , to the maximum elastic rotation, ϕ_{max} , which occurred at the onset of inelastic action (see Fig 39).

The ultimate ductility ratio is defined in terms of the rotation capacity as follows:

$$\mu_{ult} = 1.0 + R_c \quad (30)$$

In References 16 and 17, the quantities ϕ_{max} and β_{max} are measured in terms of the total member end rotation; whereas in DYNFA, these quantities are measured in terms of the relative member end rotation. Nevertheless, computing R_c in terms of either of these quantities produces similar results. Hence, the DYNFA method for computing the ductility ratios is consistent with the method used in References 16 and 17 for establishing the ductility criteria.

Members Subjected to End Moments

The limiting rotation capacities will vary depending upon the nature of the applied loads/moments and the conditions of restraint existing at the member ends. To provide a basis for the design of beam columns, data were developed for most, if not all, of the various types of loadings and end conditions encountered in frame structures. Rotation capacity data are provided in Reference 16 for beam columns subjected to bending moments applied at one or both ends of the member. These data are presented in Figures 41 and 42. The figures present the relationship between the rotation capacity and the applied axial load for members with varying slenderness ratios. In the figures, the applied axial load, P , is expressed in non-dimensional form as a fraction of the ultimate tensile capacity, P_p ; and the slenderness ratio is designated by the ratio l/r where l is the total length of the member and r is the radius of gyration about the axis of bending. Figure 40 contains an index which correlates the loading condition with the appropriate ductility criteria.

Members Subjected to Lateral Loads

The design data for laterally-loaded members were developed using the methods and data provided in References 17 and 18. Rotation capacity curves for laterally-loaded members were developed using a numerical integration procedure (Ref 18) to determine the lateral load versus end rotation relationship for the member. A typical curve is depicted in Figure 39. As illustrated in the figure, the maximum elastic (ϕ_{max}) and plastic (β_{max}) rotations are defined as the rotations which correspond to a lateral load equal to 95 percent of the peak load on the curve. This definition is based on the results of tests on beam columns subjected to applied end moments (Ref 16). These tests revealed a marked

increase in the rate of collapse as the member unloaded to a moment equal to 95 percent of its measured capacity.

The design data developed is presented in Figures 44 through 53. The loading condition considered for the generation of these data consisted of a uniformly distributed lateral load applied simultaneously with a compressive axial load. The curves in these figures express the limiting rotation capacity as a function of the slenderness ratio of the member and the applied axial load. The rotation capacities are computed for the location on the member at which the first plastic hinge forms. Each curve in these figures corresponds to a constant slenderness ratio.

In developing the design data for this report, the methods of Reference 17 were extended to situations in which rotational restraints are provided at the ends of the member. The purpose of this extension was to more accurately simulate conditions in an actual structure, where all members are provided with elastic rotational restraints at their ends.

Two general conditions of rotational end restraint were considered, namely: both ends of the member restrained by elastic elements of equal stiffness; and one end of the member elastically restrained and the other end pinned. The data provided for each condition of end restraint were computed for five values of the rotational stiffness of the restraining element. Each of Figures 44 through 53 contains the rotation capacity curves for one value of the rotational stiffness. In each figure, the stiffness of the restraining element is expressed in terms of the moment distribution factor of the beam column. Figure 43 contains an index of the figures for the various values of the distribution factors.

The distribution factor is commonly used in performing moment distribution (Hardy-Cross) analyses of continuous beams. The distribution factor at one end of a frame member is computed as follows:

$$DF_{AB} = K_{AB} / \Sigma K \quad (31)$$

where

DF_A = the distribution factor for frame member AB at joint A

K_{AB} = the relative rotational stiffness of frame member AB

ΣK = the sum of the relative rotational stiffnesses of all frame members (including member AB) intersecting at joint A.

The reader is referred to one of the standard texts on indeterminate structural analysis (such as Ref 19) for additional guidance in computing these factors.

The rotation capacity data provided in the figures cover a range of cases in which the distribution factors for the frame members vary from a minimum of 0.0625 to a maximum of 0.75. The lower values (0.0625 through 0.25) reflect conditions of restraint approaching fixidity; whereas the highest value (0.75) reflects a condition approaching a pin-ended support.

Factors of Safety and Limiting Ductility Ratios

The data contained in the aforementioned figures represents the values of the rotation capacities at a condition of imminent failure. In order to provide an ample margin against collapse of the member, a factor of safety is applied to these values in the frame design. The safety factors specified are 4 for reusable structures and 2 for non-reusable structures. Since the rotation capacity data preclude fracture of the material, limiting ductility ratios of 3 for reusable structures and 6 for non-reusable structures are specified.

The ductility ratio utilized for the design of a beam column is the minimum of either the ductility ratio derived from the data in Figures 41, 42 and 44 through 53, or the limiting ductility ratio.

Use of the Rotation Capacity Curves

The data contained in Figures 41, 42 and 44 through 53 are used in conjunction with the output of DYNFA to determine the adequacy of the members of the frame. The maximum allowable ductility ratio for a member is determined in the following manner. First, using the computed moments which occur when the maximum ductility ratio is realized in the response, sketch the moment diagram for the member. Compare this moment diagram with those shown in Figures 40 and/or 43. These figures correlate the data contained in Figures 41, 42 and 44 through 53 with the various loading conditions for which the criteria were developed. This correlation is accomplished on the basis of the moment diagrams produced by the various loading conditions.

It should be noted that the bending moment diagrams shown in Figures 40 and 43 conform to the standard beam sign convention (i.e., negative moment produces tension at upper or outside flange of member). On the other hand, the bending moments printed by DYNFA conform to a sign convention which is based entirely on

the direction of rotation produced by the moment (i.e., negative moment produces clockwise rotation). Therefore, when sketching the moment diagram for a member, the signs of the moments must be altered to conform to the standard beam sign convention.

Find the moment diagram in Figure 40 or 43 which has the closest resemblance to the moment diagram constructed using the DYNFA output and extract the ductility criteria for the member from the appropriate figure(s) of those listed. Figure 40 is generally applicable to members subjected to end moments; whereas Figure 43 applies to laterally-loaded members. Figure 40 may also apply in cases where small lateral loads are encountered. Due to the complexities inherent in the blast-induced response of a frame structure, the peak response of an exterior member may be produced by two effects, namely: the transverse loads acting directly on the member, and the bending moments applied at the ends of the member. The latter effect is usually produced by the direct loading on adjacent members or the overall sidesway response of the frame. In such cases, the bending moment diagram for the member at the instant it attains its peak response, will exhibit characteristics of the moment diagrams for each of these load cases. When this occurs, the ductility criteria for the member is taken as the minimum of the ductility criteria for the two loadings producing the member's peak response. Generally, the ductility criteria for the member will correspond to that for the lateral load case (Fig 43).

Second, from the printed output of DYNFA, read the axial load ratio, P/P_p , where P corresponds to the axial load in the member when the maximum plastic rotation occurs and P_p is the ultimate tensile capacity of the member. Enter the appropriate curve (in Fig 41, 42 or 44 through 53) with this quantity together with the slenderness ratio (L/r) for the axis of bending of the member, and read the limiting rotation capacity. Here, the total length, L , instead of the effective length, KL , (where K is the effective length factor), is used to compute the slenderness ratio. In some cases, interpolation of the data may be required in areas where the curves are extremely sensitive to variations in one or more parameters, such as: the slenderness ratio, L/r , or, in cases involving laterally-loaded members, the distribution factor, DF . Here, engineering judgement will have to be exercised to determine whether the interpolation is required. The following guidelines are offered to assist in the interpolation:

1. For values of P/P_p less than 0.20, plot a curve of the values of R_c versus L/r for a constant value of DF and P/P_p . Read from this curve the

value of R_c for the value of l/r required. For values of P/P_D greater than 0.20, interpolate for l/r by inspection.

2. To interpolate for values of DF, plot curves of R_c versus DF for constant values of P/P_D and l/r ; use at least three points for this curve. Read from this curve the value of R_c for the distribution factor, DF, required.
3. When the distribution factor is less than 0.0625 or greater than 0.75, the values of R_c shall be taken as equal to those for distribution factors of 0.0625 and 0.75, respectively.
4. When the distribution factors, DF_{AB} and DF_{BA} for a member restrained at both ends are unequal, enter the curves with the lower of the two values of the DF, and read the corresponding values of R_c for each of the following cases:
 - a. Both ends of member restrained (Ref Fig 44 through 48). The value of R_c for this case is the upper limit of the rotation capacity for the member.
 - b. One end restrained, other end pinned (Ref Fig 49 through 53). The value of R_c for this case is the lower limit of the rotation capacity for the member.

Using the two values of R_c thus obtained, the value of the rotation capacity for the member is computed as illustrated in Figure 54. In the example shown the distribution factor DF_{AB} is taken as the lower of the two distribution factors for the member. The upper portion of the figure shows the tabulated rotation capacities for cases a and b above. In the lower portion of the figure, the tabulated values of R_c are plotted versus the corresponding values of DF for end B of the member. Since data for only two points are available, the plot is taken as a straight line. The value of $(R_c)_m$ for the member is then determined by entering the plot with the actual value of distribution factor, DF_{BA} .

Finally, the limiting ductility ratio is computed by dividing the rotation capacity by the appropriate factor of safety and adding one to the resulting quotient.

Limitations in Ductility Ratio Criteria for Beam Columns

The failure criteria related to beam columns was developed on the basis of a series of non-linear analyses (Ref 16, 17 and 18) in which the following simplifying assumptions were made:

1. The member is subjected to in-plane bending only.
2. The ends of the member remain stationary; that is, no differential displacement occurs between the member ends.
3. In most cases, collapse will be caused by the overall instability of the member, and therefore, the effects of local failure modes, such as local or torsional buckling, can be neglected.

In the first instance, it was considered appropriate to base the criteria on the in-plane bending behavior of beam columns and account for the effects of bi-axial bending by using reduced bending capacities in the frame analysis, as described in Section 7. Application of the provisions in Section 7 will yield an ample margin of conservatism which precludes the need for a three-dimensional stability criteria.

The second assumption neglects the possible reductions in the rotation capacities of the columns due to sidesway motions of the frame. However, these reductions are partially accounted for by the inclusion of the $P-\Delta$ effect in the analysis. In addition, they will be offset, to some extent, by the stabilizing effects of the inertial forces on the member, which retard the buckling process.

Frame members designed according to the provisions contained in Sections 3.3.4 through 3.3.6 of Reference 1 will not be susceptible to local failures, such as local buckling or lateral torsional buckling. Therefore, the exclusion of the local failure modes from the development of the criteria is appropriate. However, in some cases, the design cannot conform to the referenced specifications and, therefore, the possible occurrence of these types of failures may reduce the available rotation capacity of the member (Ref 16). Reductions caused by lateral torsional buckling are partially accounted for by using a reduced bending capacity for the member in the analysis (Section 4.4). However, additional data is required to establish criteria for beam columns subjected to local failures. At present, there is little data available for this purpose and, therefore, in the absence of this data, it may be necessary to restrict the amount of inelastic

action to much lower levels than those specified in this section. In extreme cases, it may even be necessary to design the members to remain elastic throughout the response. Here, the designer must exercise some judgement in assessing what level of inelastic dynamic response is tolerable.

6.5 Summary of Criteria

1. Sidesway and end rotation criteria

a. Reusable structures

For sidesway, maximum $\delta/H = 1/50$

For individual frame members, $\theta_{\max} = 1^\circ$

b. Non-reusable structures

For sidesway, maximum $\delta/H = 1/25$

For individual frame members, $\theta_{\max} = 2^\circ$

2. Ductility criteria

a. Reusable structures

Members subjected to combined bending and axial tension, $\mu_{\max} = 3$

Members subjected to combined bending and axial compression:

$$\mu_{\max} = 1 + R_c/4$$

or

$$\mu_{\max} = 3, \text{ whichever governs.}$$

b. Non-reusable structures

Members subjected to combined bending and axial tension, $\mu_{\max} = 6$

Members subjected to combined bending and axial compression:

$$\mu_{\max} = 1 + R_c/2$$

or

$\mu_{max} = 6$, whichever governs.

- Note:**
1. R_C in the above equations designates the ultimate rotation capacity read from the appropriate curve of those provided in Figures 41, 42 and 44 through 53.
 2. The limits specified apply to plastic deformations occurring within the span of a member or at the member ends.

SECTION 7

DESIGN METHODS FOR BI-AXIAL BENDING OF COLUMNS

7.1 Introduction

The nature of the blast loading on rectangular buildings is such that, for any direction of shock wave propagation, some building columns, if not all, are subjected to bi-axial bending. To illustrate this, consider the structure shown in Figure 55, which is typical of those encountered in ammunition facilities. Under the influence of Shock Front I, all of the Y-direction frames will experience sideways motion, while those in the X-direction will not. In addition, all of the exterior columns on lines 1 and 4 will respond to the side-on pressures. As a consequence, all of the exterior columns will undergo bi-axial bending while the interior columns experience bending about one axis only. If the effects of Shock Front II are considered, bi-axial bending occurs in all of the columns.

When designing the interior Y-direction frames for Shock Front I, each frame can be analyzed with DYNFA as an independent structural system. However, in situations where bi-axial bending occurs (such as those described above), the frame design, and therefore the DYNFA analysis, must account for the interactions between the responses of orthogonal frames. Since the analysis performed by DYNFA is restricted to uni-axial bending of the frame members, an approach is needed whereby the applicability of the program can be extended to situations involving bi-axial bending. Such an approach is presented in this section.

7.2 Basic Procedure

A more rigorous approach to the problem would entail a three-dimensional elasto-plastic analysis of the entire building. For the general type of structure encountered, this would constitute an impractical task. An alternate approach has been developed which utilizes the DYNFA program as described below.

Briefly, several representative frames spanning in each direction are selected for analysis with DYNFA. In cases involving shock waves normal to one of the exterior walls of the building (such as Shock Wave I shown in Fig 55), the procedure can be simplified as only the exterior columns of the side wall experience bi-axial bending. Hence, the analyses of the frames positioned perpendicular to the blast direction can be omitted and the responses of the exterior columns to the side-on pressures

can be estimated on the basis of single degree-of-freedom analyses.

Two DYNFA analyses are performed for each selected frame. In the first analysis, inelastic behavior of the columns is neglected. This is accomplished by using fictitiously high bending moment capacities (at least 2 to 3 times the actual capacity) for the elements representing these members. Based on the results of the first group of analyses, the bending capacity about each axis of each building column is reduced to account for the simultaneous application of bending moments about both axes. With reduced bending capacities specified for the columns, each frame is re-analyzed with DYNFA. The inelastic behavior of the columns is included in the second group of analyses. The adequacy of all frame members is assessed on the basis of the results of this second group of analyses.

Additional information on the procedure is provided in the remainder of this section.

7.3 Use of Results of First Group of Analyses

General

The results of the first group of analyses are used for two purposes, namely: to make a preliminary assessment of the structural integrity of each frame analyzed, and to determine the reduced bending capacities of the columns for use in the second group of analyses.

Preliminary Assessment of Structural Integrity

The structural integrity of the frame members is investigated on the basis of the criteria specified in Section 6. From the DYNFA results, tabulate the following response parameters, namely: the peak sidesway deflection of each frame, and the maximum chordal angle and ductility ratio for each girder. Also tabulate the maximum chordal angle for each exterior column. Procedures for using the printed output of DYNFA to determine the sidesway deflections and chordal angles are presented in Section 9. The ductility ratios are computed and printed by DYNFA. Compare these quantities with the frame design criteria of Section 6. At this stage of the design, margins of 10 to 20 percent of the limiting values are required between the computed response parameters and the design criteria. Where such margins are not apparent, increase the sizes of the under-designed members and rerun the analysis of that particular frame. These margins are necessary to insure that the inelastic deformations of the frames in the second group of analyses will not exceed the design criteria.

Reduced Bending Capacities of Columns

Following the preliminary assessment of the structural integrity, the design proceeds with the computation of the reduced bending capacities of the columns. For a given column, tabulate at each nodal point on the member, the maximum bending moments which occur about each axis. Because only a few representative frames in each direction are analyzed, the printed output of DYNFA will not contain the peak bi-axial bending moments for every column being investigated. However, it can generally be assumed that the response histories of several successive frames spanning in one direction will be similar in magnitude, frequency and duration, provided that the members of these frames have similar properties and the loadings on each frame are similar in magnitude and duration. By virtue of this assumption, the DYNFA results for one column may be applied to another. For instance, suppose the desired quantity is the peak moment about the X-axis of column B3 (see Fig 55), but frame 2 was analyzed, not frame 3. In this case, the desired moment is taken as the peak X-axis moment for column B2. This assumption applies for estimating the responses of the leeward exterior frames; that is, the response of frame 4 can be assumed to be equal to the response of frame 1 provided that the loadings for both frames are similar. When larger pressure differentials occur between the ends of the building, it may be necessary to analyze all of the exterior frames.

With the peak-applied moments, M_x and M_y , tabulated at each nodal point, the nodal capacity reduction factors, R_n , are computed as shown below:

$$|M_x|/M_{mx} + |M_y|/M_{my} = R_n \quad (32)$$

where M_{mx} and M_{my} are defined and computed as described in Section 2.4.

To eliminate unnecessary analyses, the following limits are specified for the nodal reduction factor:

$$R_n \leq 1.25 \text{ for a reusable structure, and}$$

$$R_n \leq 1.50 \text{ for a non-reusable structure.}$$

These values apply to all but corner columns. The limits for corner columns are:

$$R_n \leq 1.5 \text{ for a reusable structure, and}$$

$$R_n \leq 2.0 \text{ for a non-reusable structure.}$$

If all nodal reduction factors for a member are greater than the limiting value, the member should be increased in size. On the other hand, if all of the reduction factors are less than one, decrease the size of the member. Compute nodal reduction factors for the new member using its bending capacities and the bending moments, M_x and M_y , computed for the original member.

Reduced moment capacities for the second group of analyses are then computed as follows:

$$\bar{M}_{mx} = |M_x|/R_n \quad (33a)$$

$$\bar{M}_{my} = |M_y|/R_n \quad (33b)$$

Moment capacity reductions are effected at every nodal point on each column. To illustrate the use of Equation (33), a sample computation is provided in Table 6 for a column subjected to biaxial bending. The column involved is modeled with two elements (2 nodal points).

7.4 Use of Results of Second Group of Analyses

In the second phase of the analysis, the frames are re-analyzed with reduced bending capacities specified for the elements representing the columns. The results of this second group of analyses are then checked against the design criteria of Section 6 to determine the adequacy of the frame members.

SECTION 8

USER'S MANUAL FOR DYNFA

8.1 Introduction

The material presented in this User's Manual includes a summary of the program capabilities, descriptions of the input data, specifications of the computer usage, dimensional limitations of the program, and the formats for the input data cards. A graphical illustration showing the arrangement of a typical data deck is presented at the end of the section. The printed output of the program is described in Section 9.

8.2 Capabilities of Program

The DYNFA program performs a multi-degree-of-freedom non-linear dynamic analysis of a frame structure subjected to arbitrary time-dependent loadings. The program considers the P- Δ and beam-column effects in the analysis. In addition, the program contains a static analysis routine. When this routine is utilized, the static nodal displacements and element loads are used as the initial conditions for the dynamic analysis.

The results of the dynamic analysis consist of the deformations of the structure expressed in terms of the nodal displacements and rotations, the axial loads, bending moments and shears in each of the elements, and the plastic deformations expressed in terms of ductility ratios for the elements.

8.3 Input Data

The input data are transmitted to the computer program via punched cards containing the following data which are related to the analytical model of the structure:

1. The coordinates of the nodal points in a right-handed Cartesian coordinate system.
2. The translational and rotational restraints specified for the support nodes.
3. The masses assigned to the mass points.
4. The nodal connectivities, cross-sectional properties, and the axial load and bending moment capacities of the elements.

5. The integration time interval and the number of these time intervals for computing the response of the structure.
6. The pressure waveforms for simulating the transient loading on the structure.
7. The tributary loading areas assigned to the mass points on the exterior members.

Before the input data can be punched on cards, identification numbers must be assigned to the nodal points, elements and pressure waveforms. Three distinct sets of identification numbers are established: one set for the nodal points, one for the elements and a third for the pressure waveforms. The numbers in each set must be in sequential order beginning with the number 1; gaps in the numbering are not permissible. The appropriate identification number must be entered on input cards containing data pertaining to any of the aforementioned input parameters. Figure 56 shows identification numbers for the nodal points and elements of a typical model. In the figure, the element numbers are enclosed in circles in order to distinguish them from the nodal point numbers. It is often desirable to do this in order to avoid confusion.

Additional information pertaining to the various input quantities is contained in the following sections.

8.4 Nodal Coordinates

The nodal coordinates locate the position of the nodal points with respect to the origin of a right-hand Cartesian coordinate system (commonly referred to as the global coordinate system). The orientation of this coordinate system and the location of its origin is chosen by the analyst. The nodal coordinates, in effect, define the geometry of the analytical model.

The sign convention of the global coordinate system must be rigidly adhered to when specifying such input parameters as the nodal coordinates, tributary areas and static loads. To facilitate the data preparation, the axes of the global coordinate system should be directed horizontally and vertically, and the positive directions of these axes should be established as follows:

1. The positive direction of the horizontal axis should be taken in the direction of the propagation of the shock front, and
2. The positive vertical axis should be directed upward.

Figure 56 shows the global coordinate system for a typical model. In order to minimize the possibility of errors in the preparation of the input data cards, the origin of the global system should be located such that most, if not all, of the nodal coordinates are positive quantities. This is accomplished by locating the origin at the lower left-hand corner of the model, as shown in Figure 56.

In the input data specifications and in the printed output of the program, the global axes in the plane of the model are designated as the X and Y axes. The global Z axis is perpendicular to the plane of the model. The positive direction of the global Z axis is established by the application of the right-hand rule.

The global coordinate system establishes the signs and directions of the nodal displacements computed by DYNFA.

8.5 Conditions of Restraint

Translational and/or rotational restraints are specified for nodal points corresponding to support points. Restraints are specified for the individual components (translations in the X and Y axes and rotation about the Z axis) of the displacement vector at a nodal point. Where no restraint is apparent, none should be specified. Figure 57 illustrates the manner in which the support conditions for a structure are specified in terms of nodal restraints.

Caution must be exercised when specifying this input data as errors will completely alter the results of the analysis. In addition, enough nodes must be restrained to prevent rigid body motion of the model.

8.6 Lumped Masses

Masses are assigned to selected nodal points in the model. The methods utilized to distribute the mass in the model are presented in Section 4.3. Two masses, one for each translational degree of freedom, may be specified for each mass point; however, the mass quantities specified for a single mass point need not be equal. Positive and/or negative signs must never be assigned to these quantities.

8.7 Input Data for Elements

General

The data required consists of the nodal connectivities, the cross-sectional properties and the axial load (tensile and compressive) and bending moment capacities. The input data related to the nodal connectivities are discussed in this section. The other items are treated in Section 4.4.

Nodal Connectivities

The nodal connectivities are the nodal points at the ends of each element. These quantities define the connectivity of the members of the structure; hence, they establish the configuration of the model.

The nodal connectivities are also used to define a right-hand Cartesian coordinate system associated with each element. This system is commonly referred to as the local coordinate system. The local axes in the plane of the model are designated the x-y axes; the local axis perpendicular to the plane of the model is designated as the z axis.

The local x axis for an element coincides with its longitudinal axis; the direction of the local x axis is determined by the order in which the nodal connectivities are entered on the element input card. The first nodal point number punched on the element card is taken by DYNFA as the origin of the local system. The positive direction of the x axis is from the first to the second nodal point number specified for the element. The direction of the positive z axis taken by the program to be identical to that chosen by the analyst for the global Z axis. The positive direction of the local y axis is determined in the program by the application of the right-hand rule. The local coordinate system for an element is shown in Figure 58. The designations JA and JB in the figure refer to the first and second nodal points entered on the element input card.

Local coordinate systems for colinear and/or parallel elements should have the same orientation. In this way, the axial loads and shears for these elements will conform to the same sign convention. It is also recommended that the elements representing a single frame member be numbered sequentially, and that the numbering of these elements begin at one end of the member and proceed to the other end. In this manner, the loads and bending moments for the member will be printed out in a properly ordered block of data.

Pinned Connections of Elements

In the DYNFA analysis, it is assumed that the relationships between all force and displacement vector components of an element are governed by identical continuity and equilibrium equations. In many structures, there may be local deviations from this condition. In frame structures, such deviations are caused by non-rigid member connections and other types of structural discontinuities such as splices in the span of a member or the pinned connections of diagonal braces in braced frames. The DYNFA program has provisions for including these structural discontinuities in the analysis. In DYNFA, structural discontinuity is simulated by inserting a local pin at the end of an element. The specification of a local pin indicates that the moment at the pinned end of the element is zero. Local pins produce discontinuities in the individual elements for which they are specified; they do not alter the conditions of continuity and equilibrium for other elements. Figure 59 illustrates the manner in which several types of member connections are specified in terms of local pins.

The careless use of local pins can lead to errors in the analysis. Where a moment discontinuity exists at the intersection of two elements, only one intersecting element should be pinned; pinning both elements at the same node will produce a singularity in the system stiffness matrix causing a premature abortion of the program execution. Local pins should never be used to specify a pinned support node. The pinned condition at a support node is automatically created by the exclusion of a rotational restraint for the node.

8.8 Integration Time Interval and Duration of Response

Integration Time Interval

Determining the integration time interval, to be utilized for the numerical integration, involves estimating the frequency of the highest mode of vibration of the model. The increment to be used should be approximately 1/20 of the period of vibration of the highest mode in the model, whether it be an extensional or bending mode.

The equation for the period of the primary extensional mode of a member is:

$$T_a = 2\pi\sqrt{LM_e/AE} \quad (34)$$

where T_a = the period of the primary extensional mode

L = the total length of the member

M_e = the concentrated mass at the end of the member. The mass utilized is associated with the dynamic degree-of-freedom in the direction of the longitudinal axis of the member. When masses are concentrated at both ends of a member, the lesser of the two should be used to compute the period.

A = the cross-sectional area of the member

E = the modulus of elasticity of the material.

The bending modes considered comprise the higher order bending modes of the individual members. Only members with intermediate mass points are investigated. The general equation for computing the natural period of vibration for the bending mode of a beam is:

$$T_b = (1/C)\sqrt{M_j L^3/EI} \quad (35)$$

where the parameters L and E are defined above and the remaining parameters are defined below:

T_b = the natural period of vibration

M_j = the summation of the masses at all intermediate mass points on the member

I = the moment of inertia of the member

C = constant

Values of the constant "C" are provided in Table 7 for beams with up to four intermediate mass points between supports. In the table, data are provided for members with different support conditions. In choosing which support condition applies for a given member, consideration should be given to the rotational restraint provided at the ends of the member by adjoining members. Only the period of the highest mode of the member is required.

The proper selection of an integration time interval is mandatory when utilizing numerical integration procedures. Using an increment that is too large will result in a numerical integration which either produces erroneous results or becomes

unstable; while using an increment that is too small entails greatly increased computer costs.

Duration of Response

The number of integration time intervals specified in the input is determined on the basis of the natural period of the sideway mode of the frame. Generally, the response is computed for a duration of one half the period of the sideway mode. In most cases, all of the significant responses will occur during this time interval. If the rebound of the frame is desired, the computation of the response is extended to a duration equal to the period of the sideway mode.

The sideway natural period of single-story rectangular frames is computed using the following equation:

$$T_s = 2\pi\sqrt{m_e/KK_L} \quad (36)$$

where

T_s = the natural period of the frame

m_e = the equivalent mass of the frame consisting of the total roof mass plus one third of the column and wall masses

K = the stiffness factor for single-story rectangular frames subjected to uniform horizontal loads. This quantity is defined in Table 8.

K_L = a load factor that modifies K
 $= 0.55(1 - 0.25\beta)$

β = base fixity factor (i.e., $\beta = 0$ for a pinned base and $\beta = 1$ for a fixed base)

The natural period for rigid frames with supplementary diagonal tracing is given by:

$$T_s = 2\pi\sqrt{m_e/(KK_L + K_b)} \quad (37)$$

All parameters in this equation, with the exception of K_b , are defined above. K_b is the horizontal stiffness of the diagonal bracing system which is given by:

$$K_b = (nA_b E \cos^3 \gamma) / L \quad (38)$$

where n = the number of diagonal braces

A_b = the cross-sectional area of a diagonal bracing member

γ = the angle between the bracing member and a horizontal plane

Note that the frame sciffness, K , is zero for braced frames with non-rigid connections.

The number of time increments specified in the input is computed as shown below:

$$NDT = 1 + T_f/\Delta t \quad (39)$$

where NDT = the number of time increments specified in the input

T_f = the desired duration of the response

Δt = the integration time increment

When computing NDT , the quotient $T_f/\Delta t$ is always rounded off to a whole number.

Time Interval for Printed Output

An additional time parameter can be specified in the input. This parameter indicates the number of integration time stations to be skipped in the printed output. Since the significant responses of the structure occur in low frequency modes, the response need not be printed out at every integration time station as much of these data would be of little interest. Generally, the time interval utilized for printing the response output is taken as 10 to 20 times the integration time increment.

8.9 Pressure-Time Histories and Tributary Areas

General

The blast loading on the frame is specified in two parts. The first part consists of the pressure-time histories; the second part consists of the tributary areas assigned to the mass points. Methods for computing these data are provided in Section 5.

Input Specification of Pressure-Time Histories

The input specification of each waveform consists of the pressures acting at discrete times. The waveform is approximated in the analysis by a series of intersecting linear segments. Each waveform must be defined for a time interval which begins at $t = 0.0$ second and extends through the specified duration of the response. Intervals of zero pressure must be accounted for in the input. Figures 60 and 61 show the digitized pressure-time data that are entered in the input for the typical waveforms depicted in Figures 28 and 29, respectively. Note that the digitized data specification for each waveform includes the time interval during which the pressure is zero.

Each pressure waveform must be assigned an identification number. These numbers are used by DYNFA to match the pressure waveforms with the corresponding tributary areas.

Input Specification of Tributary Areas

The assignment of the tributary areas to the various mass points is described in Section 5.2. Each input card containing tributary areas must also contain the nodal point number of the mass point to which the areas are assigned. Two areas may be entered for each mass point. Each tributary area that is entered in the input must be accompanied by the appropriate pressure waveform identification number.

8.10 Specification of Dead and Live Loads

The static analysis routine in DYNFA is provided for the purpose of performing the dead and live load analyses of frames. Two types of static loadings can be specified in the input. The first consists of uniformly distributed loadings on the elements. The direction of these loads is established by specifying the axis of the global system (X , $-Y$) in which they act. The second type consists of concentrated loads applied directly to nodal points. Concentrated nodal loads conform to the sign convention of the global coordinate system.

The dead load supported by a frame consists of the weight of all roof structure and some wall structure within its tributary strip of the building. The distribution of the dead load of the roof is dependent upon the positioning of the purlins. When the purlins are parallel to the frame, the dead loads of the roof are distributed in the following manner:

1. The weight of decking supported by the girder, plus its own weight, is applied as a uniformly distributed load to the elements representing the member.
2. The remainder of the roof dead load is applied as a series of concentrated nodal loads at the girder/column intersections.

When the purlins are positioned normal to the frame, either of two procedures are utilized, depending upon the number of purlins supported within the span of a particular girder. In cases involving girders supporting three or less purlins, the following procedure should be utilized:

1. The weight of the girder is applied as a uniform load to the elements representing the member.
2. The weight of a purlin and the decking it supports is applied as a concentrated load to the nodal point corresponding to the purlin/girder connection.
3. The remaining dead load of the roof supported by the frame is applied as a series of concentrated loads at the girder/column intersections.

When more than three purlins are supported within the span of a girder, all weight supported by the member, plus its own weight, is applied as uniformly distributed loads to the elements representing the girder.

It is debatable as to what portion of the wall structure is supported by the frame. Certainly, the columns support themselves; hence, the weight of these members is applied as uniform loads to the elements representing them. In single-story buildings, the secondary structure of an exterior wall supports itself. However, it is conceivable that blast-induced failures in the siding or in the connections of the siding to its support could result in the redistribution of part or all of the dead load of the wall structure to the column. To account for this, one half of the weight of the secondary structure of the wall is applied as a concentrated nodal load at the exterior girder/column connection.

8.11 Computer Usage and Restrictions

The DYNFA program is written in FORTRAN IV for the CDC 6600 computer. The FORTRAN coding of the program is given in Appendix D.

The size restrictions imposed by the dimensional constraints of the program are summarized as follows:

<u>Item</u>	<u>Maximum Number</u>
Number of nodal points	30
Number of nodal points with restraints	30
Number of mass points	30
Number of beam elements	30
Number of pressure waveforms	30
Number of discrete time stations for each pressure waveform	20
Number of nodal points with assigned tributary areas	30
Number of nodal points with concentrated loads (static)	30
Number of integration time increments	Unlimited
Number of output-time stations	Unlimited

8.12 Units

The program utilizes either the International System (SI) or the U.S. System of Units. The SI Units are contained in the program; whereas the U.S. Units must be requested by the user in the input. The units for the various input parameters are provided in Section 8.13. The units used in the output are specified in Table 9.

8.13 Formats of Input Data Cards

Introduction

This section presents the formats used for specifying the various input parameters.

There are 15 types of cards used to specify the input data for this program in addition to a terminator card required to signify the end of the data. Each type of card is described below in terms of data format, definition and field allocations. The numbers above the graphic representation of each card identify the last column in each data field of that card. The letters below designate the format for the data. In fields designated "I", the quantity entered must be right-adjusted to the last column in the field. A decimal point must not be used with "I" formatted input data. In the fields designated "F", a decimal point is required; however, the number can be located anywhere within the field. Fields designated "A" may contain alphanumeric data. A plus sign for a positive quantity must not be used as it will abort the execution of the program. Minus signs for negative quantities must be placed in the first blank column to the left of the number.

Structural Data Cards

Card Type 1: Structure Description Card (Required)

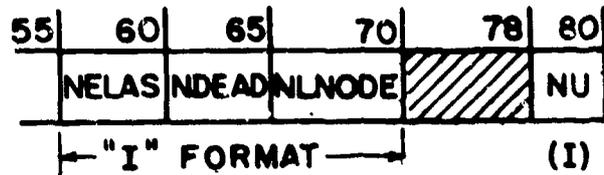
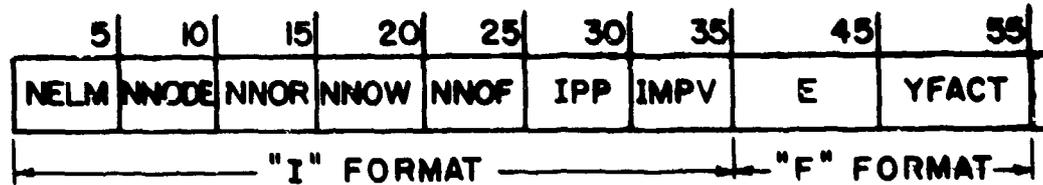
80

STRUCTURE DESCRIPTION CARD

("A" FORMAT)

This card may contain alphanumeric information in Columns 1-80.

Card Type 2: Problem Specification Card (Required)



NELM = Number of elements

NNODE = Number of nodal points

NNOR = Number of nodal points with restraints

NNOW = Number of nodal points with assigned masses
(number of mass points)

NNOF = Number of nodal points subjected to
blast loads (\leq NNOW)

IPP = 1 The program prints the nodal displacements
and rotations at every output time station.
If this parameter is omitted, the nodal
displacement history will be excluded from
the output.

IMPV = 1 The program prints the element end loads
(axial load, shear and bending moment)
at every output time station. If this
parameter is omitted, the element end
loads will be excluded from the printed
output.

E = Modulus of elasticity (SI Units: kilopascal;
U.S. Units: pounds per square inch).

YFACT = Fraction of elastic stiffness utilized after
yielding (see Section 3.3). If this parameter
is omitted, a value of 0.05 is used by the
program.

NELAS = 0 Normal Option: The program performs an elastic-plastic analysis of structure.

= 1 Inelastic behavior of elements will not be considered in the analysis.

NDEAD = 0 Normal Option: Program performs static analysis on structure prior to the start of dynamic analysis.

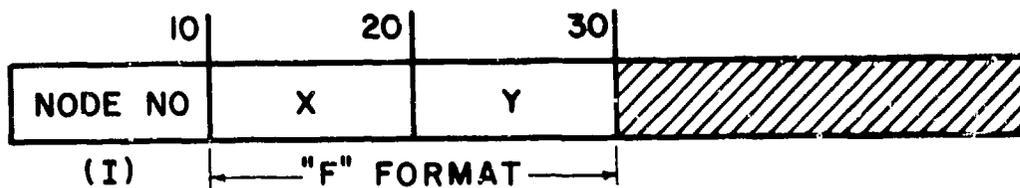
= 1 Static analysis of structure is omitted.

NLNODE = Number of nodal points with static loads. If concentrated loads are applied to some of the nodal points, an entry must be made in the field.

This parameter is omitted if concentrated nodal loads are not utilized.

NJ = US: The program accepts the input in the U.S. System of Units. If no entry is made in the field, the input is assumed to be in SI Units.

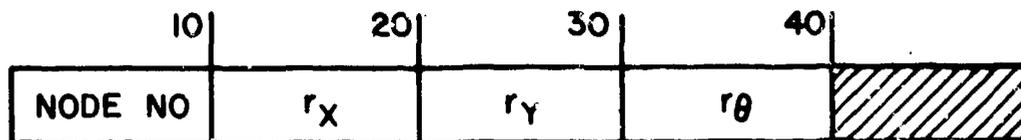
Card Type 3: Nodal Coordinates Card (one card is required for each nodal point)



NODE NO = The nodal point number

X, Y = Nodal coordinates in the global coordinate system (SI Units: meter; U.S. Units: inch)

Card Type 4: Nodal Restraint Card (Enter restraint cards only for support nodes - see Fig 57)



("I" FORMAT - ALL FIELDS)

NODE NO = The nodal point number

r_x = 1; component of the nodal displacement in the direction of the global X axis is restrained.

= 0; no restraint applied.

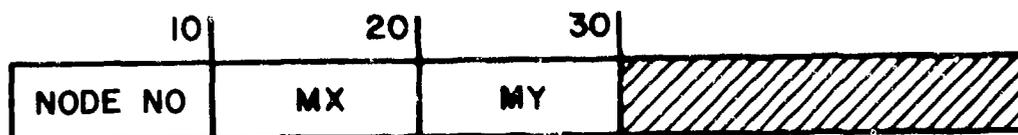
r_y = 1; component of the nodal displacement in the direction of the global Y axis is restrained.

= 0; no restraint applied.

r_θ = 1; rotation at nodal point is restrained.

= 0; no restraint applied.

Card Type 5: Nodal Mass Card (Enter cards for mass points only.)



(I) ← "F" FORMAT →

NODE NO = The nodal point number of the mass point

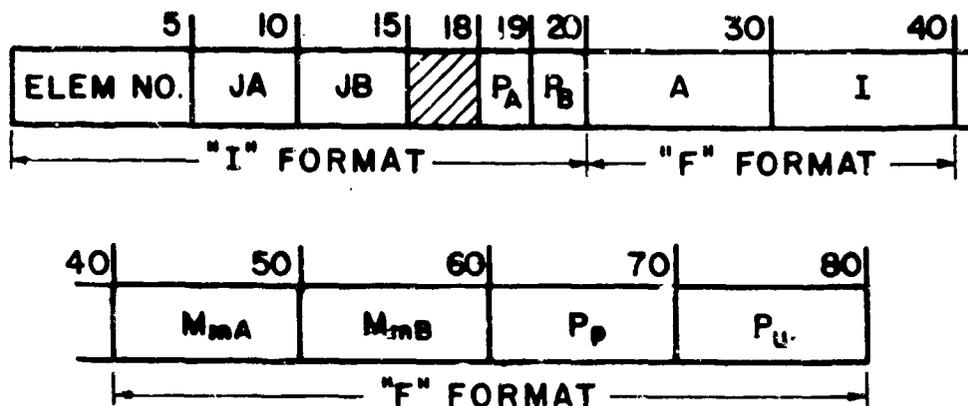
MX = Nodal mass assigned to dynamic degree-of-freedom in X-direction

MY = Nodal mass assigned to dynamic degree-of-freedom in Y-direction

UNITS: SI System - kilogram

U.S. System - pound-force (conversion to
units of mass done by program)

Card Type 6: Element Cards (one card required for each element)



ELEM NO = The element number

JA = Nodal point at end A of element
(see Fig 58)

JB = Nodal point at end B of element

P_A = 1: Local pin at end A of element
(see Fig 59)

P_B = 1: Local pin at end B of element
(see Fig 59)

A = Cross-sectional area (SI Units: centimeter
square; U.S. Units: inch square)

I = Moment of inertia (SI Units: centimeter
fourth; U.S. Units: inch fourth)

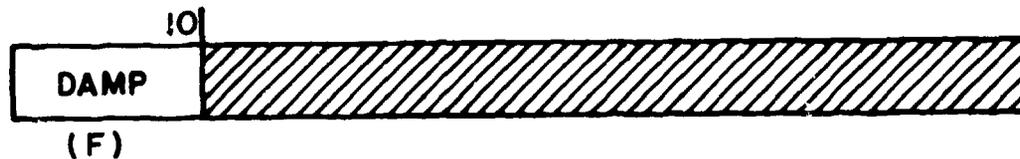
M_{mA} = Ultimate bending capacity of element at
end A (SI Units: kilonewton-meter;
U.S. Units: pound-inch)

M_{mB} = Ultimate bending capacity of element at
end B (Units: see above)

P_p = Ultimate dynamic load capacity in axial tension (SI Units: kilonewton; U.S. Units: pound)

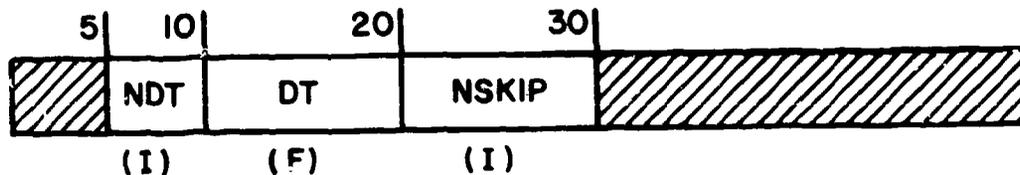
P_u = Ultimate dynamic load capacity in axial compression (Units: see above)

Card Type 7: Damping (Required)



DAMP = Percentage of damping expressed as a decimal

Card Type 8: Integration Time Card (Required) (Section 8.8)



NDT = Total number of time integration steps

DT = Integration time interval (Units: second)

NSKIP = Number of integration time steps to be skipped between printout of structure's response. For most applications, specify: $10 \leq \text{NSKIP} \leq 20$. An entry is required when the quantity one (1) is entered for the parameters IPP and IMPV on Card Type 2.

Loading Data Cards

Input Cards for Dynamic Loads

The input required for each pressure waveform consists of one Pressure Waveform Specification Card (Card Type 10) plus several P-T Cards (Card Type 11). One P-T card is required for each point in the waveform. All of the input cards for a waveform must be grouped together in the data deck.

Card Type 9: Loading Specification Card (Required)



(I)

NWF = Number of pressure waveforms

Card Type 10: Pressure Waveform Specification Card
(one card required for each waveform)



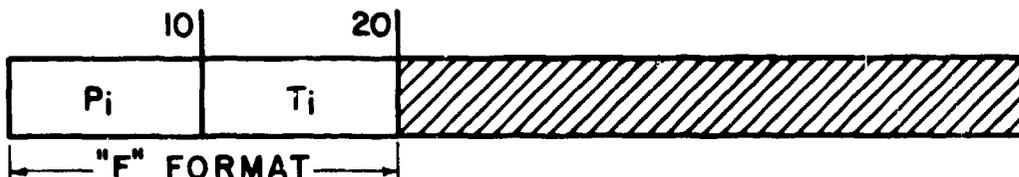
(I)

(I)

WF NO = Pressure waveform identification number

NPOINT = Number of points (pressure versus time)
in waveform

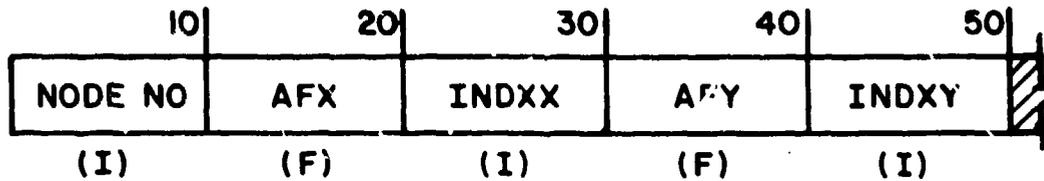
Card Type 11: P-T Card (one card required for each point in
waveform) (see Fig 60 and 61)



P_i = The pressure of point i (SI Units: kilo-
pascal; U.S. Units: pound per square inch)

T_i = The time associated with point i
(Units: second)

Card Type 12: Tributary Area Card (enter cards only for mass points subjected to blast loads)



NODE NO = The nodal point number

AFX = The tributary area associated with dynamic degree-of-freedom in the X-direction
(SI Units: meter square; U.S. Units: inch square)

INDXX = Identification number of pressure waveform associated with load in the X-direction

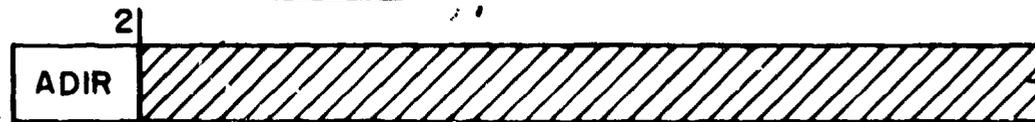
AFY = The tributary area associated with dynamic degree-of-freedom in the Y-direction
(Units: see above)

INDXY = Identification number of pressure waveform associated with load in the Y-direction

NOTE: 1. The quantities AFX and AFY can be positive or negative depending on the direction of the applied force with respect to the global coordinate system.

Input Cards for Static Loads

Card Type 13: Element Uniform Loads Direction Indicator Card (Required)



(A)

ADIR = The direction of the uniform loads specified in terms of the axis of the global coordinate system in the direction of the applied loads. If the

loads act in the negative direction of the axis, a minus sign must be included (i.e., -X, -Y). The entry must be right-adjusted in the field.

Card Type 14: Element Uniform Loads Card (one card is required for each element)



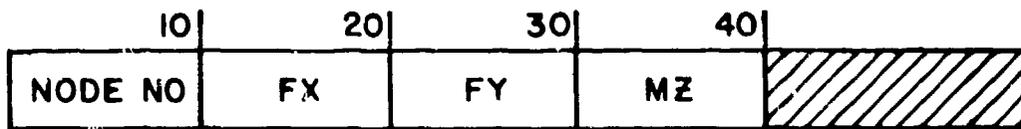
(I)

(F)

ELEM NO = The element number

WUNIF = The uniform load acting on the element
(SI Units: kilogram per meter; U.S. Units: pound-force per inch)

Card Type 15: Nodal Load Cards (enter cards only for those nodes at which static loads are acting)



(I)

"F" FORMAT

NODE NO = The nodal point number

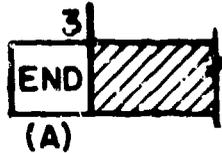
FX = Static load in X-direction (SI Units: kilogram; U.S. Units: pound-force)

FY = Static load in Y-direction (Units: see above)

MZ = Static moment around Z-axis (SI Units: kilogram-meter; U.S. Units: pound-force-inch)

NOTE: The number of these cards (Card Type 15) entered must be specified by an entry for the parameter NLNODE on Card Type 2.

End of Data Indicator



The word "END" must be punched in card columns 1 through 3.

8.14 Arrangement of Input Data Deck

A graphical representation of a typical input data deck is provided in Figure 62. Note that the data cards containing each of the principal input quantities (nodal coordinates, masses, etc.) are entered in a separate block of cards.

8.15 Multiple Job Processing

Several problems can be processed in one computer run by simply stacking the input data decks for each problem one after the other. The end of data indicator is then placed after the last data deck.

SECTION 9

COMPUTER PROGRAM OUTPUT

9.1 Introduction

This section presents a description of the printed output of DYNFA. Included also is a discussion on the utilization of the output for the design of the frame members.

The printed output of the computer program consists of four parts:

1. A summary of the units (in either the SI or U.S. System of Measurement) for the input and output parameters.
2. A summary of the input data for the analysis.
3. The response of the structure to the applied loads.
4. A compilation of significant response parameters.

The first item requires no further explanation; the others are described in the remainder of this section. Samples of the printed output are provided in Appendix B.

9.2 Summary of the Input Data

The second part of the output consists of a printed summary of the input data entered on punched cards. The data printed includes the program control parameters; the nodal coordinates, restraints and masses; the element table containing the element nodal connectivities, cross-sectional properties and capacities; the pressure waveforms and tributary areas; and the dead and live loads in the form of uniform loads on the members and concentrated loads at the nodes. Presentation of the input in this form facilitates checking of these data.

9.3 Response of the Structure

The response of the structure to either static or dynamic loads can be expressed in terms of the following parameters:

1. The displacements of the nodal points.
2. The axial loads, shears and bending moments at the ends of the elements.

3. The elastic and plastic components of the rotations at the ends of the elements.

In the printed output of DYNFA, the structure's response to static loads is expressed in terms of the nodal displacements and element end loads (parameters 1 and 2 listed above); and its response to blast loads is expressed in terms of time histories of all three of the parameters listed above. The printing sequence consists of the structure's response to the static loads followed by its response to the blast loads.

The units utilized for the output of the response are given in Table 9.

The format of the printed output of the dynamic response consists of three tabulations of data, one for each of the parameters listed above. These data are printed out at every output time station (see Section 8.8). Each output time station is identified by its respective response time which is printed just prior to the three tabulations of response data. The first tabulation of response data contains the displacements of all of the nodal points in the model. Three components of displacement are printed for each nodal point. They are the translations in the X and Y directions of the global coordinate system and the rotation about the global Z axis.

Immediately following the displacements are the element axial loads, shears and bending moments. These quantities are referenced to the local coordinate systems of the respective elements as shown in Figure 63. The quantities printed and the symbols used to identify them in the output are listed as follows:

1. The element number (ELEM)
2. The axial load at end A (PA).
3. The ratio of the axial load at end A to the axial load capacity (PA/PC). The capacity used for the computation depends upon whether the load is tension or compression.
4. The shear at end A (VA).
5. The bending moment at end A (MA).
6. The ratio of the bending moment at end A to the bending moment capacity of the element at end A (MA/MMA).

7. The shear at end B (VB).
8. The bending moment at end B (MB).
9. The ratio of the bending moment at end B to the bending moment capacity of the element at end B (MB/MMB).

All of the ratios (Items 3, 6 and 9) are printed as the absolute value of the quantity.

The third tabulation of data consists of the elastic and plastic components of the element end rotations (see Section 6.2 and Fig 37 and 38). These quantities are not printed for the static load case. The rotations are computed and printed out at both ends of each element. When the end of the element is in the elastic condition, the elastic rotation varies with time, while the plastic component remains constant at a value of zero. In the plastic condition, the elastic component remains constant, while the plastic component varies. The plastic rotation increases in magnitude until a peak value is reached, at which time the program records the peak value and resets the plastic component to zero. Successive deformations are recorded as elastic rotations until the element, once again, attains the plastic condition.

In some cases, anomalies occur in the time histories of the element end rotations. The most common occurrence is a decrease in the moment while the element is in the plastic condition. This is caused by an increase in the axial load occurring simultaneously with the decrease in the bending moment. As discussed in Section 3.3, the time dependency of the bending moment is not related to that of the axial load. In addition, the rate of change of the axial load is much greater than that of the bending moment; therefore, it is conceivable, upon inspecting Equation (10) of Section 3.3, that the relative decrease in the moment could be offset by the relative increase in the axial load, thereby indicating that the element remains in the plastic condition while the moment is decreasing. The event described generally occurs after the peak plastic rotation has been achieved. It usually exists for a relatively short duration and therefore has a negligible impact on the overall results of the analysis.

Another anomaly that might be encountered is the apparent fluctuation of an element into and out of the plastic condition. This generally occurs in the early stages of plastic behavior. It is again caused by the differing time variations of the axial loads and bending moments. Generally these fluctuations are short.

in duration, and as such, they have a negligible impact on the results of the analysis and can be ignored.

9.4 Compilation of the Significant Response Parameters

A compilation of the significant response parameters is printed at the end of the output. The data printed consists of the maxima and minima of various response parameters together with the response times at which the maxima and minima are recorded.

Three tabulations of data are provided. The first two contain response data related to the elements. The data printed in the first tabulation, together with the symbols to identify each parameter in the output, are listed below:

1. The maximum and minimum axial load (MAXP) together with the corresponding bending moment (ASSOC.M) and shears (ASSOC.V).
2. The maximum and minimum bending moments (MAXM) together with the corresponding axial loads (ASSOC.P) and shears (ASSOC.V). Included also is the moment capacity of each element (printed under the column heading MM) and the ratios of the peak moments to the moment capacities (MAXM/MM).

The above quantities are printed for both ends of each element.

The second tabulation contains the maximum and minimum values of the elastic and plastic element end rotations, which are printed under the column headings ELAS and PLAS, respectively. These data are provided for both ends of each element. Provided also are the maximum and minimum ductility ratios computed using the tabulated maximum and minimum element end rotations. The ductility ratios are listed under the column heading D.R. Included with the ductility ratios are the axial load to tensile capacity ratios (printed under the column heading P/PP which occur when the maximum and/or minimum plastic rotations are achieved. An indicator is provided with these ratios which specifies whether the axial load used in the computation is either a tensile or compressive load. The indicators used are letters: "T" for tension and "C" for compression.

The third tabulation consists of the maximum and minimum values of the three components (translations in the X and Y axes and rotation about the Z axis) of the nodal displacements. Here, the column headings used are X and Y for the translations and R for the rotations.

9.5 Use of the Printed Output

General

The output of DYNFA is used for four primary purposes:

1. To detect errors in either the input data or the analysis.
2. To evaluate the response of the structure.
3. To determine the adequacy of the frame members on the basis of the frame design criteria given in Section 6.
4. To design the connections of the frame members.

The first two are explained in Problem A.5 of Appendix A. The third and fourth are discussed in the subsequent paragraphs.

In order to ascertain the adequacy of the frame members, three groups of response quantities must be extracted from the printed output and then compared to the design criteria specified in Section 6.

Sideway Deflections

The first group consists of the sideway deflections of each story of the frame. These quantities are determined by computing the maximum differential displacements between stories of the structure. Since all of the stories usually attain their peak sideway deflection at or about the same time in the response, the maximum differential displacements generally can be computed using the tabulated maxima and minima of the nodal displacements printed at the end of the output. Limiting values of the sideway deflection are given in Section 6.1.

Chordal Angles

The second group of response data consists of the maximum values of the chordal angle, θ , for all of the exterior members of the frames. The chordal angles are the end rotations of the individual members with respect to a chord joining the member ends, as illustrated in Figures 64 and 65. Limiting values for the chordal angles are specified in Section 6.1.

A chordal angle is determined by computing the differential displacement between a nodal point at the midspan of a member and

a chord joining the member ends. Dividing the differential displacement by one half the member length yields the tangent of the chordal angle.

The direction of the nodal displacements, utilized for the computation of the chordal angle, must be perpendicular to the initial position of the longitudinal axis of the member. Since most frame members are parallel to one of the global axes, only one component of the nodal displacement (either the X or Y component, as the case may be) is required for the computation. However, if the member is sloped, both translational components of the nodal displacement are used.

The computation of the maximum chordal angles for an exterior girder is relatively simple as the vertical deflections at the ends of the member are normally quite small. Consequently, the chord joining the ends of the member remains closely aligned with the original position of the longitudinal axis of the member. Therefore, the maximum differential displacement of the member with respect to the chord corresponds to the maximum vertical displacement of the nodal point at the midspan of the member. These quantities can be extracted directly from the tabulated maxima and minima of the nodal displacements printed at the end of the output. Figure 64 illustrates the measurement of the chordal angle for a girder.

The computation of the chordal angles for exterior columns and sloped roof girders is more involved as the deflections at the ends of these members are of the same order of magnitude as the midspan deflections; hence, the maximum differential displacement is usually less than the maximum absolute displacement of the nodal point at the midspan of the member. Here, the differential displacement at midspan is computed at several time stations in order to determine the maximum value. In most cases, the peak chordal rotation occurs during the time interval in which the member is subjected to the blast pressures. For a given member, the time interval to be investigated commences with the initial application of the loading on the member and extends to a few time increments beyond the time at which the member attains its peak plastic response. Figure 65 illustrates the measurement of the chordal angle for an exterior column.

Ductility Ratios

The ductility criteria given in Section 6 limits the magnitudes of the ductility ratios that can be realized in the response. The peak plastic and elastic rotations at the element ends are used to compute these quantities. Maximum ductility

ratios are computed and printed by DYNFA at both ends of every element. To use these data for investigating a member, extract from the output the ductility ratio and corresponding axial load ratio for all of the elements representing the member. Also, note from the output whether the axial loads are either in tension or compression. Compare the computed ductility ratios with the limits specified in Sections 6.3 and 6.4.

Establishing Adequacy of Frame

The members of the frame are considered adequate if all of the design criteria are satisfied. When one or more of the limiting values of the criteria are exceeded by more than 15 percent, the member or members in question must be increased in size and the analysis of the frame must be repeated. If a severe condition of overdesign is apparent, the overdesigned members should be decreased in size and the frame analysis repeated.

Appendix A provides two examples which demonstrate the use of the printed output in the design of frames.

Connection Design

The fourth group of data extracted from the output consists of the maximum values of the moments, shears and axial loads which act at the ends of the members. These quantities are used to design the connections in the frame. The connections considered are the corner frame connections, the girder-to-column connections and the column base connections. Since the maximum values of the moments, shears and axial loads do not occur simultaneously, at least three conditions must be considered in the design. Each condition considered consists of the peak value of one of these quantities together with the values of the other two which occur simultaneously. The maxima and minima of the axial loads and bending moments are tabulated at the end of the printed output. The peak shear (and corresponding axial loads and bending moments) can be extracted from the printed output of the response history.

The connections should be designed in accordance with the provisions in Chapter 6 of Reference 1. Procedures and equations for the design of various types of standard frame connections are contained in the many texts on the design of steel structures (see References 14 and 20). Examples of some of typical framing connections are provided in Appendix C of Reference 1.

SECTION 10

CONCLUSIONS AND RECOMMENDATIONS

The criteria and procedures presented in this report provide a rational basis for the analysis and design of the primary structural frames of steel buildings subjected to the blast environment generated by a high-explosive detonation. The primary emphasis is on structural steel applications for acceptor structures located in the low-to-intermediate pressure level range.

It is recommended that this material be implemented in the blast-resistant design of structures within facilities for the manufacture and storage of explosive materials.

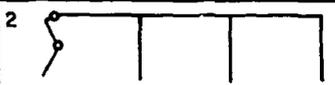
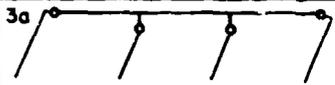
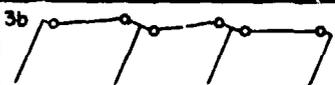
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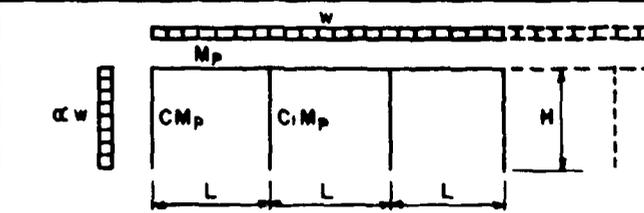
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Table 1

Collapse mechanisms for rigid frames with fixed and pinned bases

Collapse Mechanism	Plastic Moment N_p	
	Pinned Bases	Fixed Bases
 <p>BEAM MECHANISM</p>	$\frac{wL^2}{16}$	$\frac{wL^2}{16}$
 <p>BEAM MECHANISM</p>	$\frac{\alpha w H^2}{4(2C+1)}$	$\frac{\alpha w H^2}{4(3C+1)}$
 <p>PANEL MECHANISM</p>	$\frac{\alpha w H^2}{2} \cdot \frac{1}{2+(n-1)C_1}$ ($C_1 < 2$) [*]	$\frac{\alpha w H^2}{4} \cdot \frac{1}{1+(n-1)C_1+C}$ ($C_1 < 2$) [*]
 <p>PANEL MECHANISM</p>	$\frac{\alpha w H^2}{4n}$ ($C_1 \geq 2$) [*]	$\frac{\alpha w H^2}{2} \cdot \frac{1}{2(n+C)+(n-1)C_1}$ ($C_1 \geq 2$) [*]
 <p>COMBINED MECHANISM</p>	$\frac{w}{8n} (\alpha H^2 + \frac{n}{2} L^2)$	$\frac{w}{2} \cdot \frac{\alpha H^2 + \frac{n}{2} L^2}{2(2n+C)+(n-1)C_1}$
 <p>COMBINED MECHANISM</p>	$\frac{3}{8} \alpha w H^2$ $C + \frac{1}{2} + \frac{C_1(n-1)}{2}$ ($C_1 < 2$) [*]	$\frac{3}{8} \alpha w H^2$ $\frac{5}{2} C + (n-1)C_1 + \frac{1}{2}$ ($C_1 < 2$) [*]
 <p>COMBINED MECHANISM</p>	$\frac{3}{8} \alpha w H^2$ $C + (n - \frac{1}{2})$ ($C_1 \geq 2$) [*]	$\frac{3}{8} \alpha w H^2$ $\frac{5}{2} C + (n-1) \frac{C_1}{2} + (n - \frac{1}{2})$ ($C_1 \geq 2$) [*]
 <p>COMBINED MECHANISM</p>	$\frac{w}{8} \left[\frac{3\alpha H^2 + (n-1)L^2}{C + (2n - \frac{3}{2})} \right]$	$\frac{w}{8} \left[\frac{3\alpha H^2 + (n-1)L^2}{\frac{5}{2} C + (n-1) \frac{C_1}{2} + (2n - \frac{3}{2})} \right]$

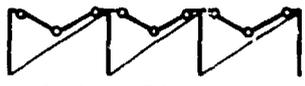
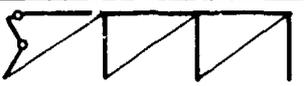
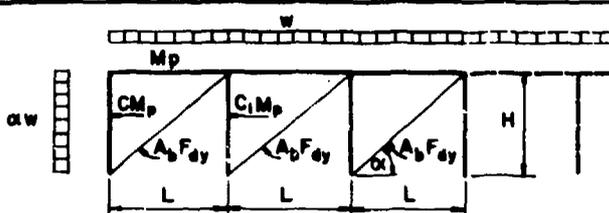


n = Number of bays = 1, 2, 3...
 w = Uniform equivalent static load

* For $C_1 = 2$ hinges form in the girders and columns at interior joints.

Table 2

Collapse mechanisms for rigid frames with supplementary bracing and pinned bases

Collapse Mechanism	Plastic Moment M_p
 <p>BEAM MECHANISM</p>	$\frac{wL^2}{16}$
 <p>BEAM MECHANISM</p>	$\frac{\alpha w H^2}{4(2C+1)}$
 <p>PANEL MECHANISM</p>	$\left(\frac{\alpha w H^2}{2} - m A_b F_{dy} H \cos \alpha \right) \cdot \frac{1}{2+(n-1)C_1}$ ($C_1 \geq 2$)*
 <p>PANEL MECHANISM</p>	$\frac{\alpha w H^2}{4n} - \frac{m A_b F_{dy} H \cos \alpha}{2n}$ ($C_1 \geq 2$)*
 <p>COMBINED MECHANISM</p>	$\frac{w}{8n} \left(\alpha H^2 + \frac{n}{2} L^2 \right) - \frac{m A_b F_{dy} H \cos \alpha}{4n}$
 <p>COMBINED MECHANISM</p>	$\frac{\alpha w H^2 - \frac{m}{2} A_b F_{dy} H \cos \alpha}{C + \frac{1}{2} + \frac{C_1}{2}(n-1)}$ ($C_1 \geq 2$)*
 <p>COMBINED MECHANISM</p>	$\frac{\alpha w H^2 - \frac{m}{2} A_b F_{dy} H \cos \alpha}{C + (n - \frac{1}{2})}$ ($C_1 \geq 2$)*
 <p>COMBINED MECHANISM</p>	$\frac{\alpha w \left[3\alpha H^2 + (n-1)L \right] - \frac{m}{2} A_b F_{dy} H \cos \alpha}{C + (2n - \frac{3}{2})}$
 <p>n = Number of bays = 1, 2, 3... w = Uniform equivalent static load</p>	

* For $C_1 = 2$ hinges form in the girders and columns at interior joints.

Table 3

Collapse mechanisms for frames with supplementary bracing, non-rigid girder-to-column connections and pinned bases

Collapse Mechanism	Resistance	Framing Type
 <p>BEAM MECHANISM EXTERIOR GIRDER</p>	$M_p = wL^2/8$ $M_p = wL^2/12$ $M_p = wL^2/16$	(1) (2) (3)
 <p>BEAM MECHANISM INTERIOR GIRDER</p>	$M_p = wL^2/8$ $M_p = wL^2/16$	(1) (2) & (3)
 <p>BEAM MECHANISM BLASTWARD COLUMN</p>	$M_p = \alpha wH^2/8$ $M_p = \frac{\alpha wH^2}{4(2C+1)}$	(1) & (2) (3)
 <p>PANEL MECHANISM</p>	$A_b F_{dy} = \alpha wH/2m \cos \alpha$ $A_b F_{dy} = \frac{\alpha wH}{2m \cos \alpha} - \frac{2M_p}{mH \cos \alpha}$	(1) & (2) (3)
 <p>COMBINED MECHANISM</p>	$A_b F_{dy} = \frac{3\alpha wH}{4m \cos \alpha} - \frac{(2C+1)M_p}{mH \cos \alpha}$	(3)

GIRDER FRAMING TYPE:

- (1) GIRDER SIMPLY SUPPORTED BETWEEN COLUMNS
- (2) GIRDER CONTINUOUS OVER COLUMNS
- (3) GIRDER CONTINUOUS OVER COLUMNS AND RIGIDLY CONNECTED TO EXTERIOR COLUMNS ONLY

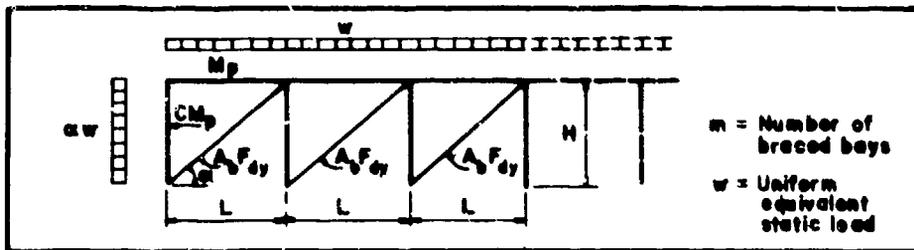


Table 4

Dynamic load factors (DLF) and equivalent static loads for preliminary design

Collapse mechanism*	Structure	
	Reusable	Non-reusable
Beam mechanism 1	1.0	0.80
Beam mechanism 2	R_α	$0.80R_\alpha$
Panel or combined	$0.5R_\alpha$	$0.35R_\alpha$

where $R_\alpha = 1.0 - b_h \sin \alpha_f / 8S$

= 1.0 for normal shock wave

α_f = angle of incidence between shock front and blastward wall

S = the minimum of:

1. height of structure
2. distance from frame to leeward end of wall
3. sum of:
 - a. distance from frame to blastward end of wall
 - b. length of adjacent blastward wall

Equivalent static vertical load = $q_v \times \text{DLF} = w$

Equivalent static horizontal load = $q_h \times \text{DLF} = \alpha w$

*Refer to Tables 1 through 3 for definition of beam, panel and combined mechanisms.

Table 5

Limiting width-thickness ratios for the compression flanges of single-web members subjected to plastic bending

F_y (kPa)	F_y (ksi)	$b_f/2t_f$
248184	36	8.5
289548	42	8.0
310230	45	7.4
344700	50	7.0
379170	55	6.6
413640	60	6.3
448110	65	6.0

where F_y is the specified minimum static yield stress for the steel, b_f is the flange width and t_f is the flange thickness.

Table 6

Sample bi-axial bending moment capacities

Column Node	DYNFA results for first group of analyses		Uni-axial capacities		Nodal reduction factors	Reduced capacities for second group of analyses	
	$ M_x $ (kN-m)	$ M_y $ (kN-m)	M_{mx} (kN-m)	M_{my} (kN-m)		$\bar{M}_{mx} = M_x /R_n$ (kN-m)	$\bar{M}_{my} = M_y /R_n$ (kN-m)
1	175	17	300	100	0.75	233	23
2	215	33	300	100	1.05	205	31
3	200	50	300	100	1.17	171	43

Note: $(|M_x|/R_n)M_{mx} + (|M_y|/R_n)M_{my} = 1$; hence, $\bar{M}_{mx} \leq M_{mx}$ and $\bar{M}_{my} \leq M_{my}$.

Table 7
 Values of constant "C" for Equation (35)

Support conditions	Number of intermediate mass points			
	1	2	3	4
Simple	1.57	6.28	14.1	25.2
Fixed	3.56	9.82	19.2	31.8
Fixed-Hinged	2.45	7.95	16.6	28.4

Note: The units to be utilized with these constants are:

mass - kilogram

length - meter

modulus of elasticity - kilopascal

moment of inertia - centimeter to the fourth power

natural period - second

Table 8

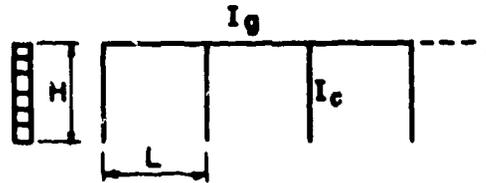
Stiffness factors for single-story multi-bay frames
subjected to uniform horizontal loading

$$\text{STIFFNESS FACTOR } K = \frac{EI_{ca}}{H^3} \cdot C_2 \cdot [1 + (0.7 - 0.1\beta)(n-1)]$$

n = Number of Bays

β = Base Fixity Factor^a

$$D = \frac{I_g/L}{I_{ca}(0.75 + 0.25\beta)/H}$$



I_{ca} = Average Column Moment of Inertia.
 $= \sum I_c / (n+1)$

D	C ₂		
	$\beta = 1.0$	$\beta = 0.5^b$	$\beta = 0$
0.25	26.7	14.9	3.06
0.50	32.0	17.8	4.65
1.00	37.3	20.6	6.04

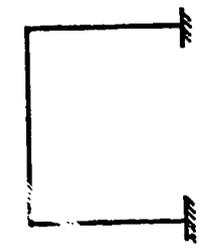
^a Values of C₂ are Approximate for this β

^b $\beta = 1.0$ For Fixed Base
 $= 0.0$ For Hinged Base

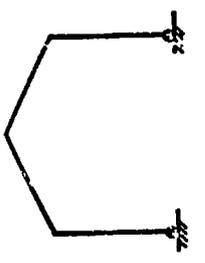
Table 9

Units for parameters in printed output of structural response

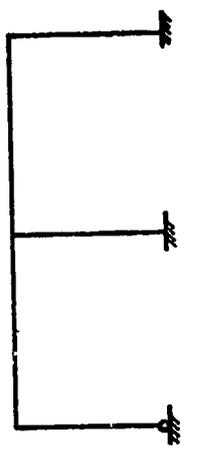
Parameter	Units	
	SI System	U.S. System
Time	Second	Second
Displacements:		
Translation	Millimeter	Inch
Rotation	Degree	Degree
Element Loads:		
Axial Load	Kilonewton	Pound
Shear	Kilonewton	Pound
Bending Moment	Kilonewton-meter	Pound-inch



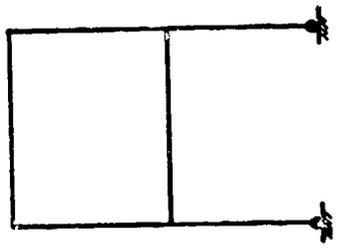
RECTANGULAR FRAME
(Rigid Support)



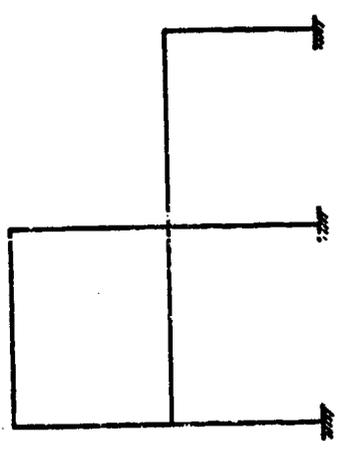
GABLE FRAME
(Pinned Support)



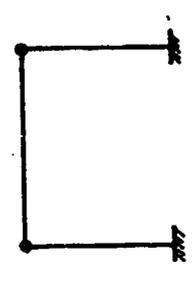
MULTI-BAY FRAME
(Combined Rigid and Pinned Supports)



MULTI-STORY FRAME

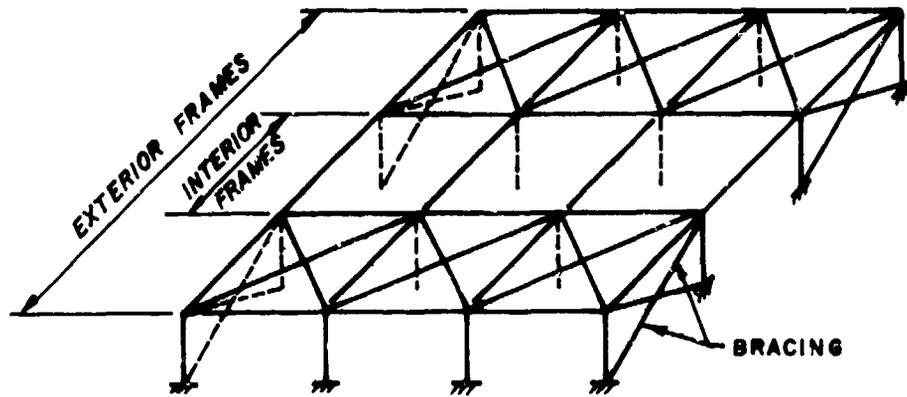


MULTI-STORY AND MULTI-BAY FRAME

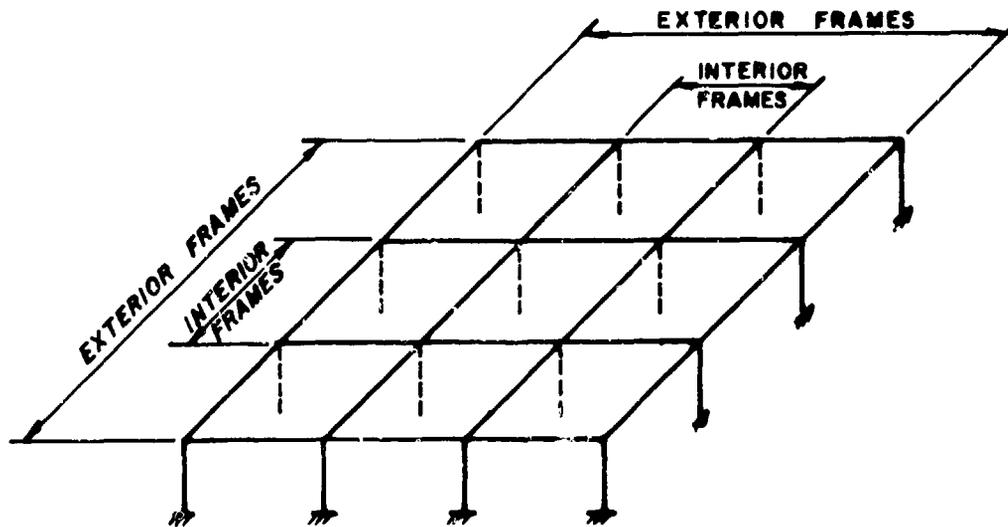


PINNED CONNECTED FRAME

Fig 1 Typical frame structures

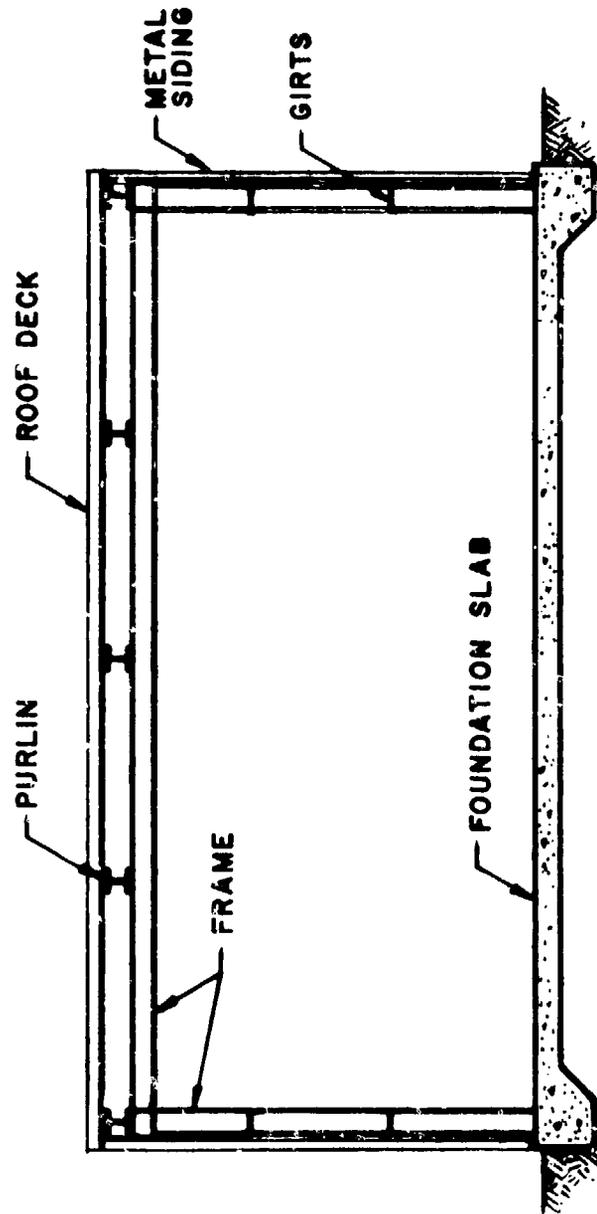


a) ONE DIRECTIONAL RIGID FRAMING



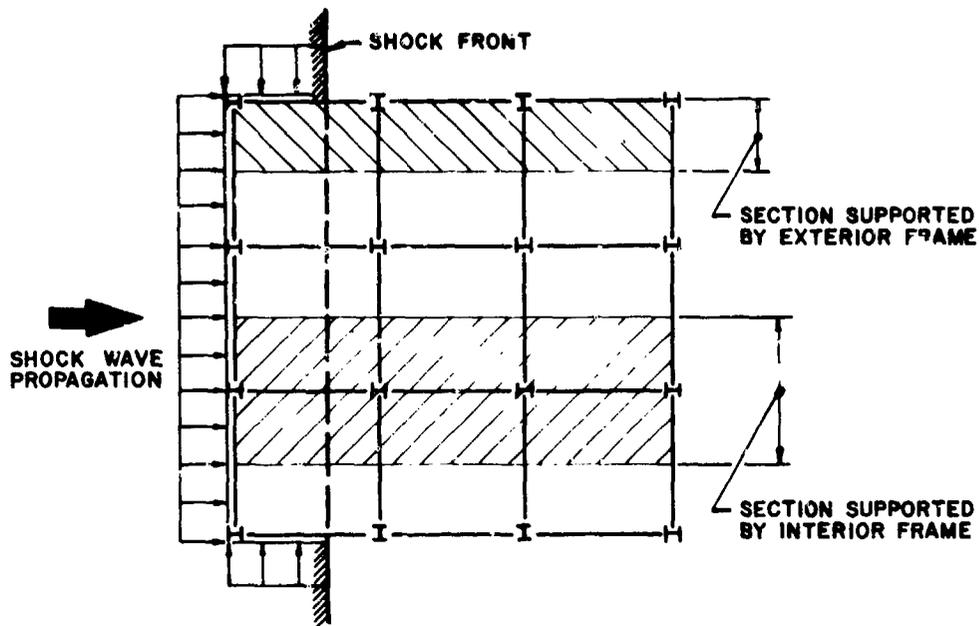
b) TWO DIRECTIONAL RIGID FRAMING

Fig 2 Framing systems

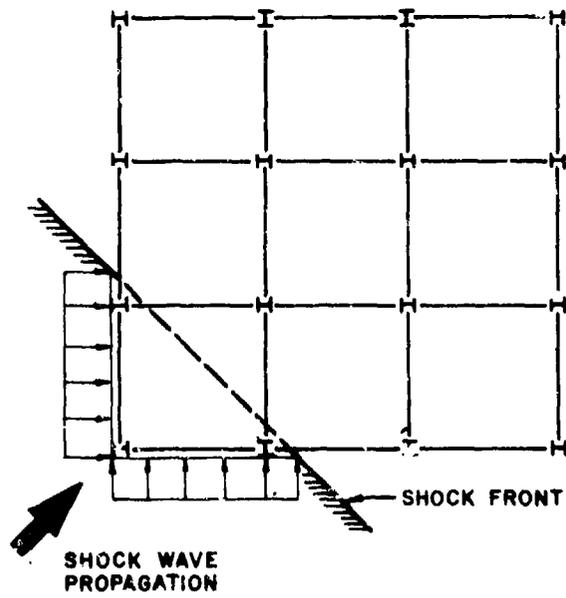


ELEVATION

Fig 3 Elevation of a typical frame



a) NORMAL SHOCK WAVE



b) QUARTERING SHOCK WAVE

Fig 4 Loading conditions for bi-axial bending

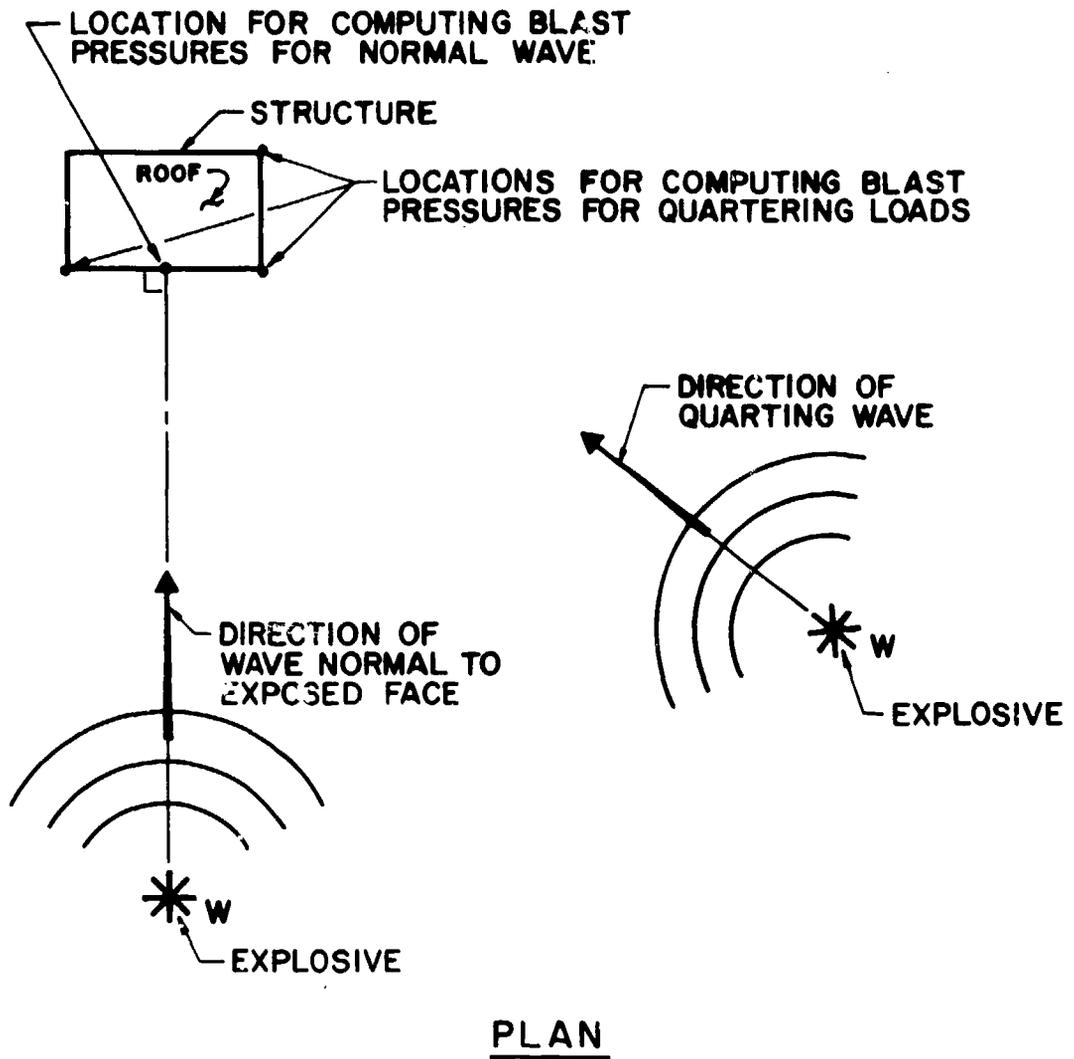
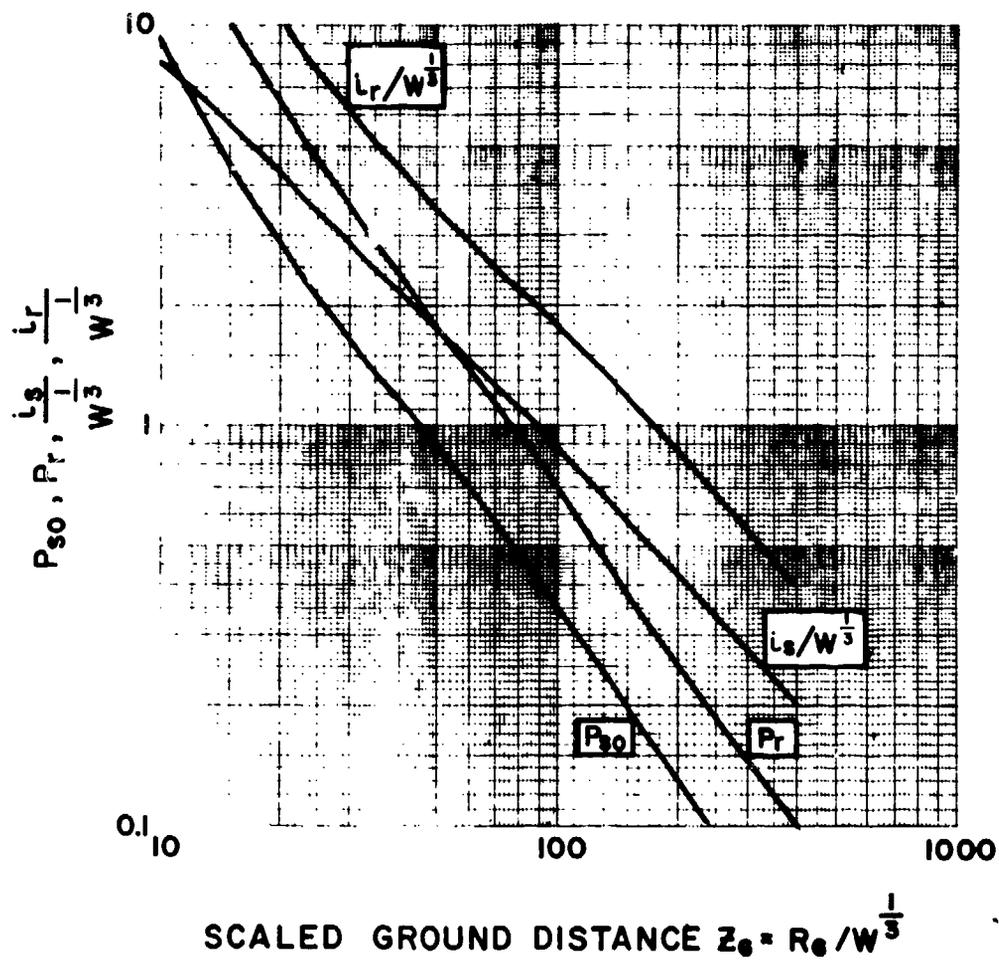


Fig 5 Locations for computing blast pressures for preliminary frame design



- P_{s0} = Peak positive incident pressure, psi
 P_r = Peak positive normal reflected pressure, psi
 $i_s/W^{1/3}$ = Scaled unit positive incident impulse, psi-ms/lb^{1/3}
 $i_r/W^{1/3}$ = Scaled unit positive normal reflected impulse, psi-ms/lb^{1/3}

Fig 6 Supplementary data for shock-wave parameters for hemispherical TNT surface explosion at sea level

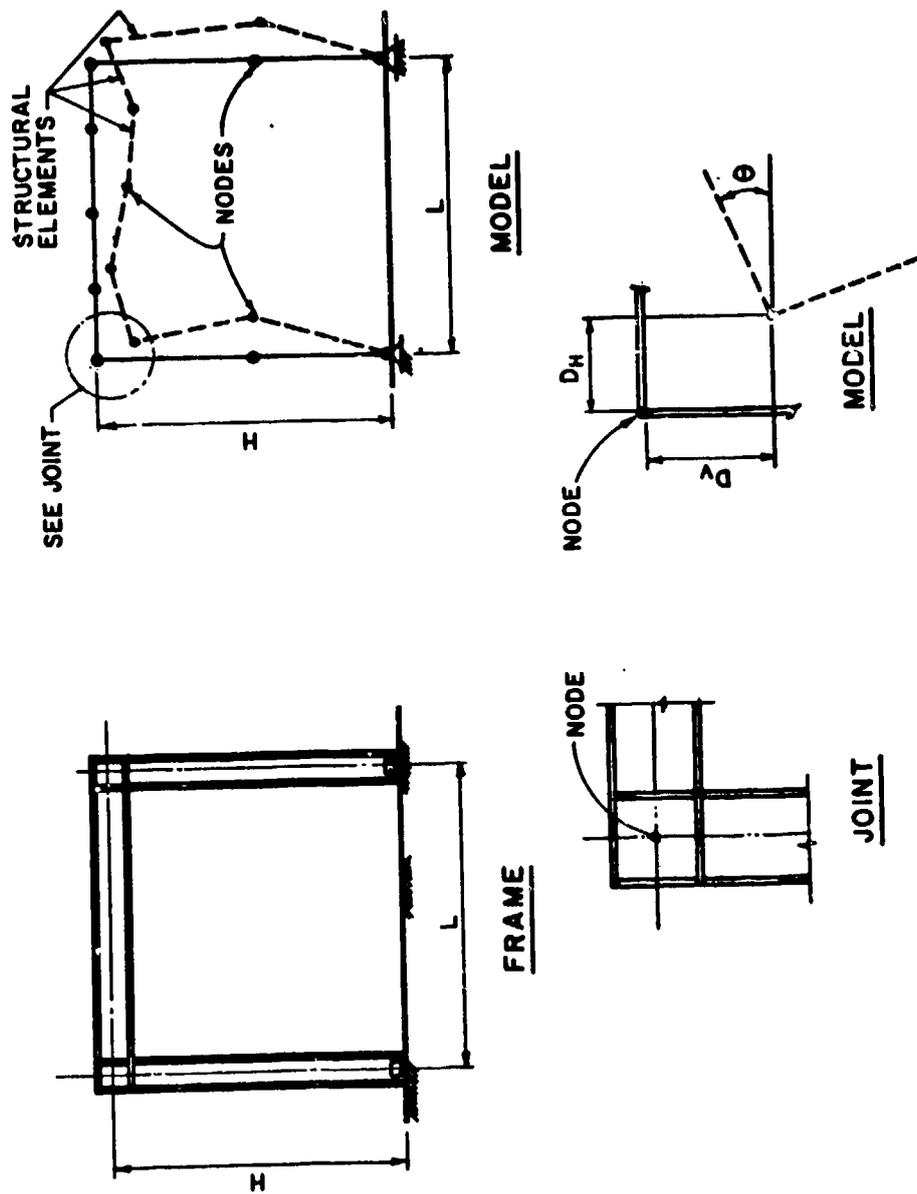


Fig 7 Lumped parameter representation of a typical frame

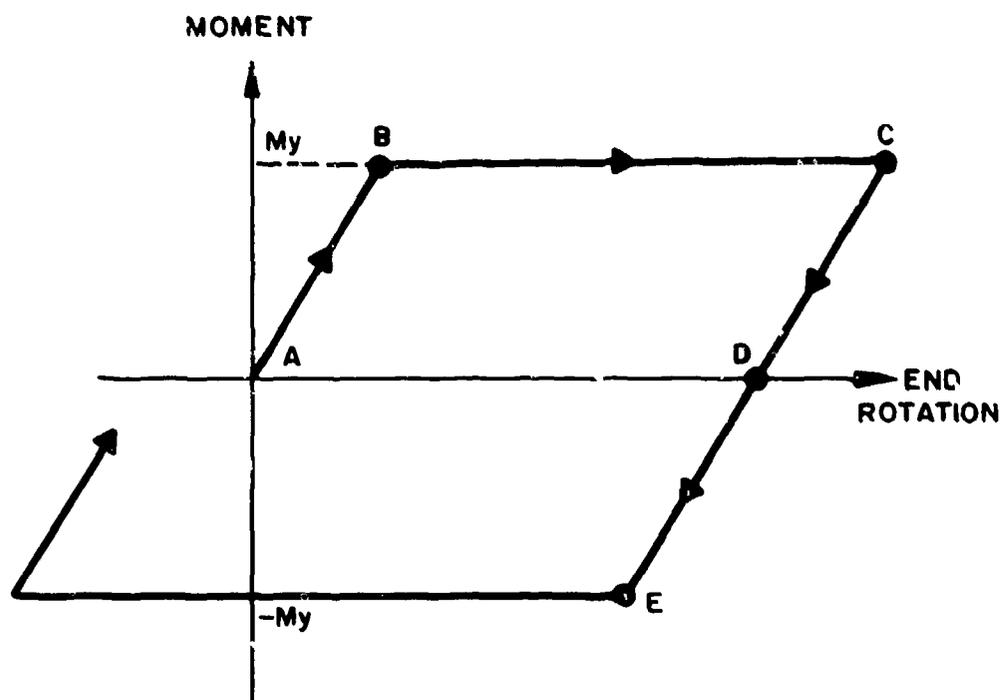
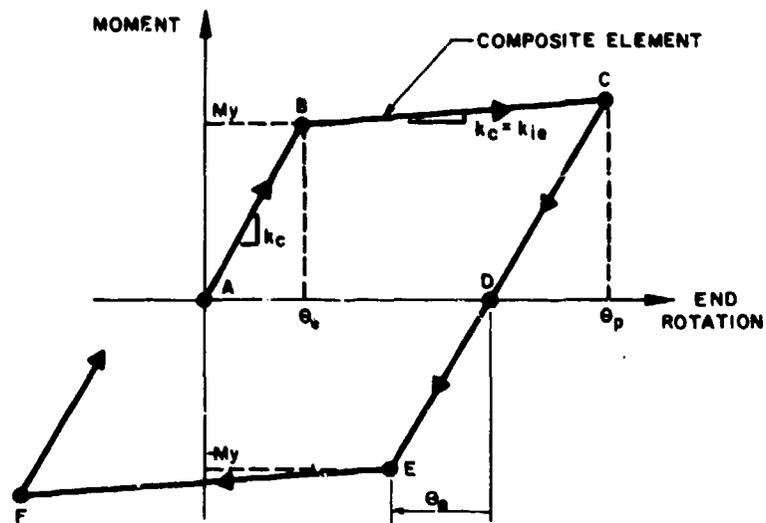
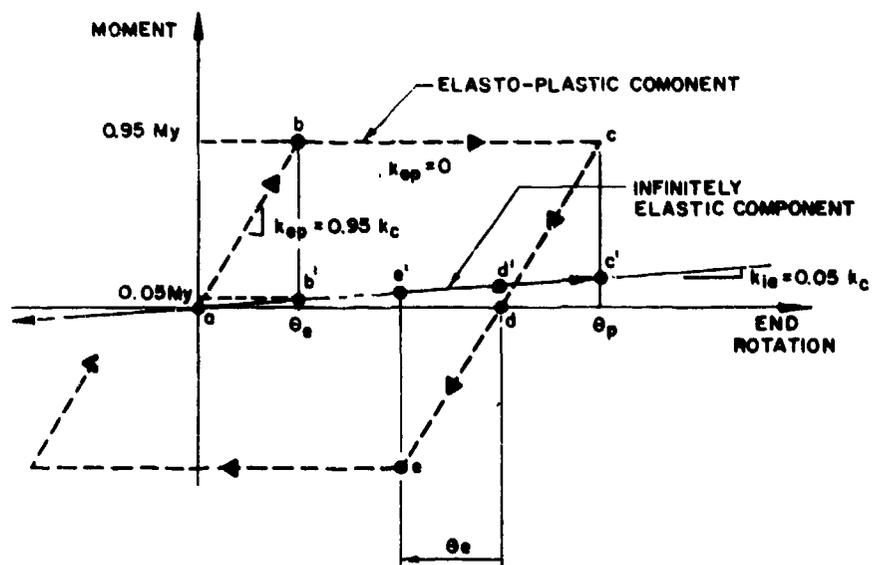


Fig 8 Elasto-plastic moment versus end rotation relationship for constant axial load

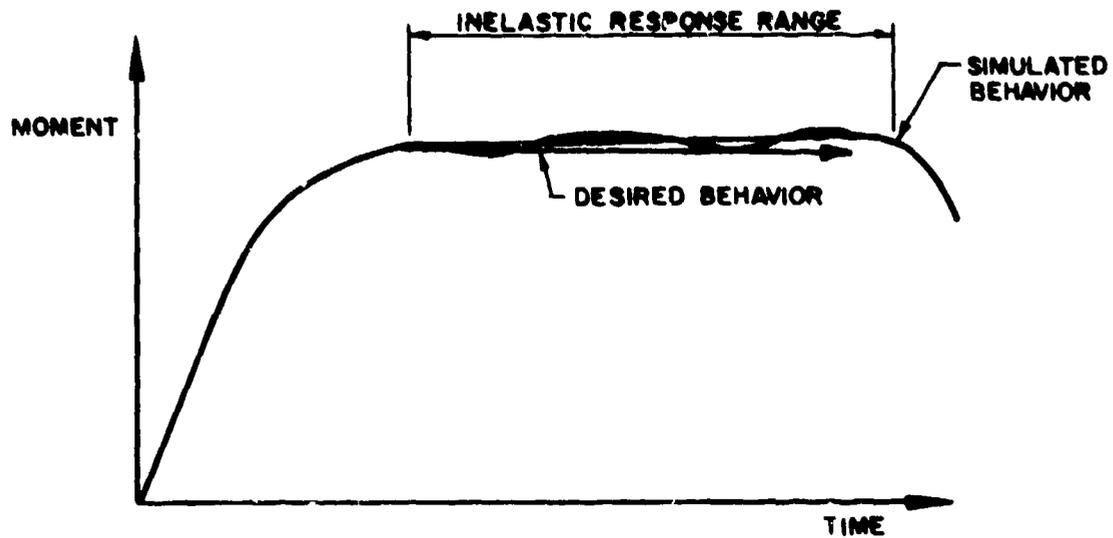


a) COMPOSITE MOMENT VERSUS ROTATION RELATIONSHIP OF COMPONENTS

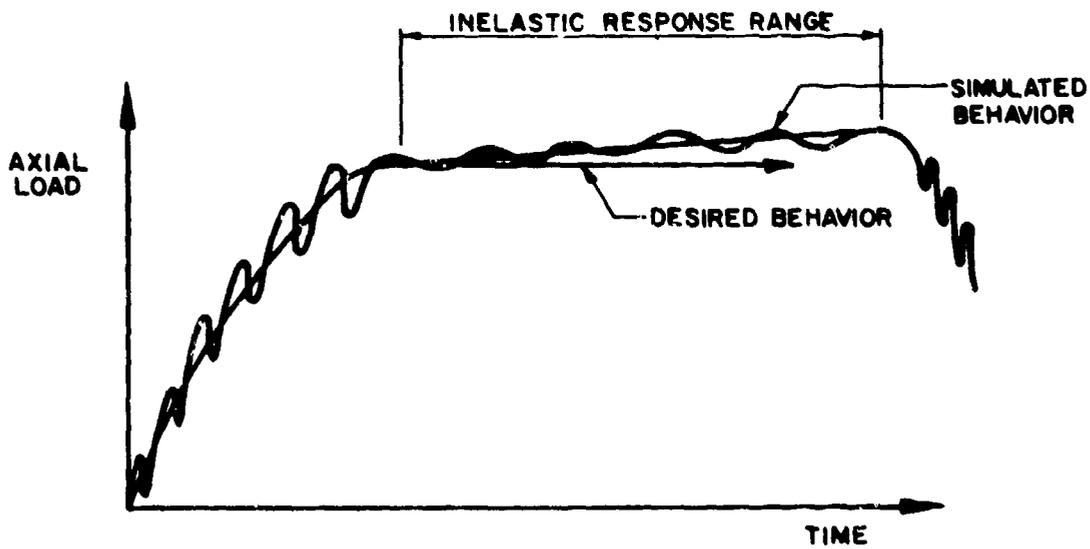


b) INDIVIDUAL MOMENT VERSUS ROTATION RELATIONSHIP OF COMPONENTS

Fig 9 Behavior of component elements



a) MOMENT RESPONSE HISTORY OF ELEMENT



b) AXIAL LOAD RESPONSE HISTORY OF ELEMENT

Fig 10 Response history of element

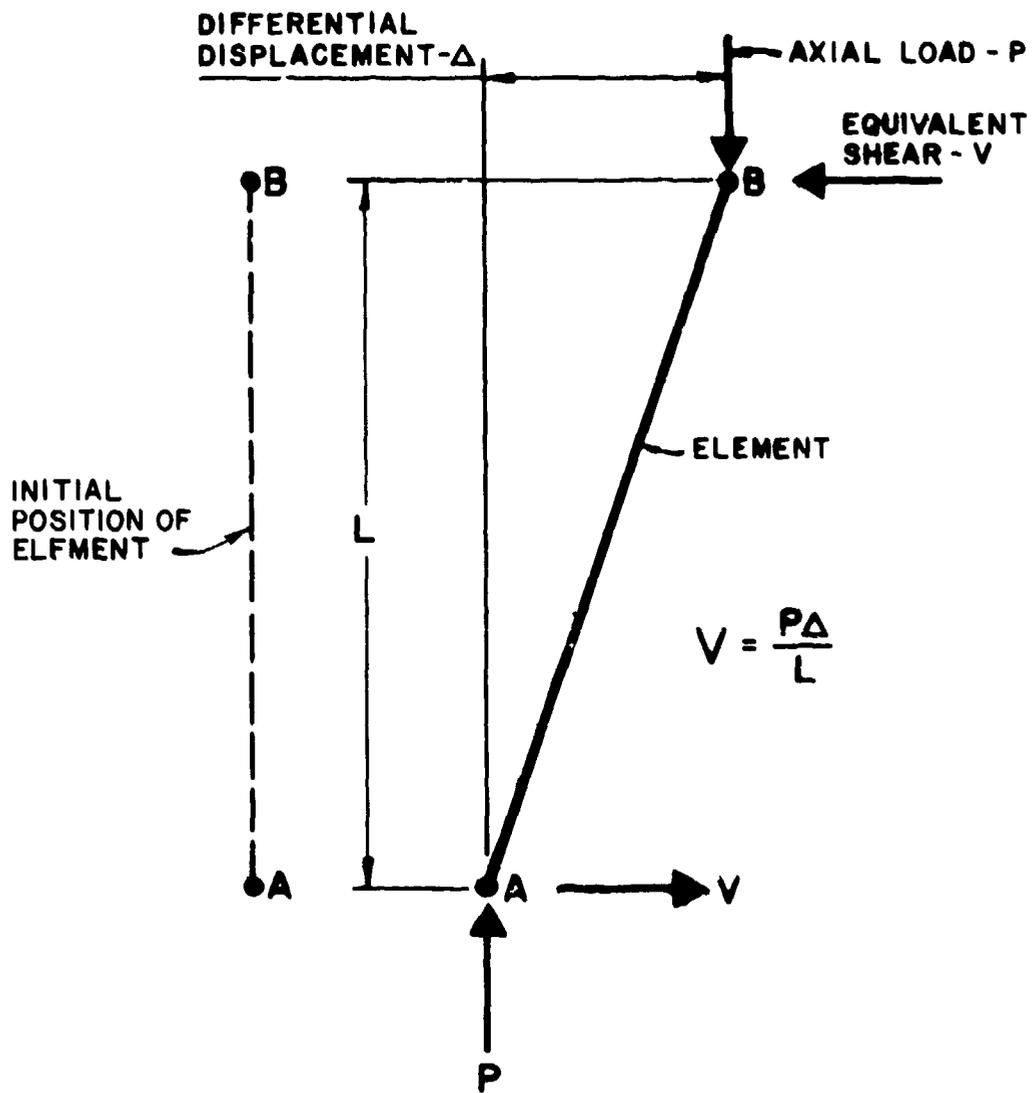


Fig 11 Calculation of second order effects

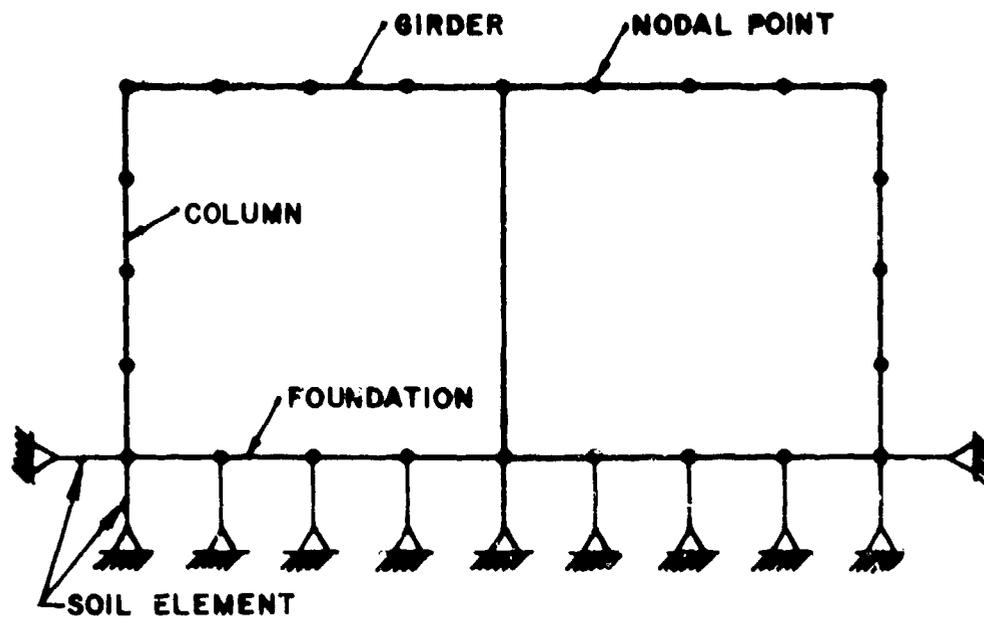


Fig 12 Model of frame including foundation and soil

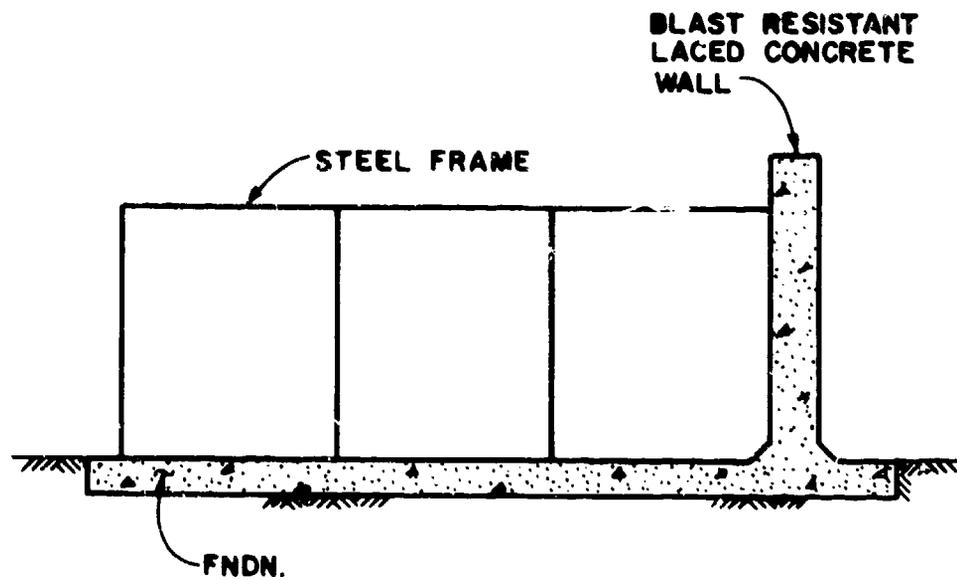
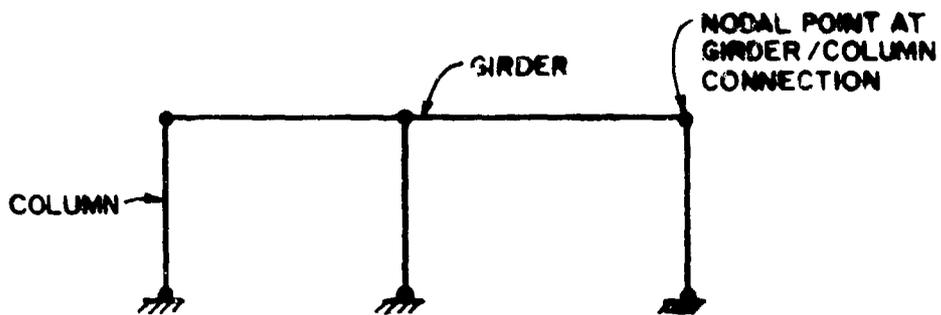
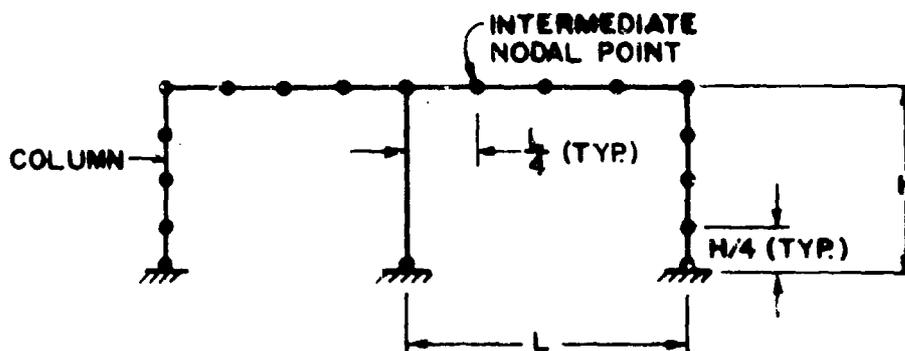


Fig 13 Laterally supported frame

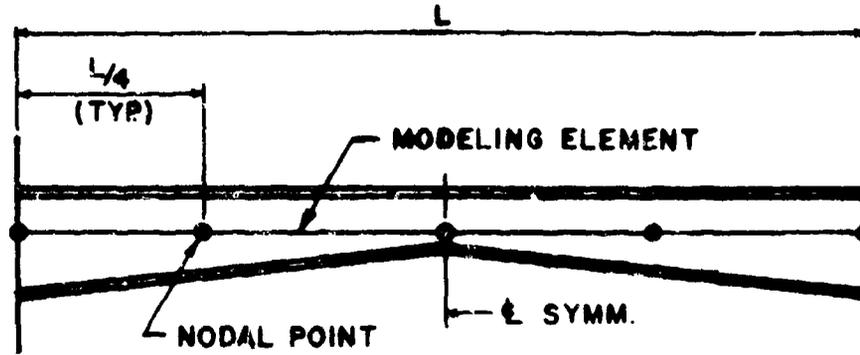


a) PLACEMENT OF NODAL POINTS TO DEVELOP SIDESWAY MODE OF FRAME

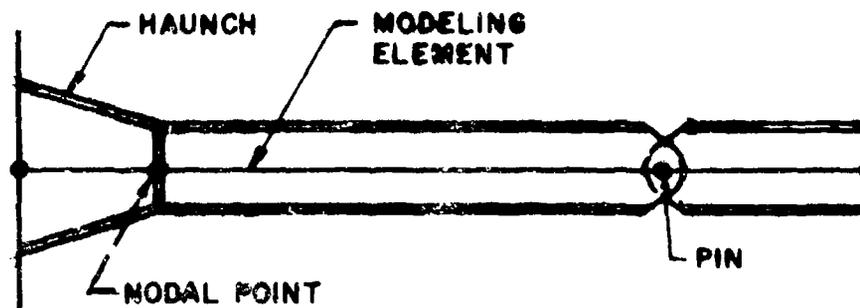


b) PLACEMENT OF NODAL POINTS TO DEVELOP INDIVIDUAL BENDING MODES OF EXTERIOR MEMBERS

Fig 14 Locations of nodal points in model

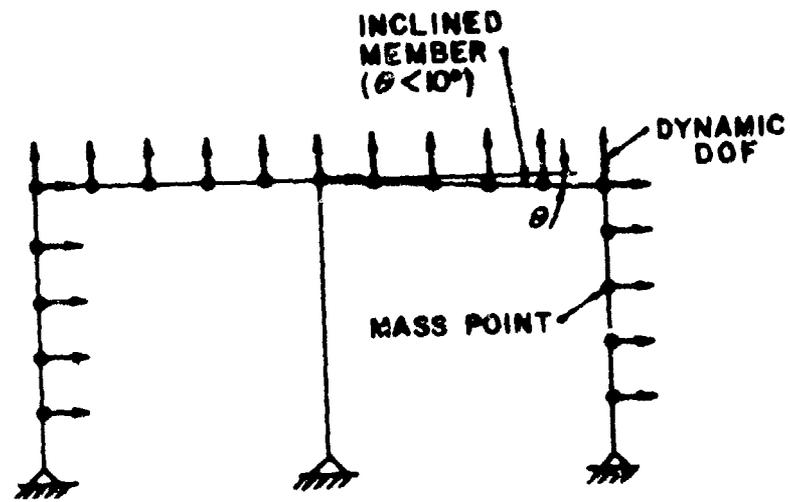


a) GRADUALLY TAPERED MEMBER

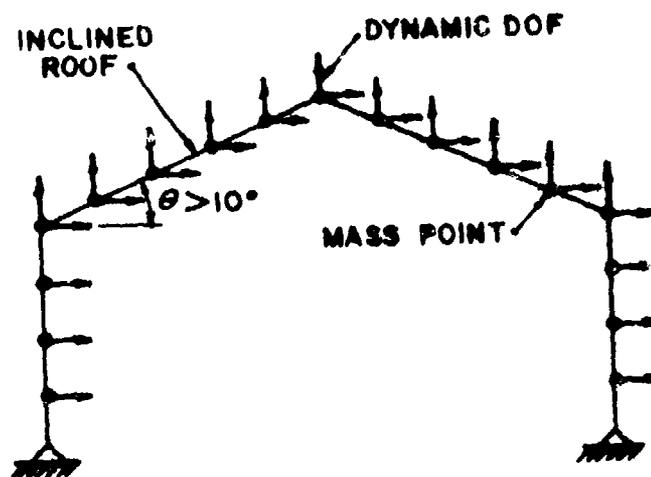


b) MEMBER WITH ABRUPT DISCONTINUITIES

Fig 15 Modeling of members with varying cross-sections



RECTANGULAR FRAME



GABLED FRAME

Fig 16 Dynamic degrees of freedom for typical frames

Framing plan	Reference figure	Mass assigned to horizontal dynamic DOFS on column		Mass assigned to vertical dynamic DOFS on column (M_{EV}^H)
		at intermediate mass points (M_{IH}^H)	at end points* (M_{EH}^H)	
1. Girts parallel to column	18a	$M_W / (N_{DOFH} + 1)$	$(M_W - N_{DOFH} M_{IH}) / 2$	$M_W / 2$
2. Girts normal to column a. $ng > 3$ $N_{DOFH} \neq ng$ b. $ng \leq 3$ $N_{DOFH} = ng$	18b	$M_W / (N_{DOFH} + 1)$ $[M_W / (N_{DOFH} + 1)] + m_g$	$(M_{IH})_W / 2$ $M_W / [2(N_{DOFH} + 1)]$	$M_W / 2$ $(M_W + \sum_1^{ng} m_g) / 2$

where M_W = for cases 1 and 2a: mass of all structure (siding, girts, column) within panel area $H_T \times H$

for case 2b only: mass of siding and column within panel area $H_T \times H$

M_{IH}^H = one-half the mass of siding within panel area ($M_1 + M_2$) $\times H$, plus mass of column

m_g = mass of one girt within panel

N_{DOFH} = number of horizontal dynamic degrees-of-freedom at intermediate mass points within panel

ng = number of girts within panel

* When intermediate mass points are not utilized on the member ($N_{DOFH} = 0$), the expressions provided above for M_{EH}^H are not applicable. For this case, $(M_{EH}^H)_W = (M_{EV}^H)_W$.

Fig 17 Expressions for computing concentrated masses for exterior wall panels

Framing plan	Reference figure	Mass assigned to horizontal dynamic DOFS on girder (M_{EH}^i)/ R	Mass assigned to vertical dynamic DOFS on girder	
			at intermediate mass points (M_{IV}^i)/ R	at end points* (M_{EV}^i)/ R
1. Purlins/joists parallel to girder	20a	$M_R/2$	$M_R^i/(N_{DOFV} + 1)$	$(M_R - N_{DOFV}M_{IV}^i)/2$
2. Purlins/joists normal to girder	20b	$M_R/2$	$M_R^i/(N_{DOFV} + 1)$	$(M_{IV}^i)/R/2$
a. $n_p > 3$ $N_{DOFV} \neq n_p$		$(M_R + \sum_1^{n_p} m_p)/2$	$[M_R^i/(N_{DOFV} + 1)] + m_p$	$M_R/[2(N_{DOFV} + 1)]$
b. $n_p \leq 3$ $N_{DOFV} = n_p$				

where M_R = for cases 1 and 2a: mass of all structure (decking, purlin, girder) within panel area $W_T \times L$

= for case 2b only: mass of decking and girder within panel area $W_T \times L$

M_R^i = one-half the mass of decking within panel area ($M_i^i + W_i^i$) $\times L$ plus mass of girder

m_p = mass of one purlin within panel

N_{DOFV} = number of vertical dynamic degrees-of-freedom at intermediate mass points within panel

n_p = number of purlins within panel

* When intermediate mass points are not utilized on the member ($N_{DOFV} = 0$), the expressions provided above for M_{EV}^i are not applicable. For this case, use $(M_{EV}^i)_R = (M_{EH}^i)_R$.

Fig 19 Expressions for computing concentrated masses for roof and floor panels

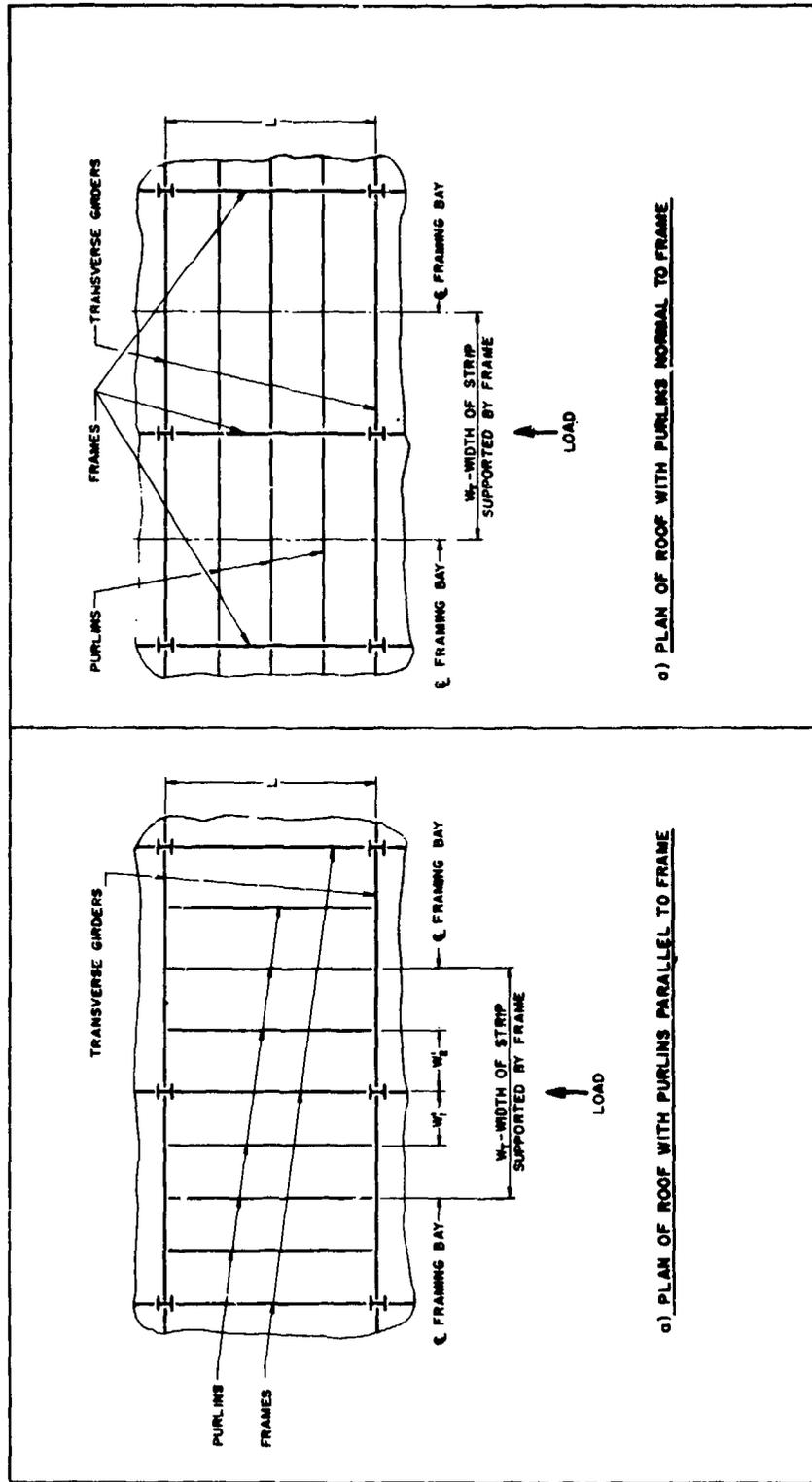


Fig 20 Typical framing plans of roof and floor panels

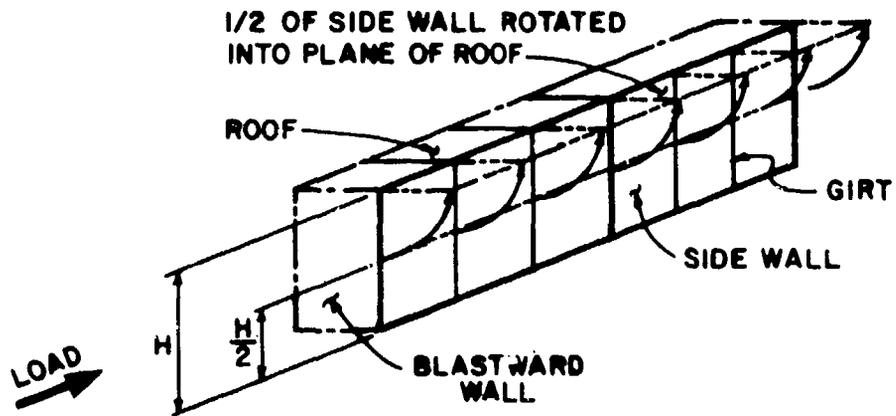
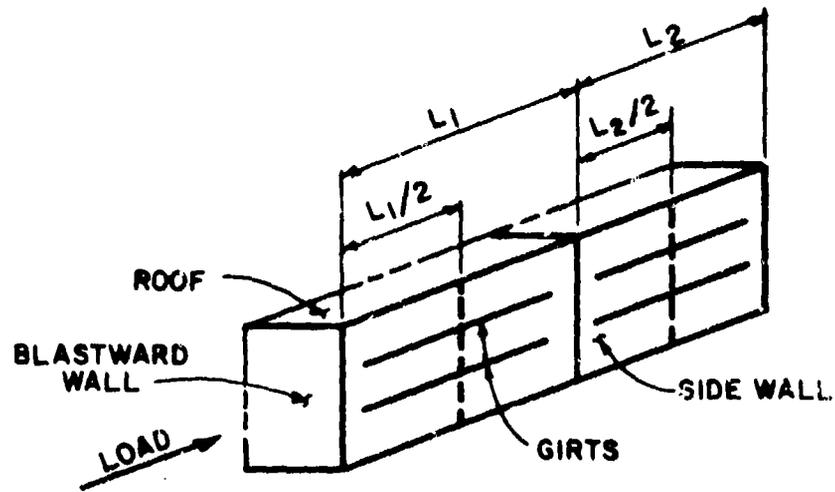
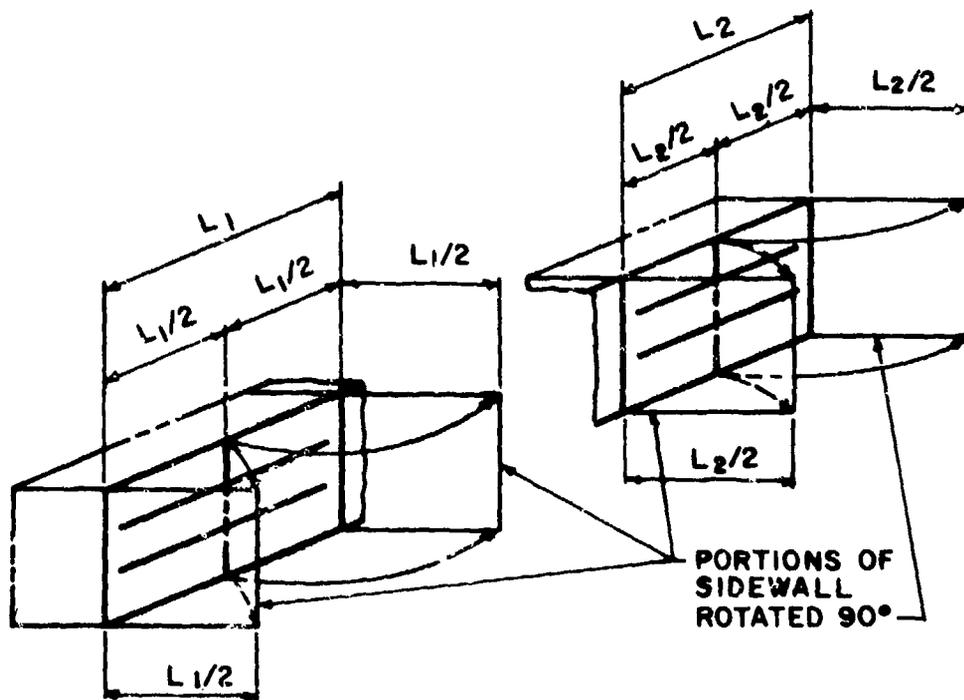


Fig 21 Portion of sidewall mass lumped with end frame:
sidewall with vertical girts

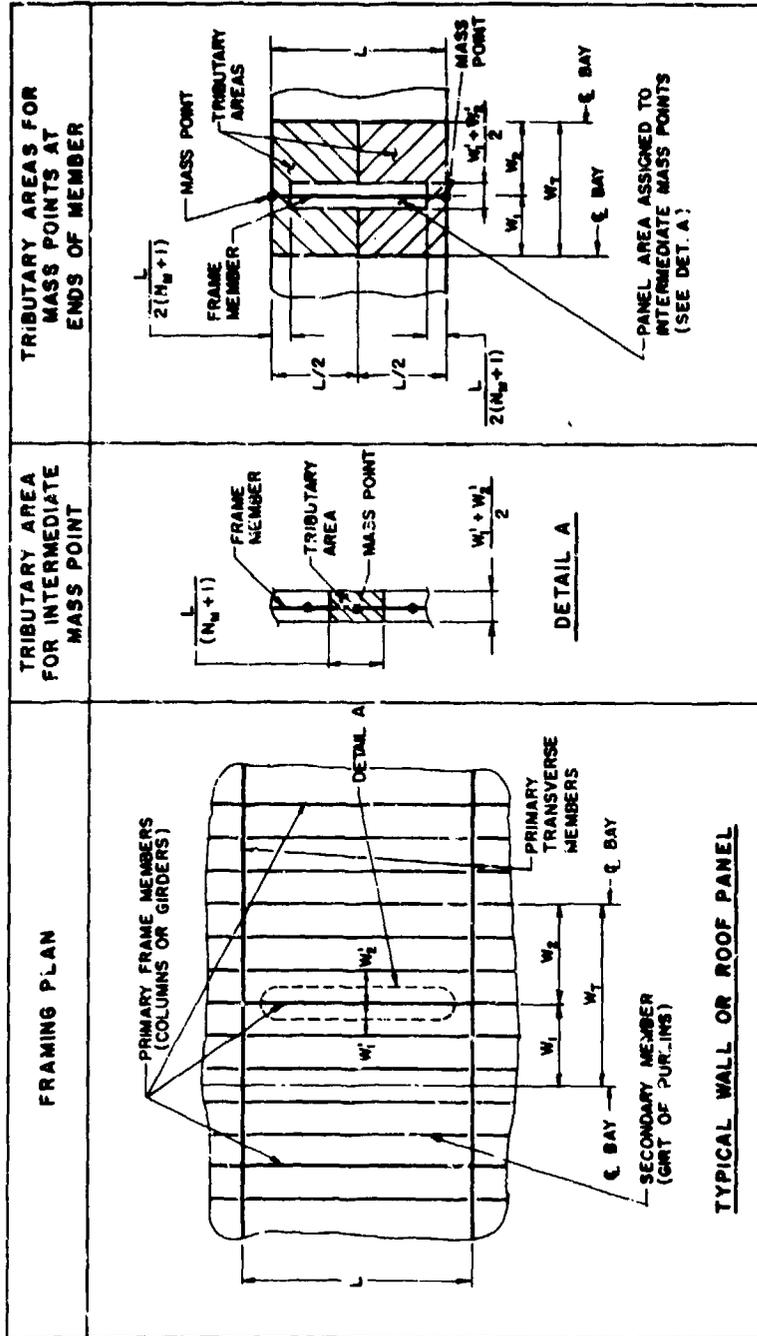


a) FRAMING OF SIDEWALL



b) SECTIONS OF SIDEWALL ROTATED 90°

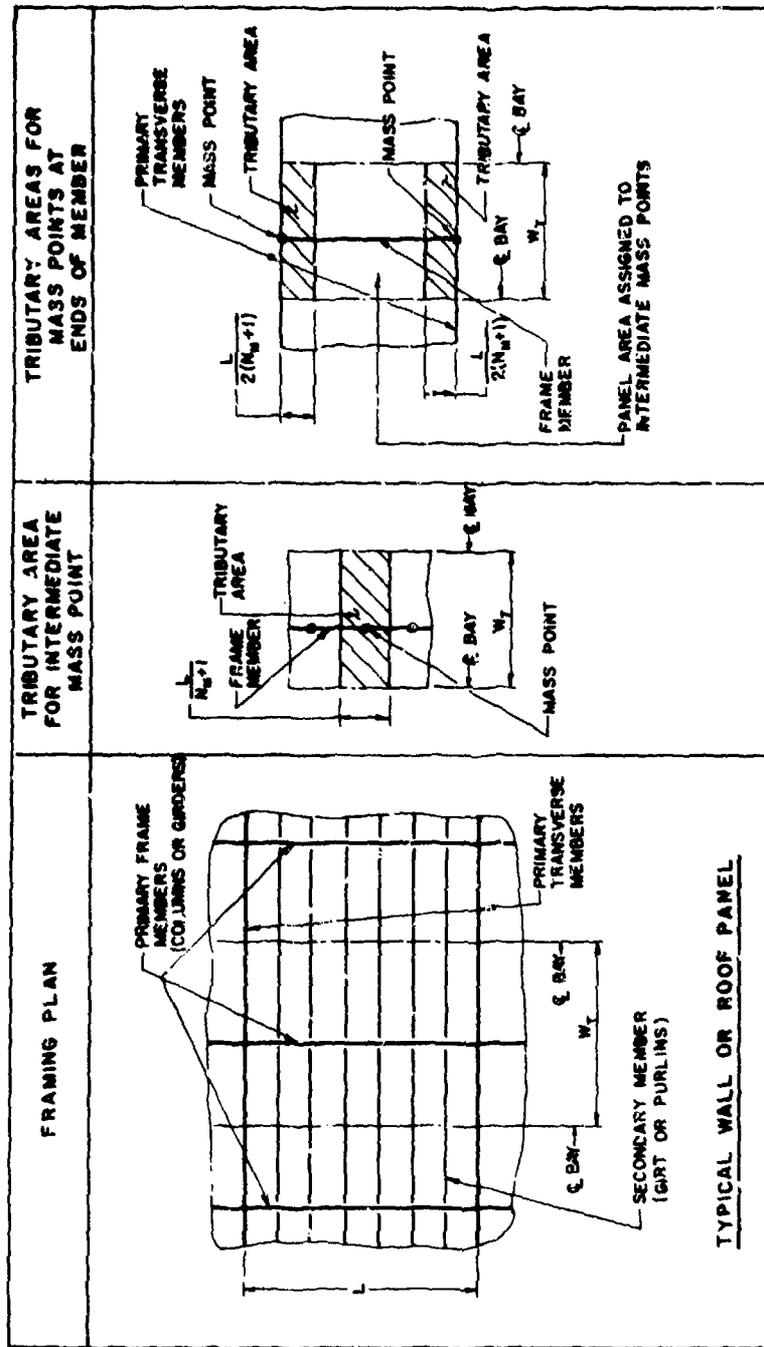
Fig 22 Distribution of sidewall mass on end frame: sidewall with horizontal girts



WHERE: W_T = TRIBUTARY WIDTH OF BUILDING SUPPORTED BY FRAME
 $N_B = N_{B,WH}$ FOR A WALL PANEL (REF FIG. 17)
 $= N_{B,ROOF}$ FOR A ROOF PANEL (REF FIG. 19)

NOTE: WHEN INTERMEDIATE MASS POINTS ARE NOT UTILIZED (I.E., $N_{B,WH} = 0$ OR $N_{B,ROOF} = 0$), ASSIGN 1/2 OF THE TOTAL AREA ($W_T \cdot L$) TO THE MASS POINT AT EACH END OF THE MEMBER

Fig 23 Dimensions of tributary areas for panel with secondary members parallel to frame



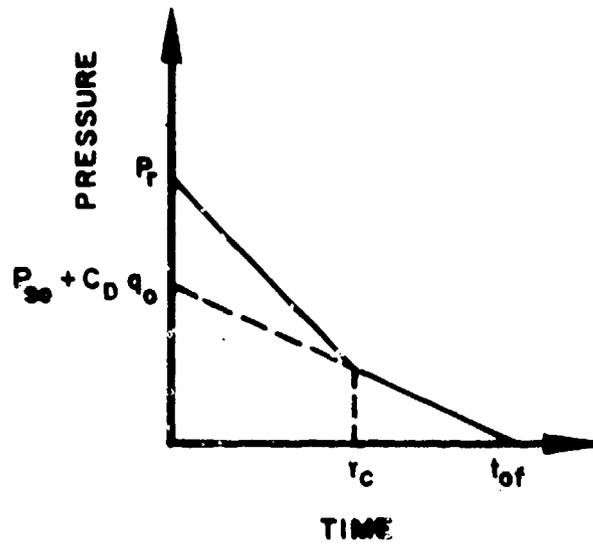
WHERE: w_T = TRIBUTARY WIDTH OF BUILDING SUPPORTED BY FRAME

N_B = N_{col} FOR A WALL PANEL (REF FIG.17)

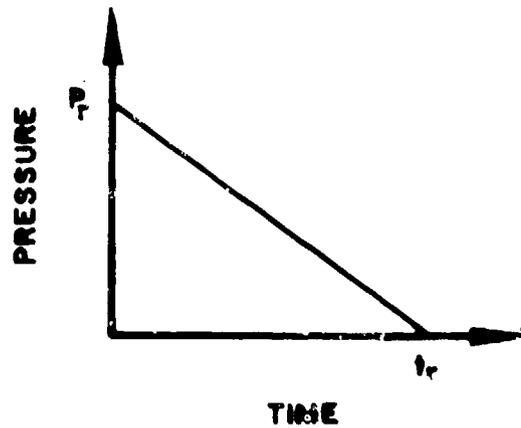
N_B = N_{purl} FOR A ROOF PANEL (REF FIG.19)

NOTE: WHEN INTERMEDIATE MASS POINTS ARE NOT UTILIZED (i.e., $N_{col} = 0$ OR $N_{purl} = 0$), ASSIGN 1/2 OF THE TOTAL PANEL AREA ($w_T \cdot L$) TO THE MASS POINT AT EACH END OF THE MEMBER

Fig 24 Dimensions of tributary areas for panel with secondary members perpendicular to frame



a) LONG DURATION BLAST LOAD



b) SHORT DURATION BLAST LOAD

Fig 26 Blastward wall reflected pressure waveforms

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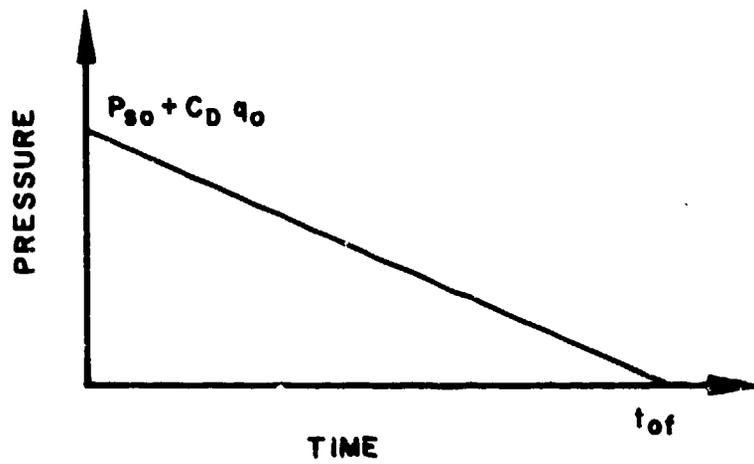


Fig 27 Pressure history for roof, side wall and leeward wall

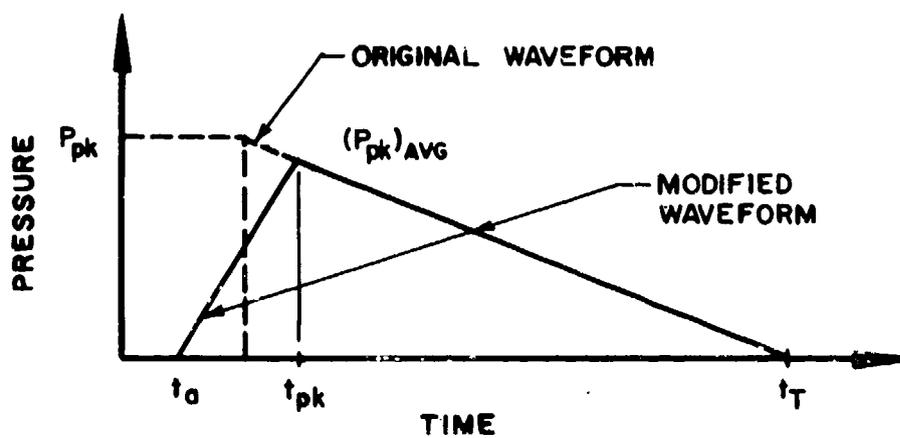


Fig 28 Phased and modified pressure waveform with linear decay

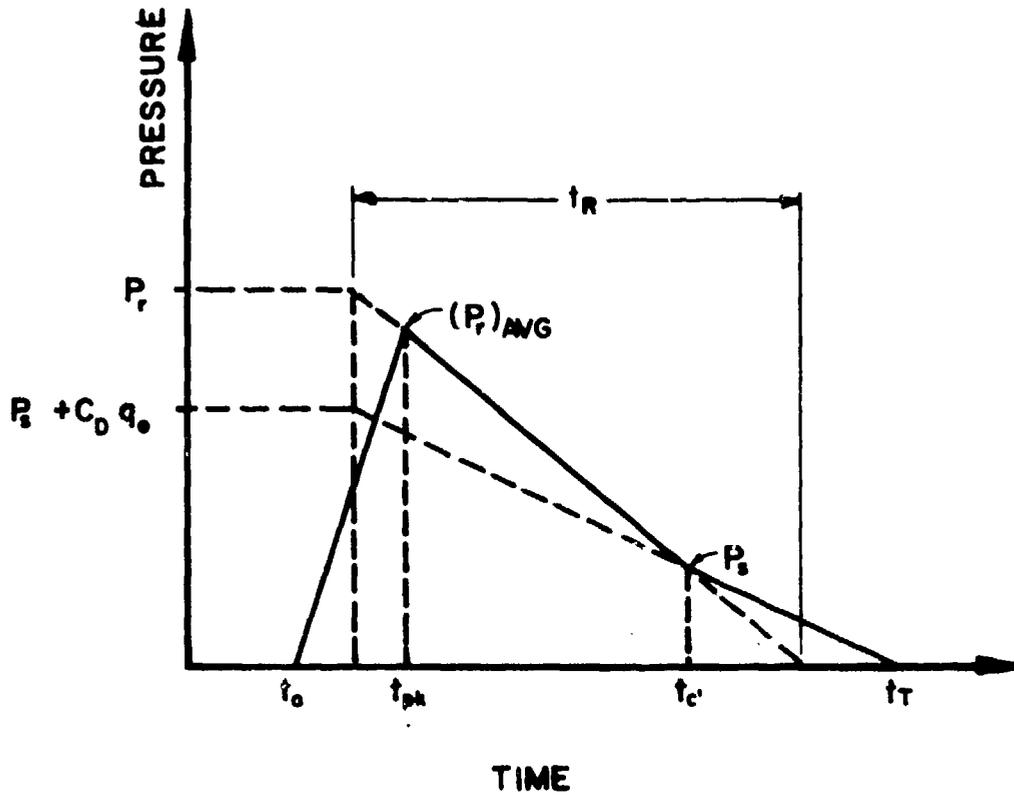


Fig 29 Phased and modified reflected pressure waveform with bilinear decay

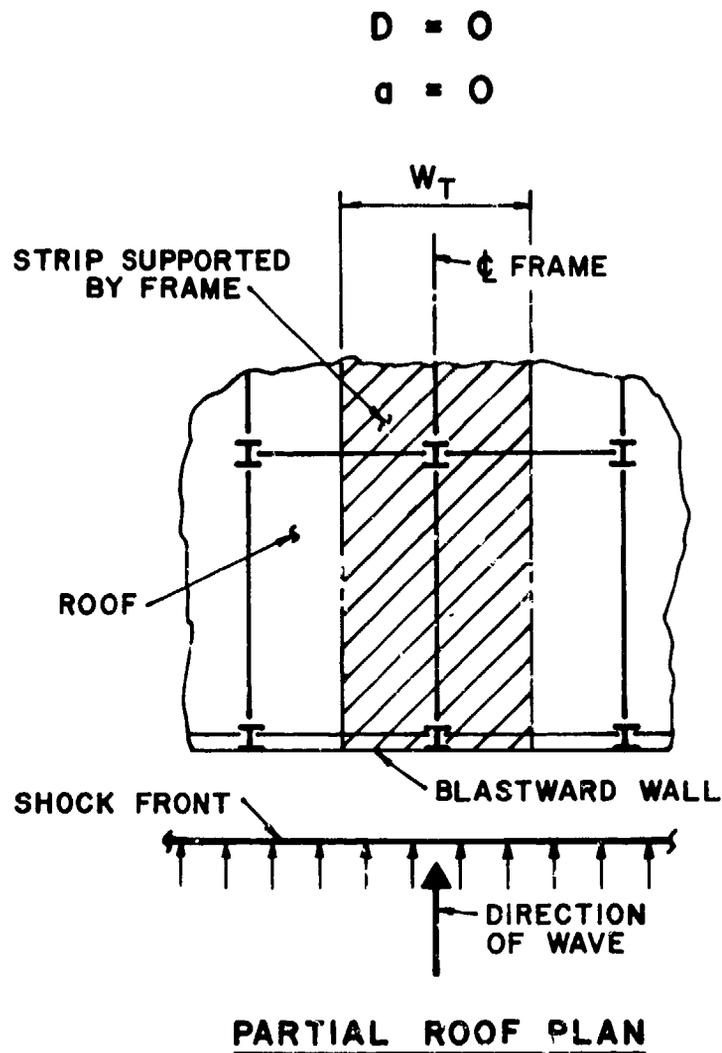
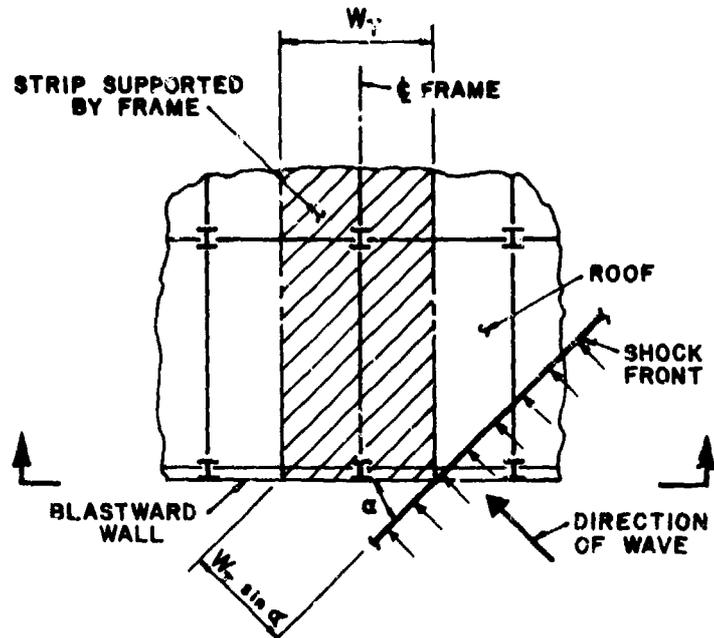
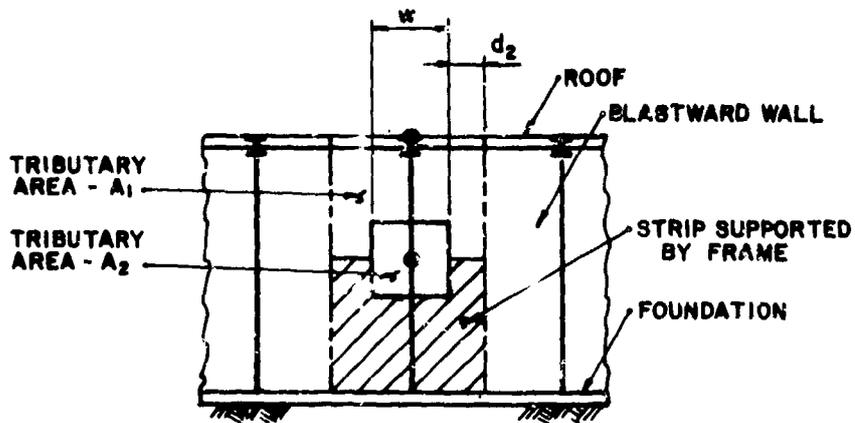


Fig 30 Example of values for a and D for tributary areas on blastward wall - normal shock wave

$$\begin{aligned} \text{For } A_1 : D &= 0 & \alpha &= W_T \sin \alpha \\ \text{For } A_2 : D &= d_2 \sin \alpha & \alpha &= w \sin \alpha \end{aligned}$$



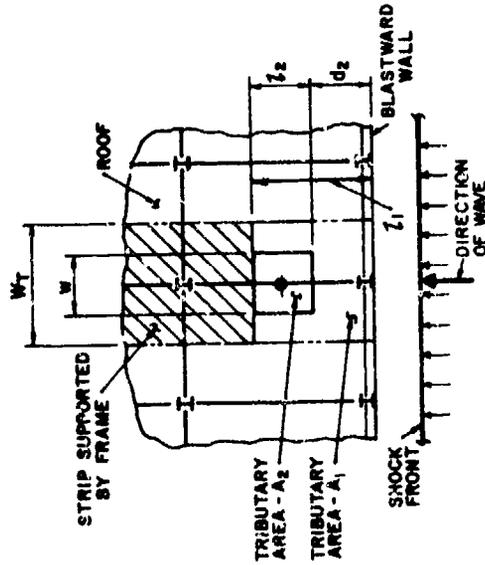
PARTIAL ROOF PLAN



ELEVATION

Fig 31 Examples of values of α and D for tributary areas on blastward wall - quartering shock wave

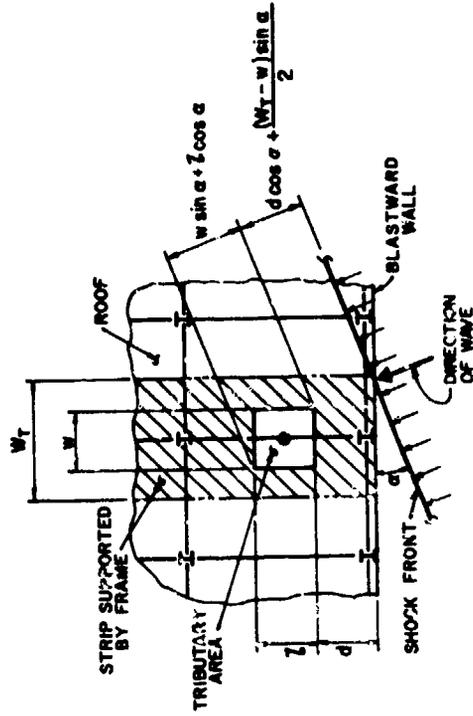
For A_1 : $D=0$; $a=Z_1$
 For A_2 : $D=d_2$; $a=Z_2$



PARTIAL ROOF PLAN

g) NORMAL SHOCK WAVE

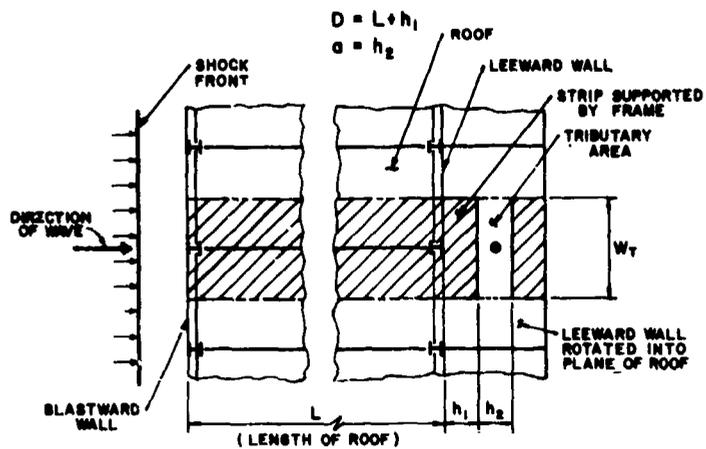
$D = d \cos \alpha + 1/2 (W_T - w) \sin \alpha$
 $a = w \sin \alpha + Z \cos \alpha$



PARTIAL ROOF PLAN

b) QUARTERING SHOCK WAVE

Fig 32 Examples of values of a and D for tributary areas on roof

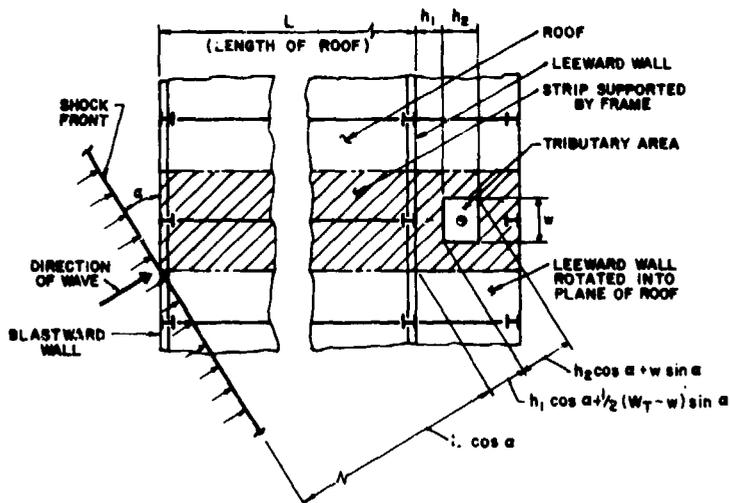


PARTIAL ROOF PLAN

a) NORMAL SHOCK WAVE

$$D = (L + h_1) \cos \alpha + 1/2(W_T - w) \sin \alpha$$

$$\alpha = h_2 \cos \alpha + w \sin \alpha$$



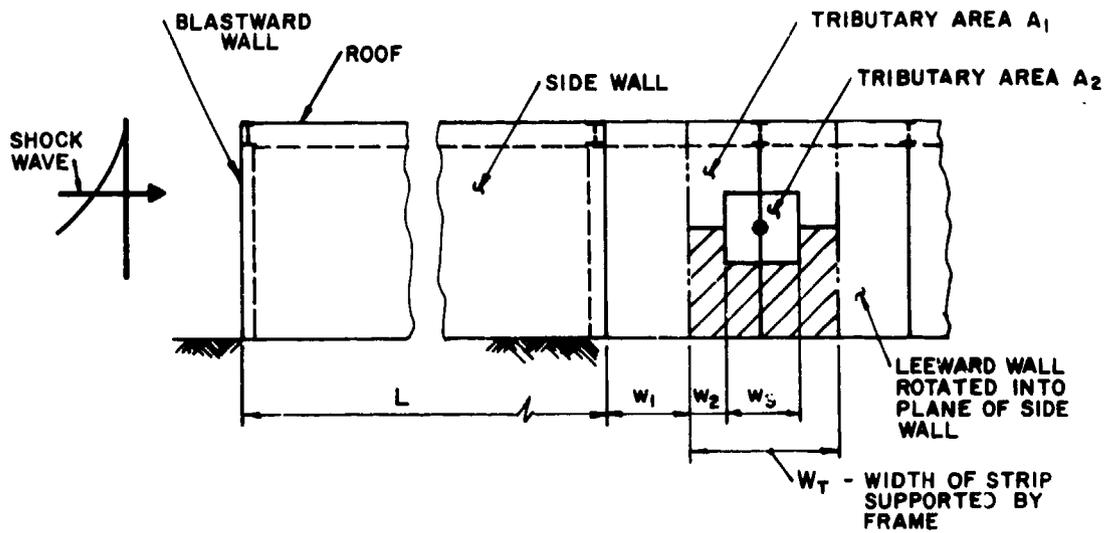
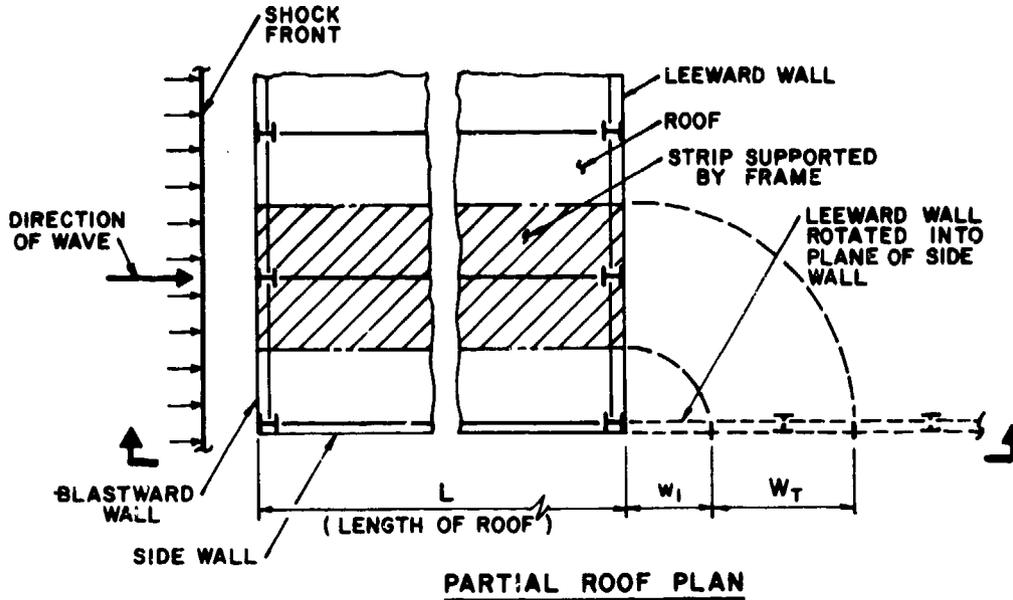
PARTIAL ROOF PLAN

b) QUARTERING SHOCK WAVE

Fig 33 Examples of values of α and D for tributary areas on leeward wall with wall taken as extension of roof

For A_1 : $D = L + w_1$; $a = W_T$

For A_2 : $D = L + w_1 + w_2$; $a = w_3$



ELEVATION

Fig 34 Examples of values of a and D for tributary areas on leeward wall with wall taken as extension of adjacent wall - normal shock wave

FOR A_1 : $D = L \cos \alpha + w_1(1.0 - \sin \alpha)$; $\alpha = W_T$
 FOR A_2 : $D = L \cos \alpha + w_1(1.0 - \sin \alpha) + w_2$; $\alpha = W_3$

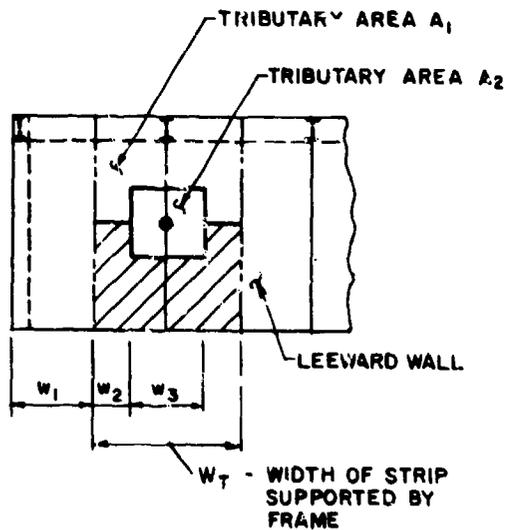
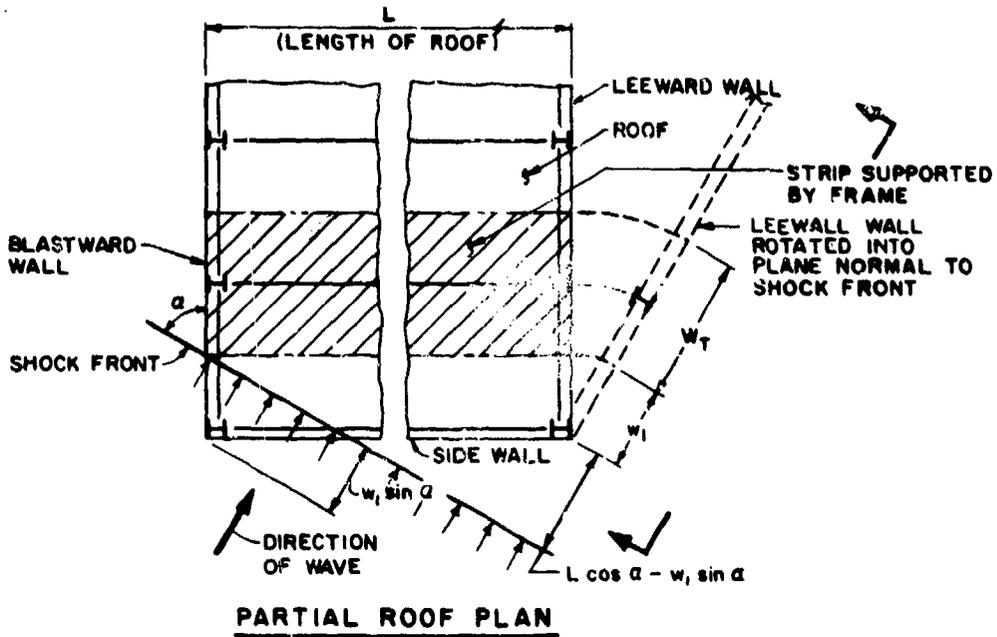
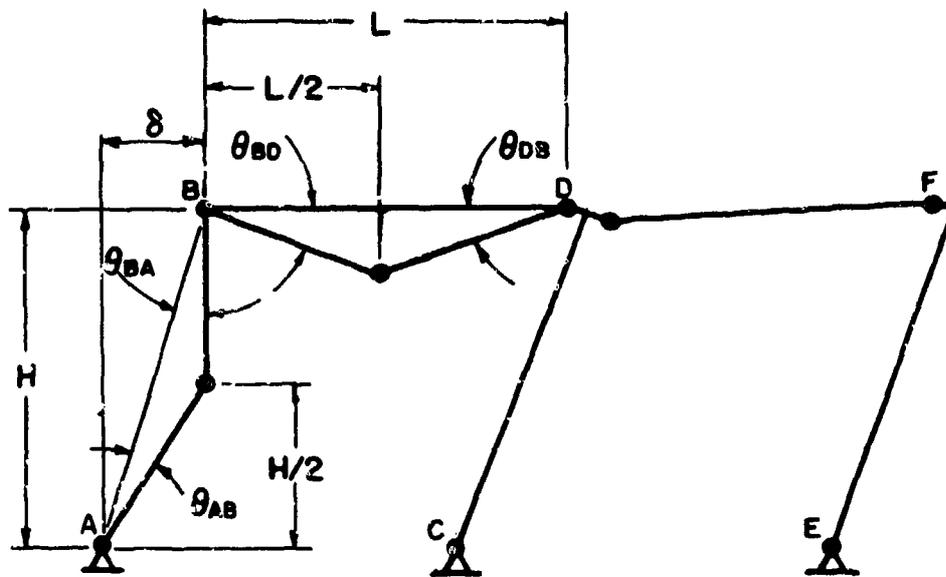


Fig 35 Examples of values of α and D for tributary areas on leeward wall with wall taken as extension of adjacent wall - quartering shock wave



FRAME STRUCTURE

Fig 36 Sidesway deflection and member end rotations for frame structures

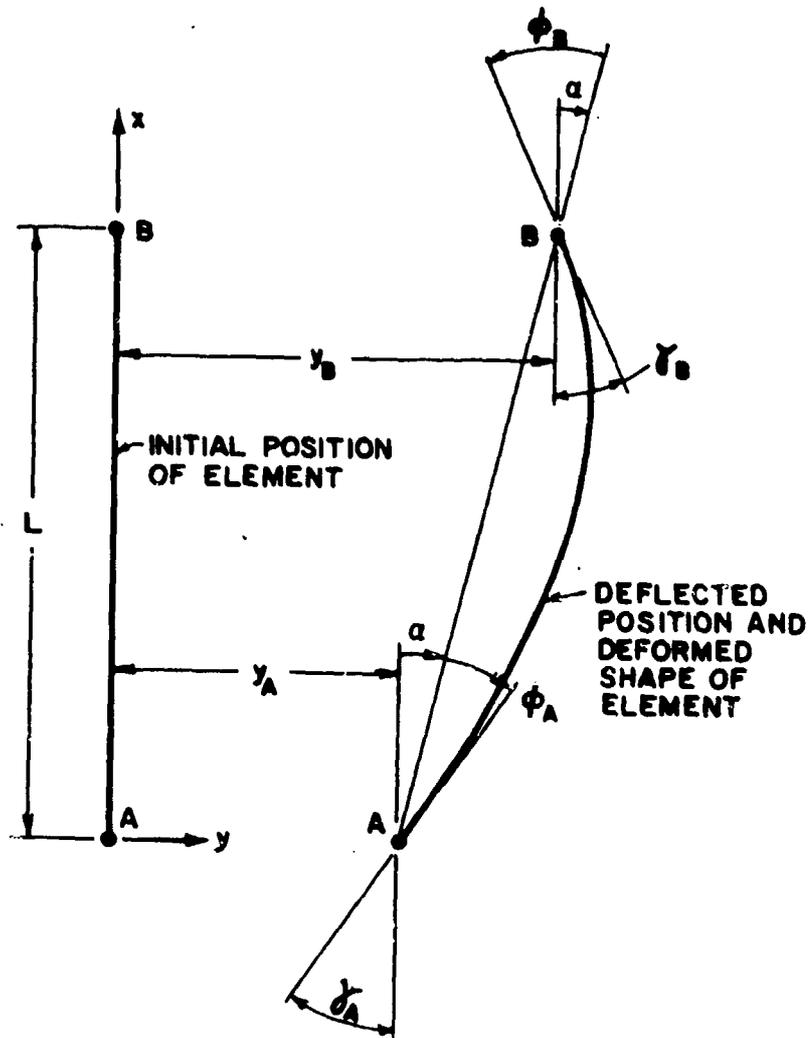


Fig 37 Components of element end rotation in the elastic response range

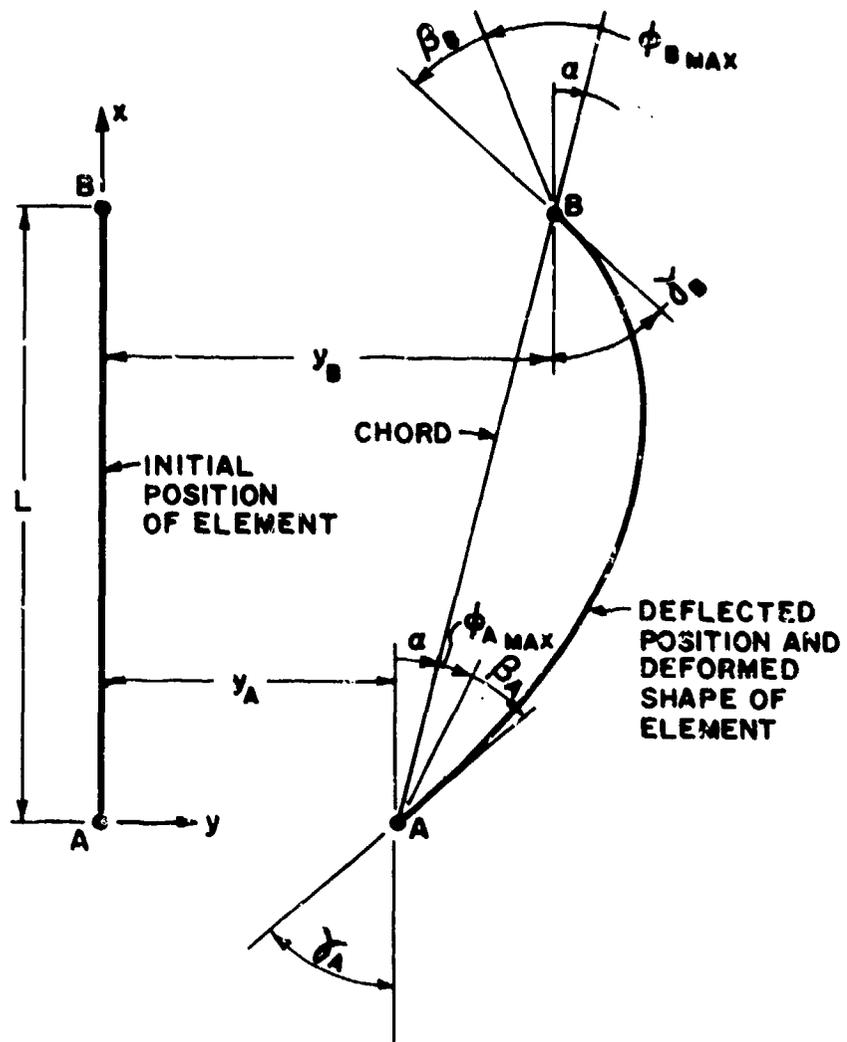
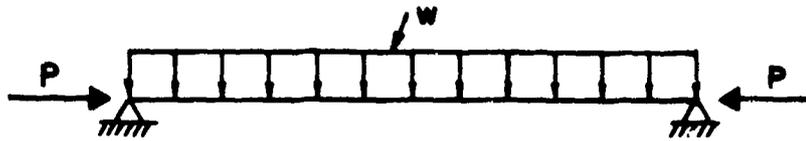
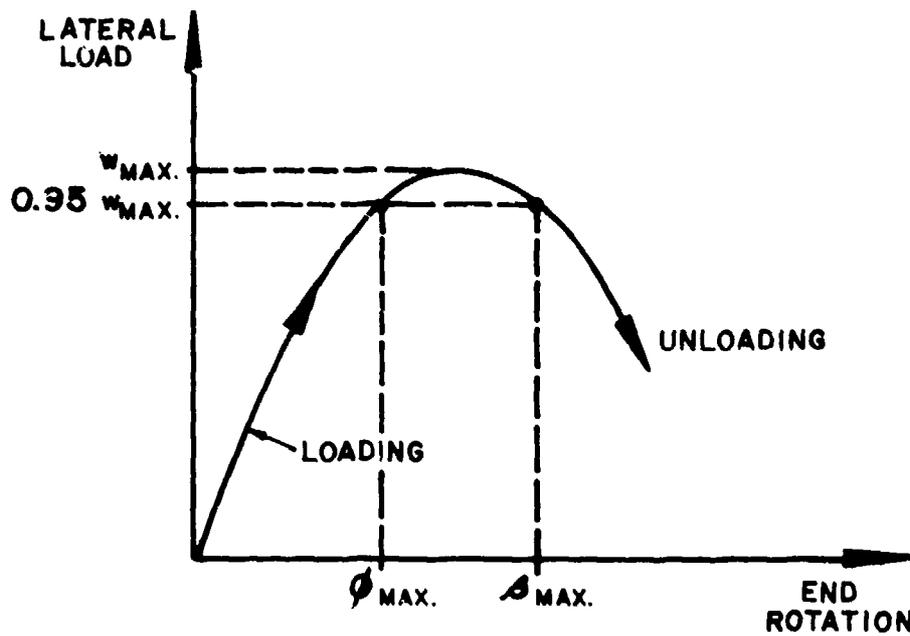


Fig 38 Components of element end rotation in the inelastic response range



a) LOADING



b) RESPONSE

Fig 39 Load deformation response for laterally loaded beam column

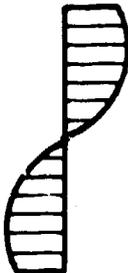
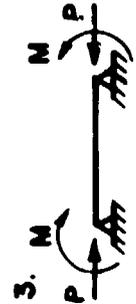
LOADING	MOMENT DIAGRAM	APPLICABLE FIGURE
1. 		41
2. 		41
3. 		42

Fig 40 Index of figures for rotation capacity curves of beam columns subjected to end moments

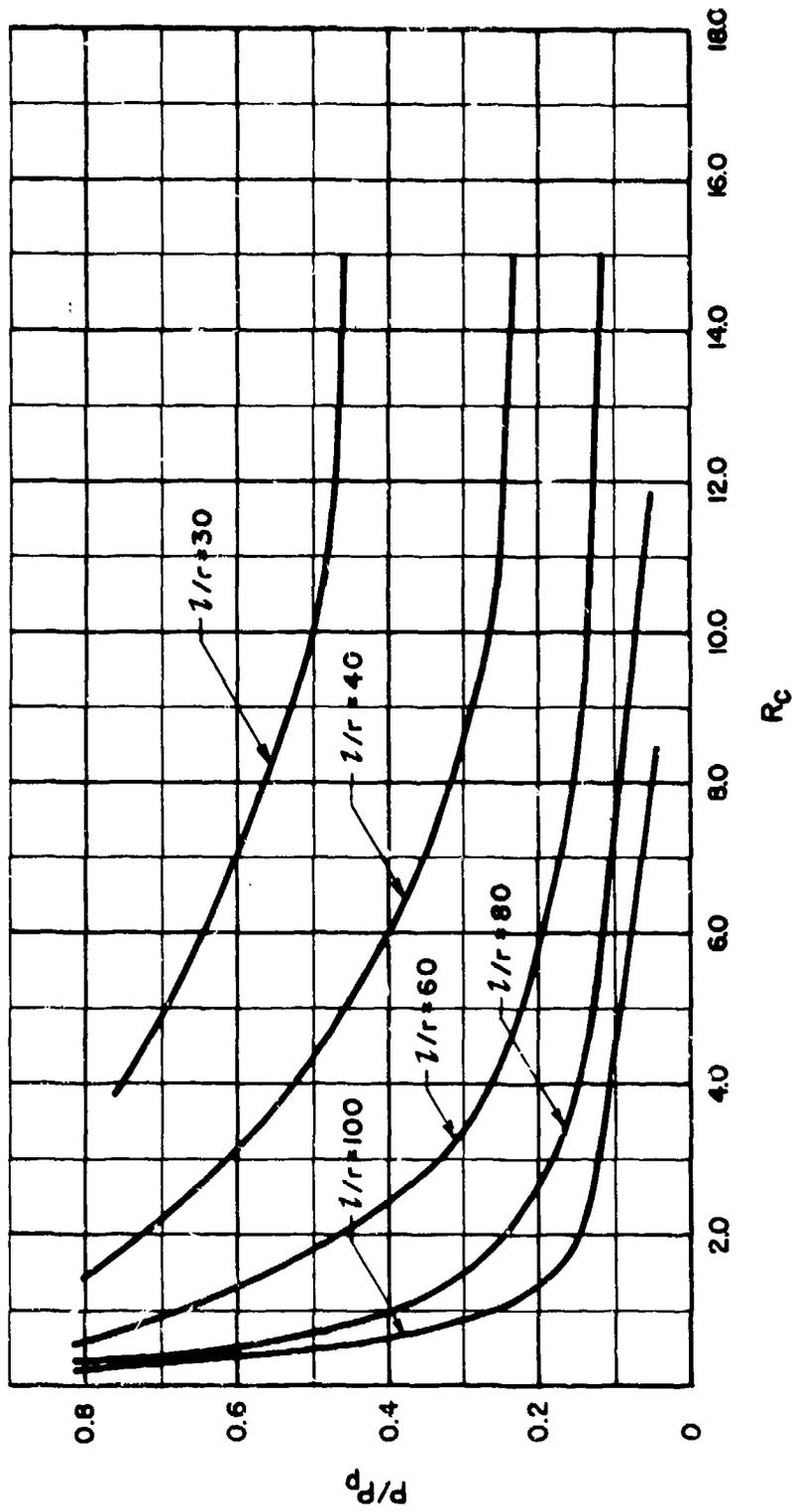


Fig 41 Rotation capacity curves for beam column bent to single curvature by a moment applied at one end or bent to reverse curvature

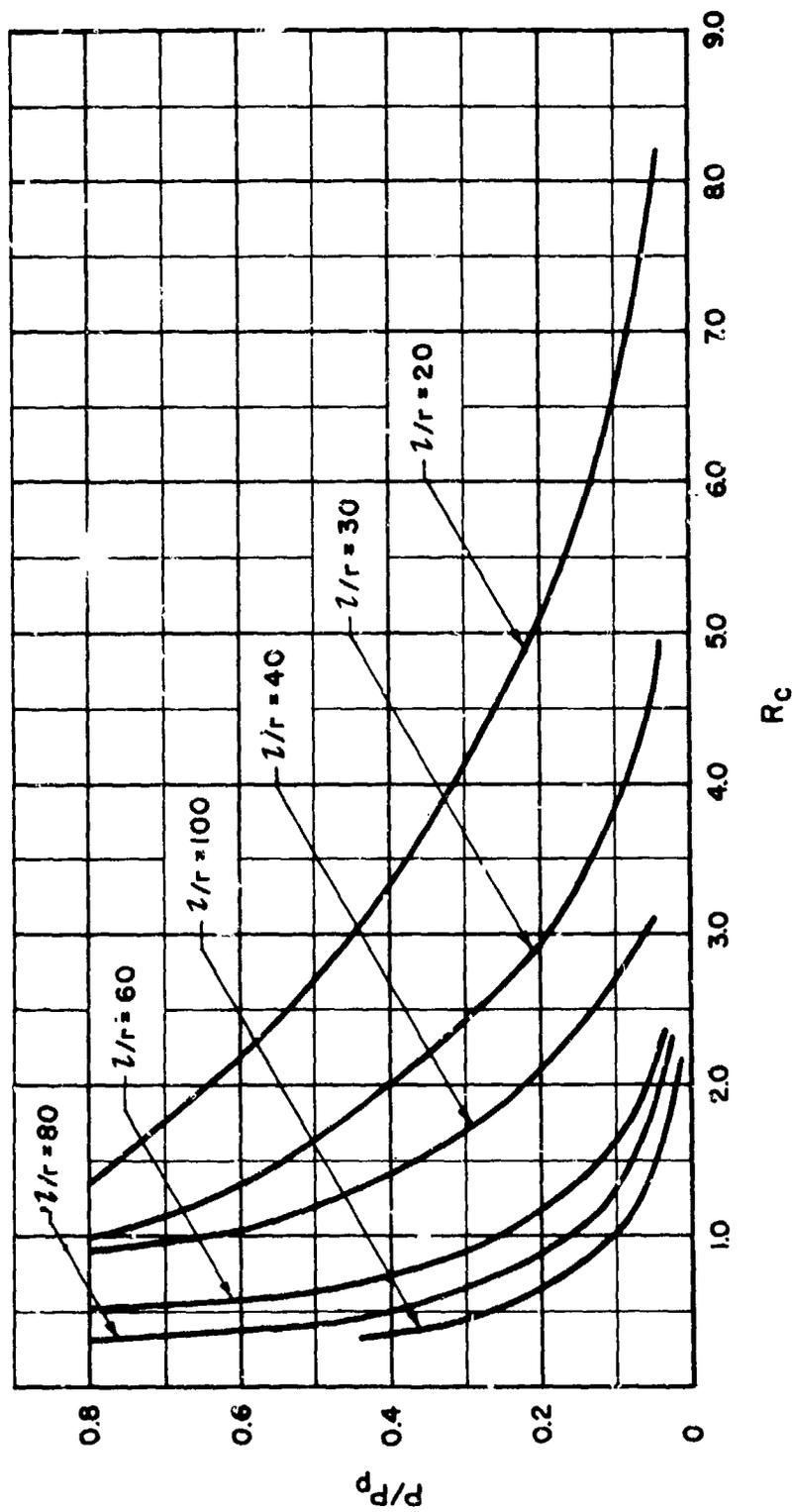


Fig 42 Rotation capacity curves for beam column bent to single curvature by equal and opposite end moments

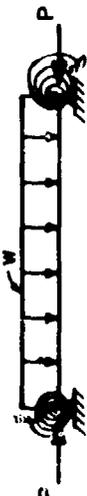
LOADING AND CONDITION OF RESTRAINT AT ENDS	MOMENT DIAGRAM	DISTRIBUTION FACTOR AT RESTRAINED END	APPLICABLE FIGURE
1. ELASTIC RESTRAINT BOTH ENDS 		0.0625 0.1250 0.2500 0.5000 0.7500	44 45 46 47 48
2. ELASTIC RESTRAINT ONE END 		0.0625 0.1250 0.2500 0.5000 0.7500	49 50 51 52 53

Fig 43 Index of figures for rotation capacity curves of beam columns subjected to lateral loads

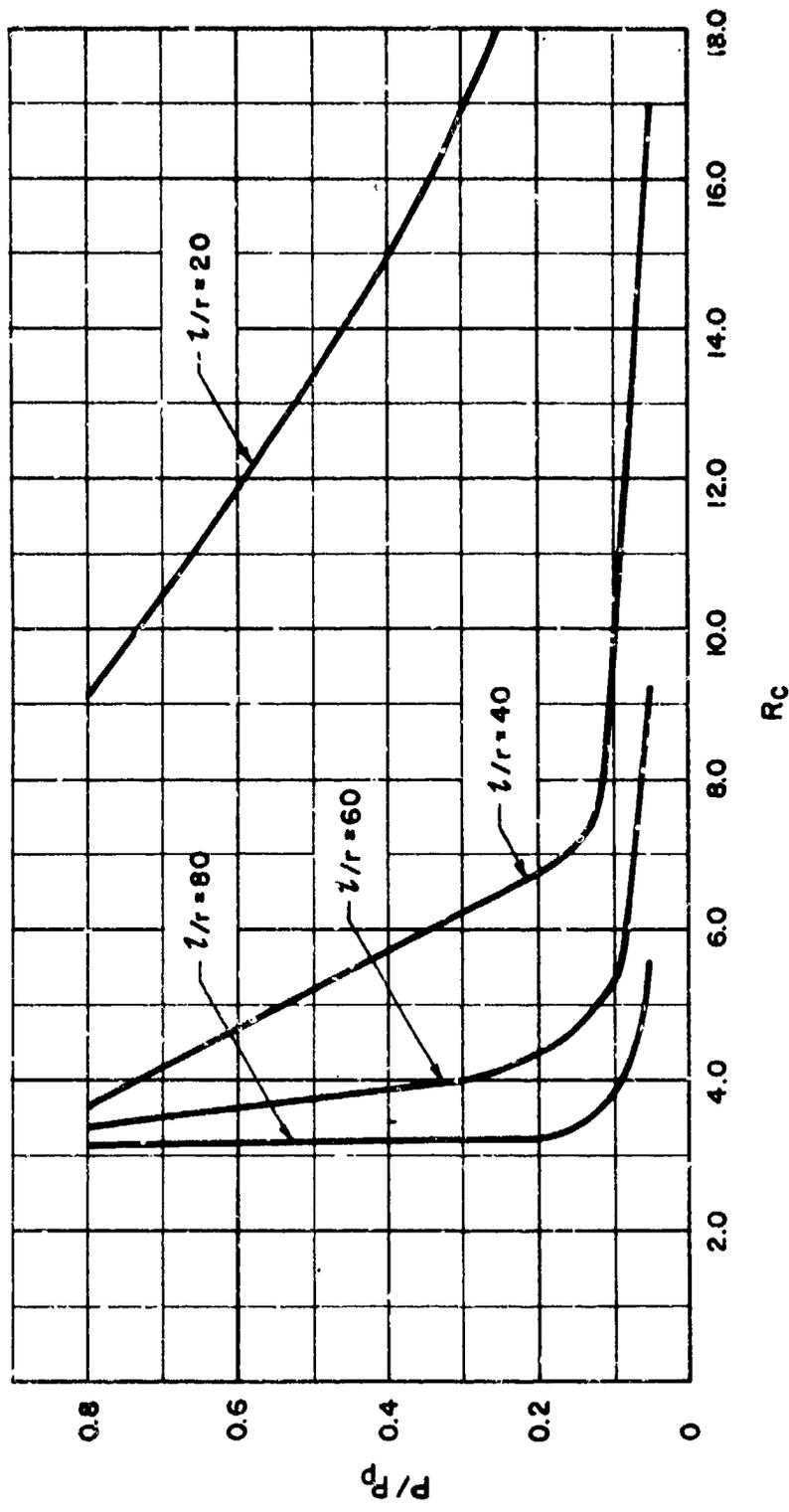


Fig 44 Rotation capacity curves for laterally loaded beam column - elastic restraint both ends: distribution factor = 0.0625

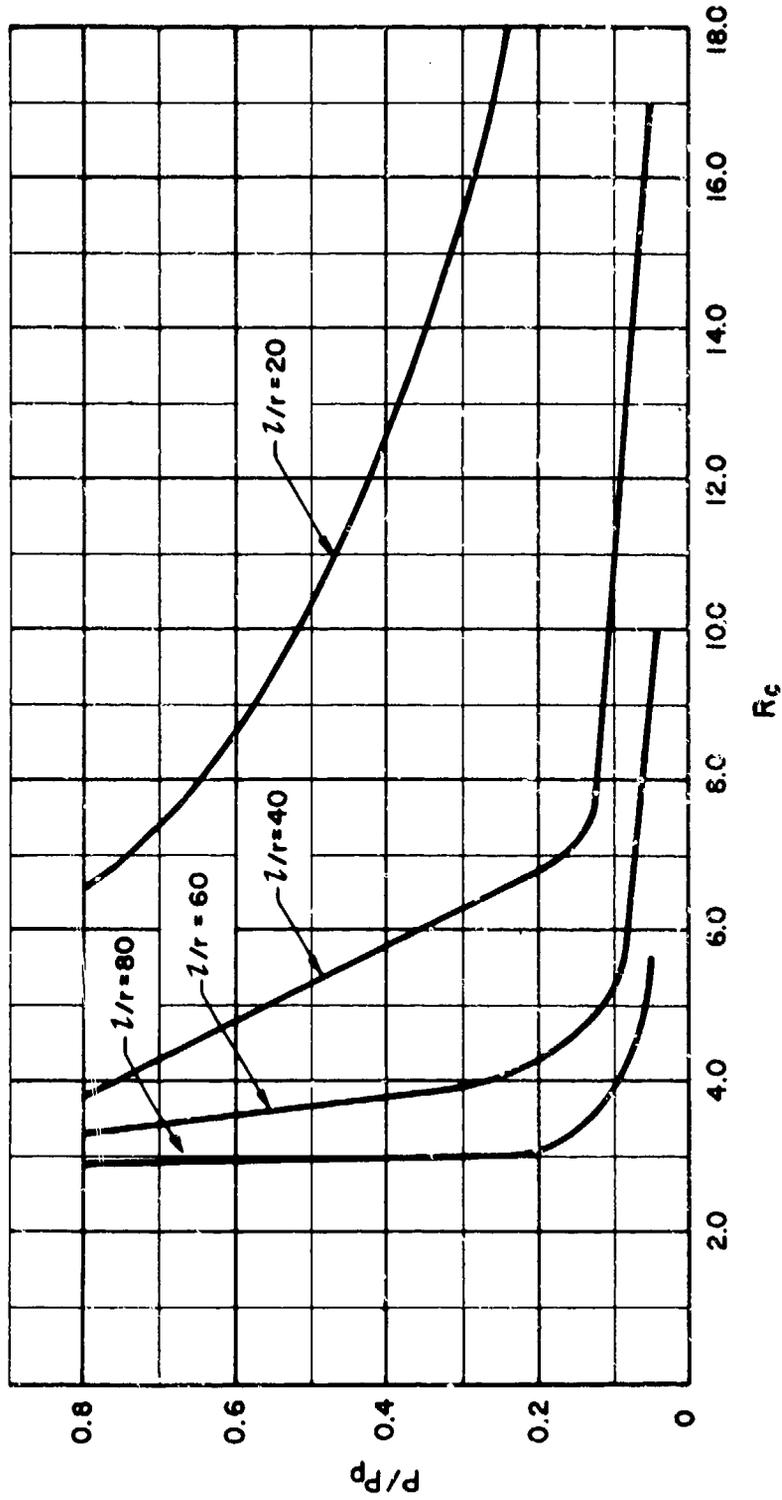


Fig 45 Rotation capacity curves for laterally loaded beam column - elastic
restraint both ends: distribution factor = 0.125

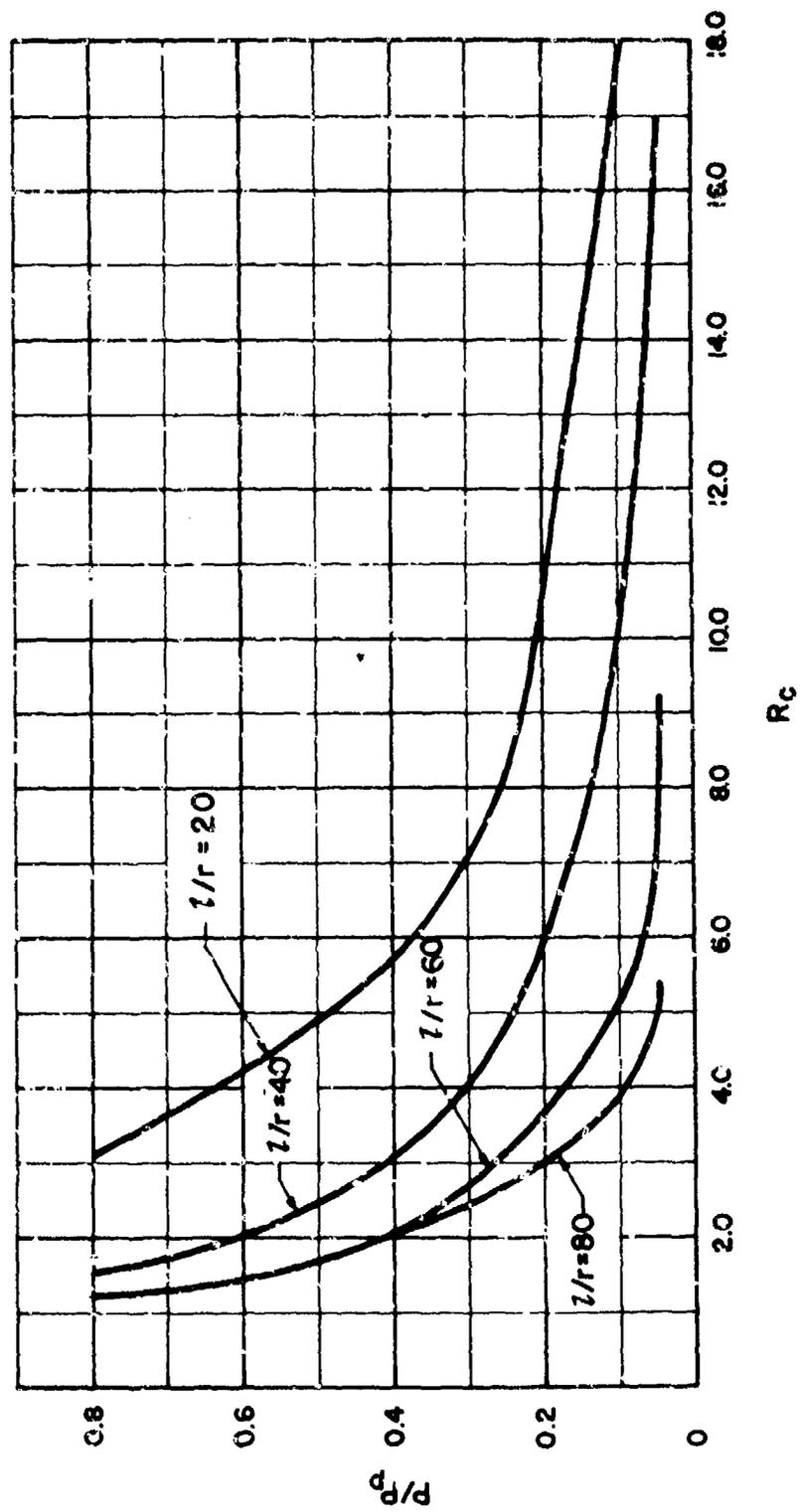


Fig 46 Rotation capacity curves for laterally loaded beam column - elastic restraint both ends: distribution factor = 0.250

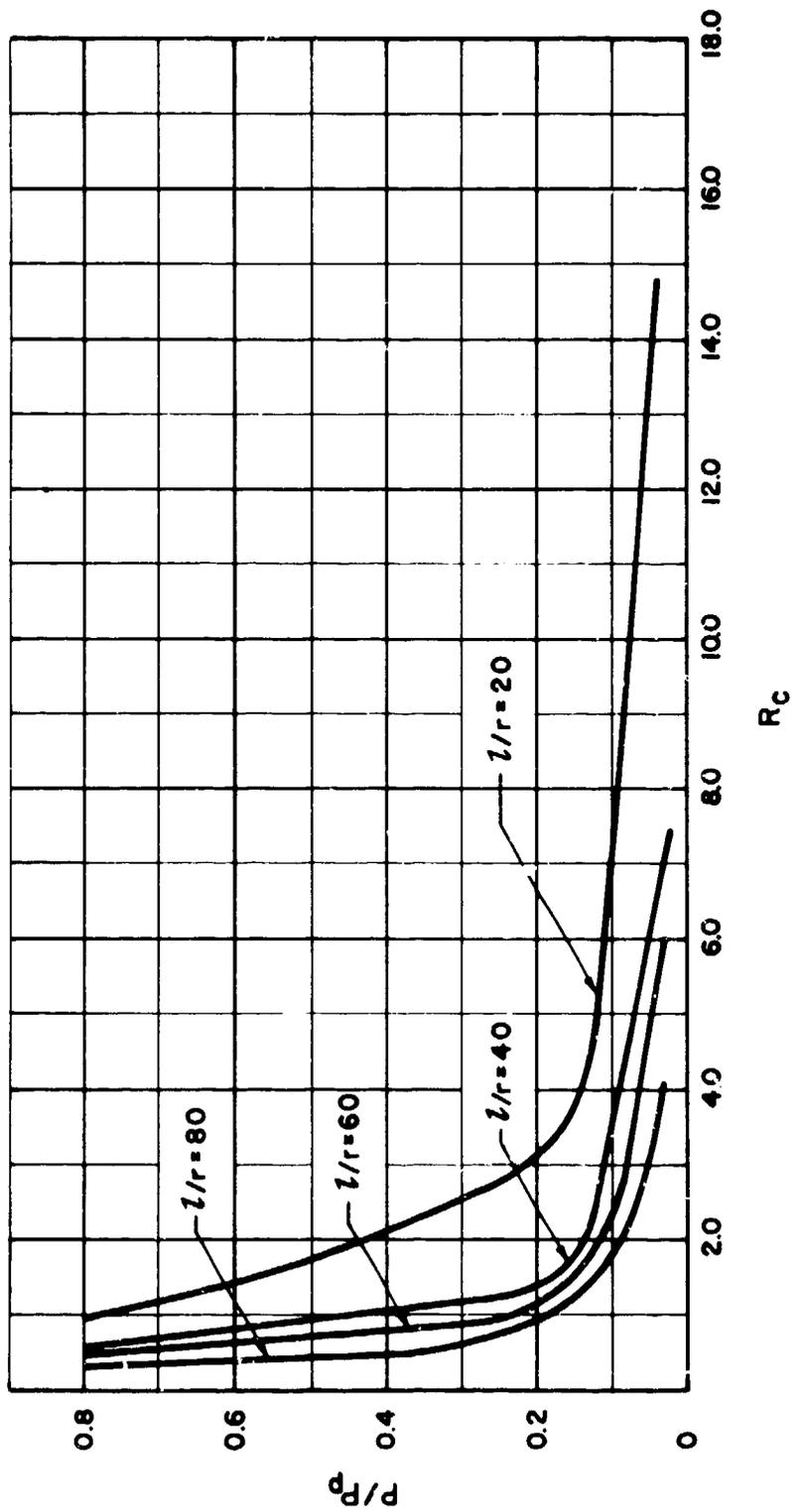


Fig 4/ Rotation capacity curves for laterally loaded beam column - elastic restraint both ends: distribution factor = 0.500

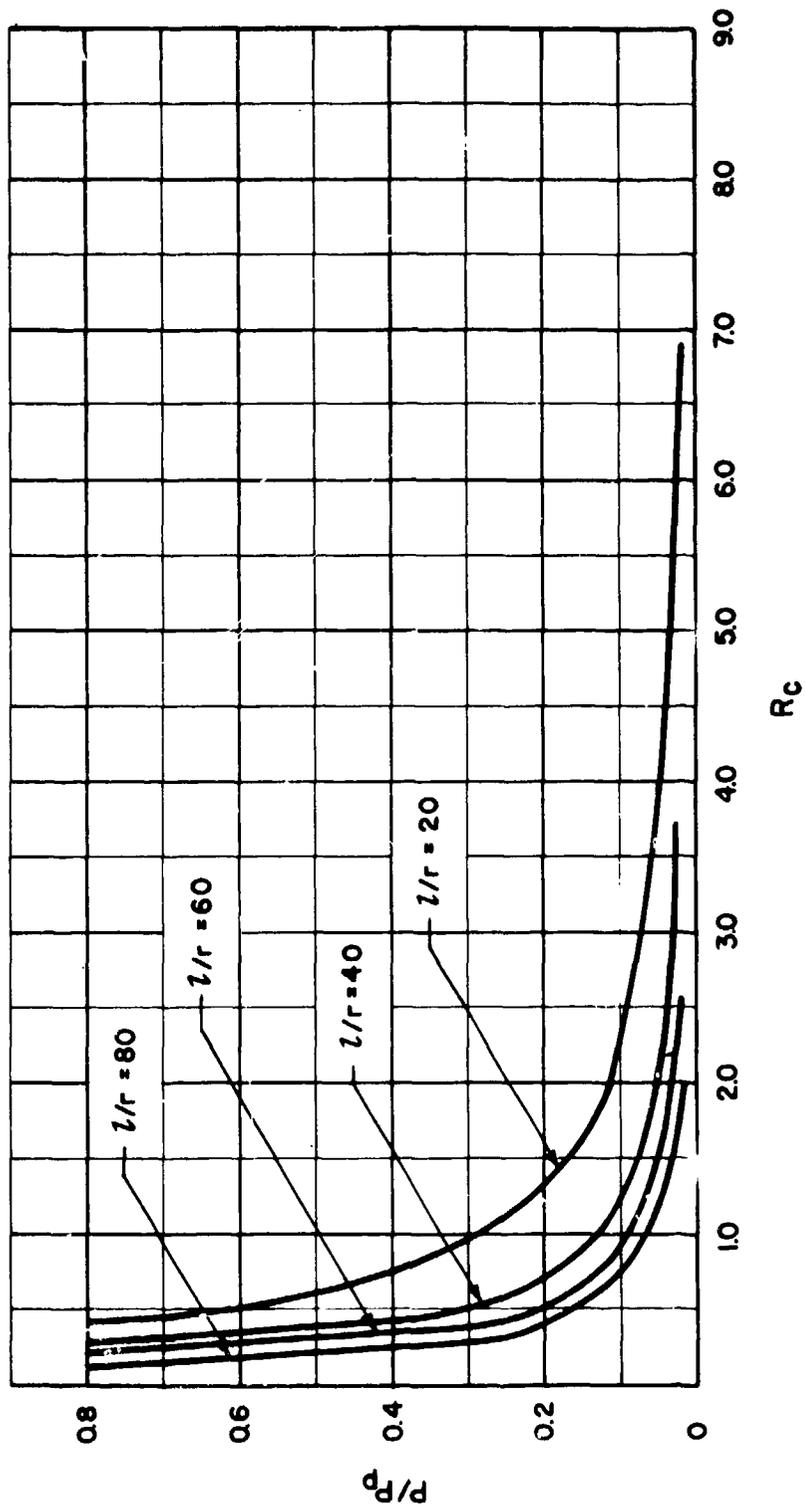


Fig 48 Rotation capacity curves for laterally loaded beam column - elastic restraint both ends: distribution factor = 0.750

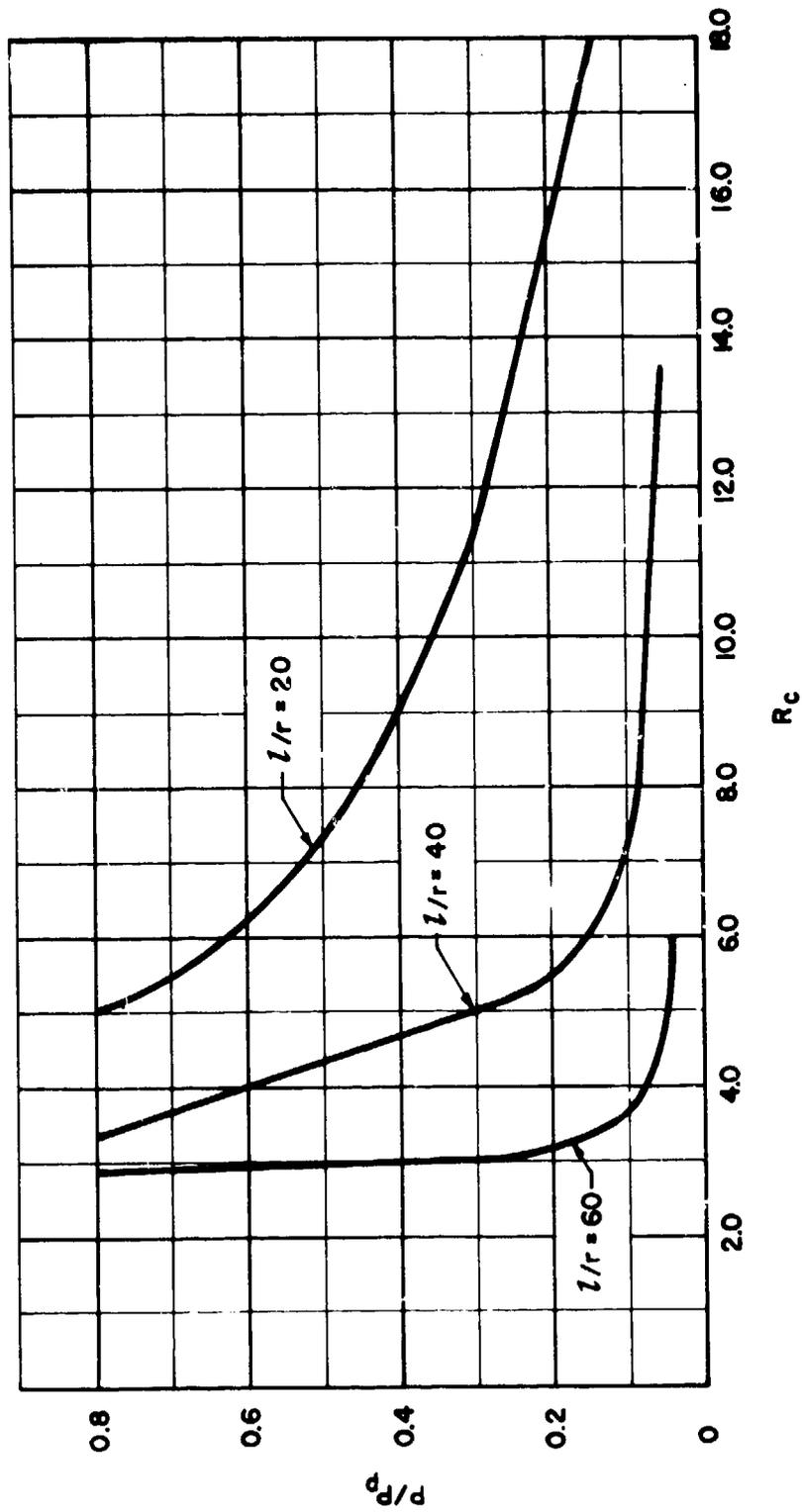


Fig 49 Rotation capacity curves for laterally loaded beam column - elastic restraint one end, other end pinned: distribution factor = 0.0625

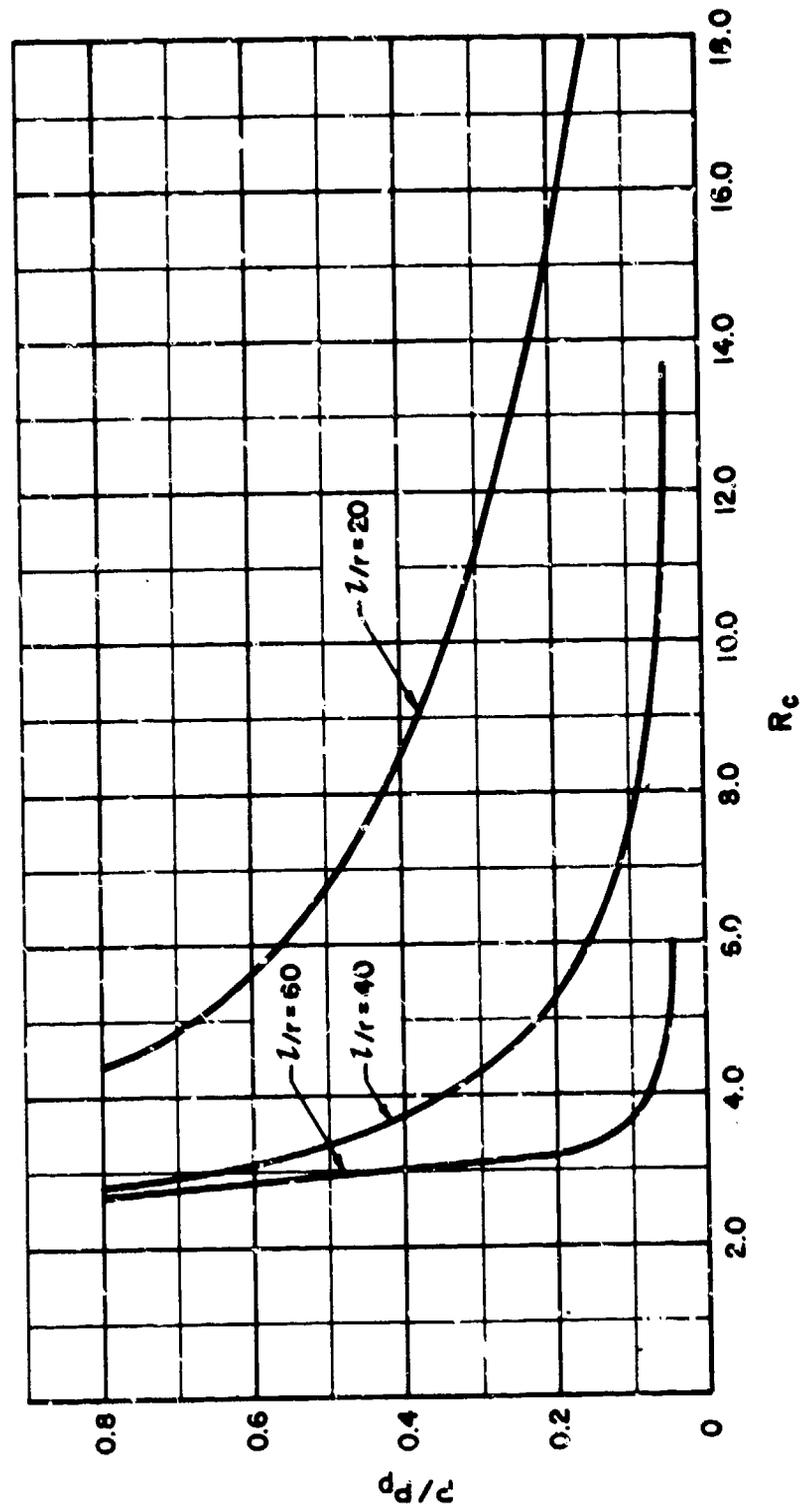


Fig 50 Rotation capacity curves for laterally loaded beam column - elastic restraint one end, other end pinned: distribution factor = 0.125

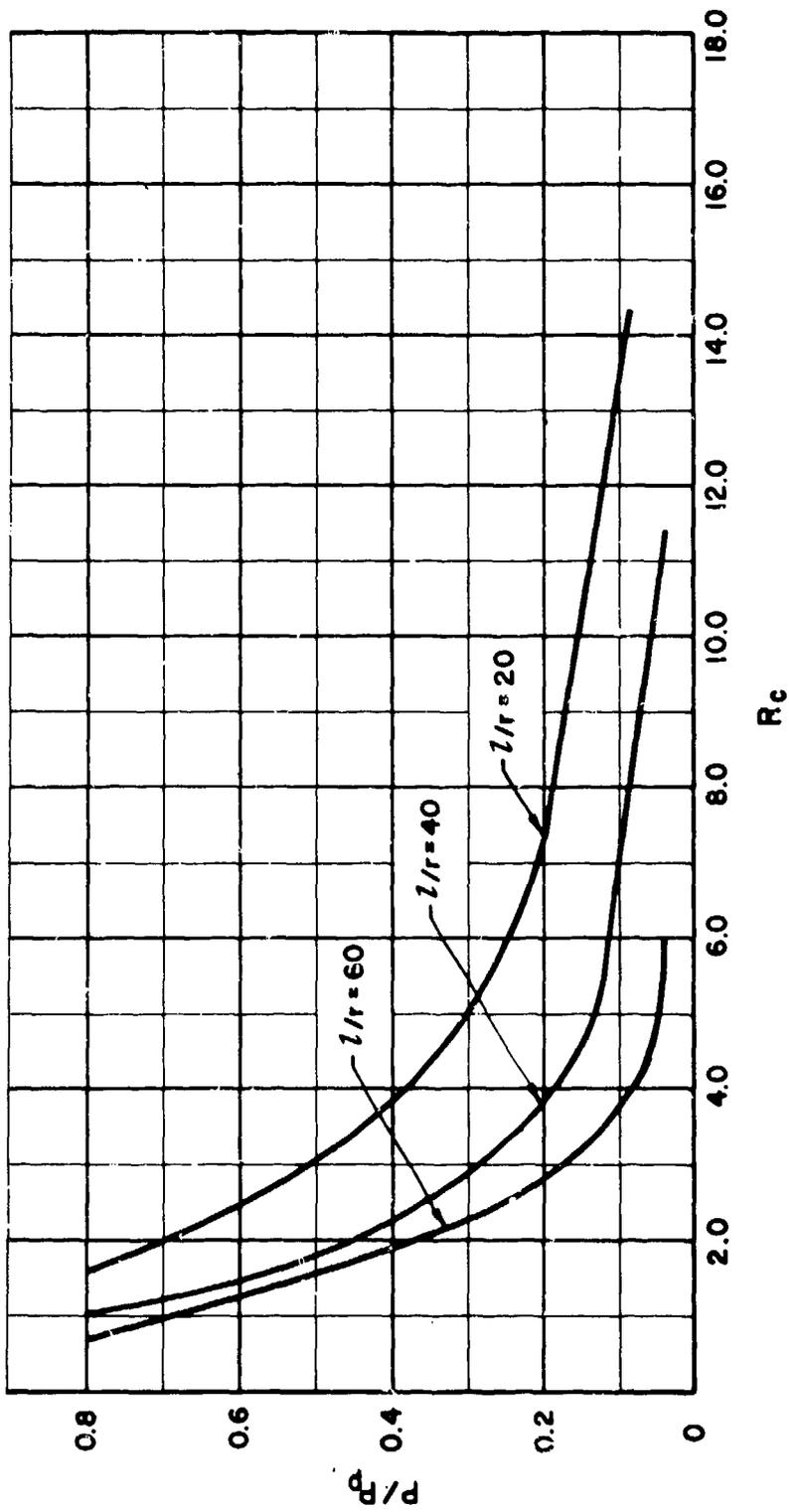


Fig 51 Rotation capacity curves for laterally loaded beam column - elastic restraint one end, other end pinned: distribution factor = 0.250

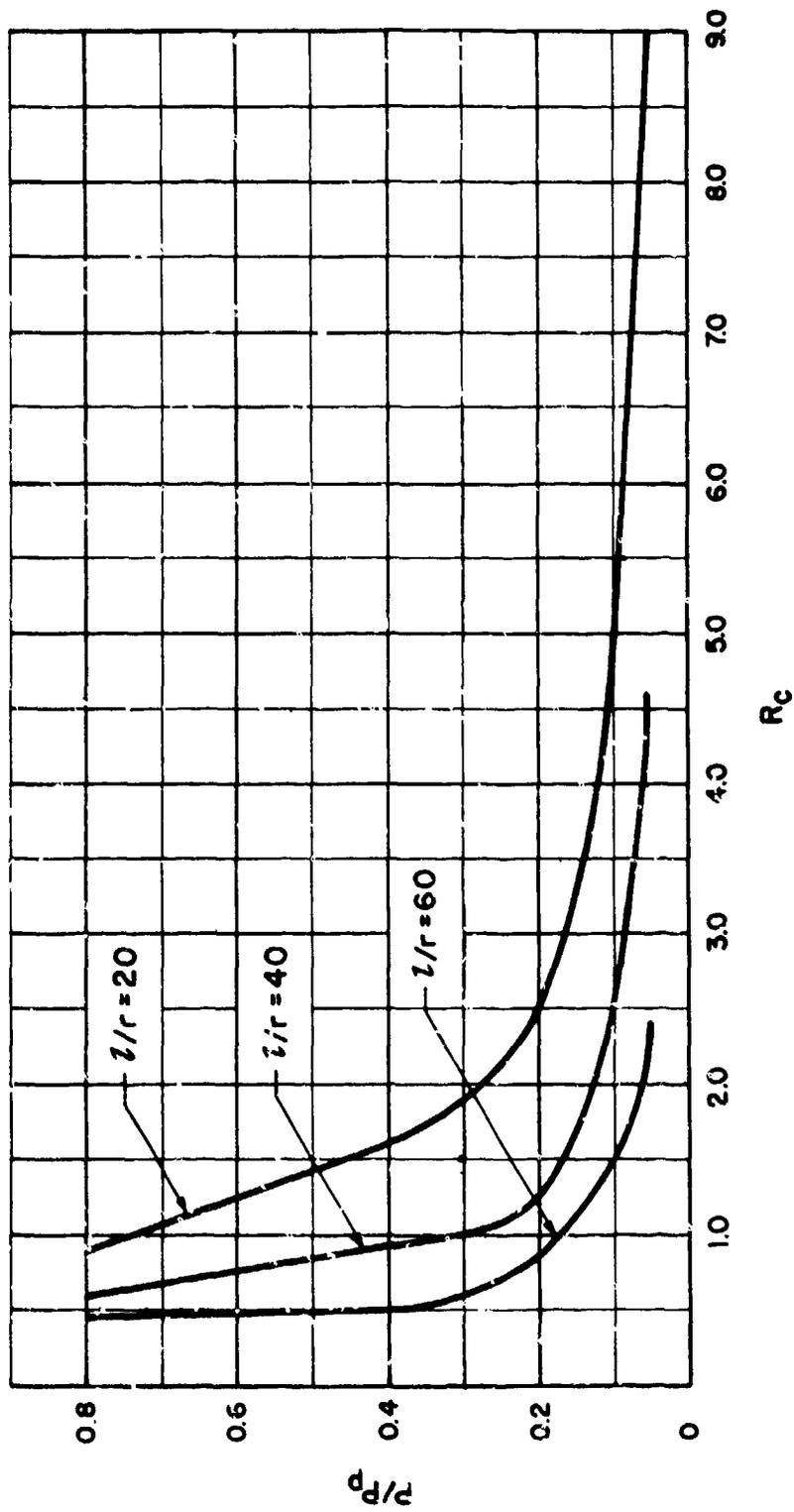


Fig 52 Rotation capacity curves for laterally loaded beam column - elastic restraint one end, other end pinned: distribution factor = 0.500

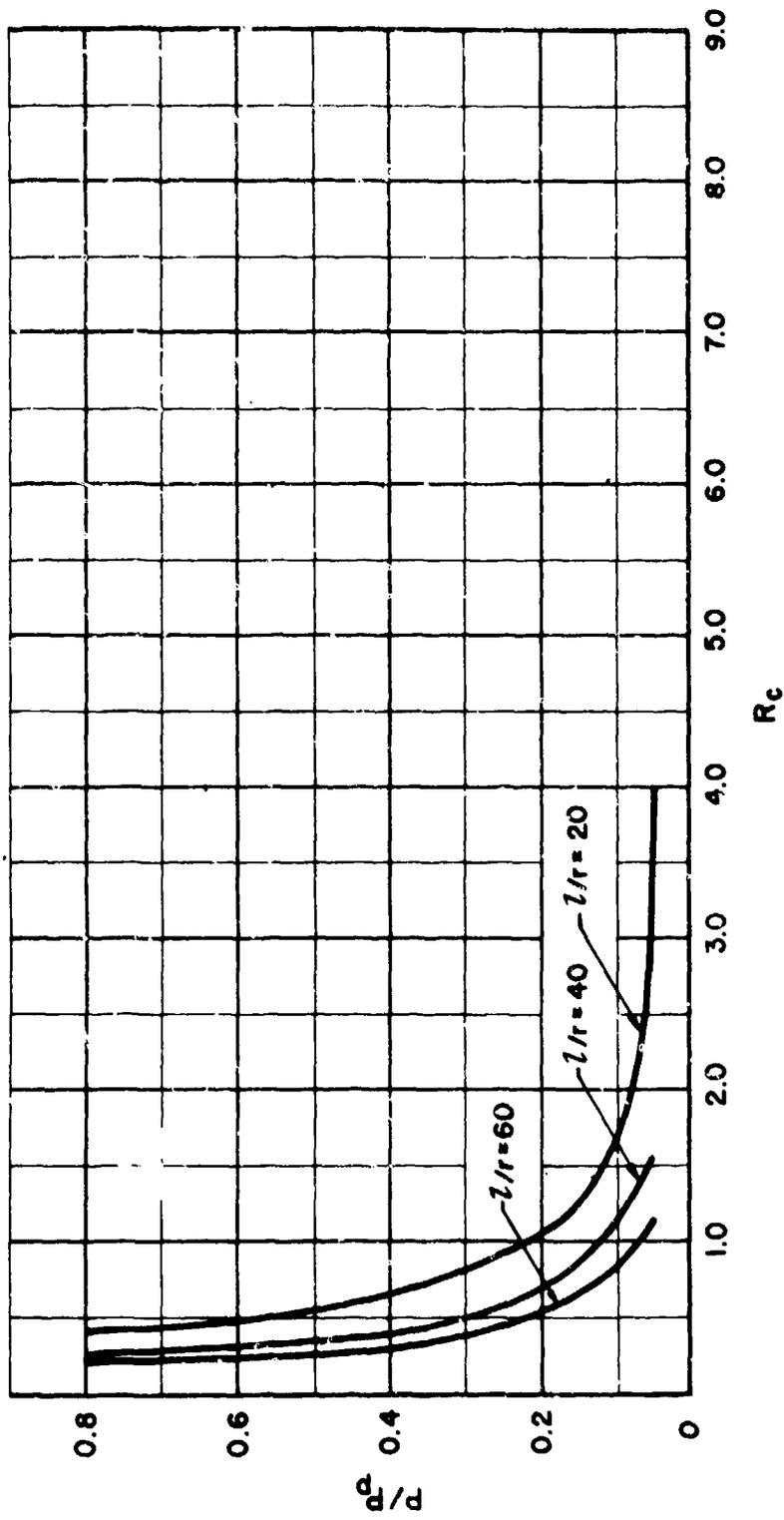
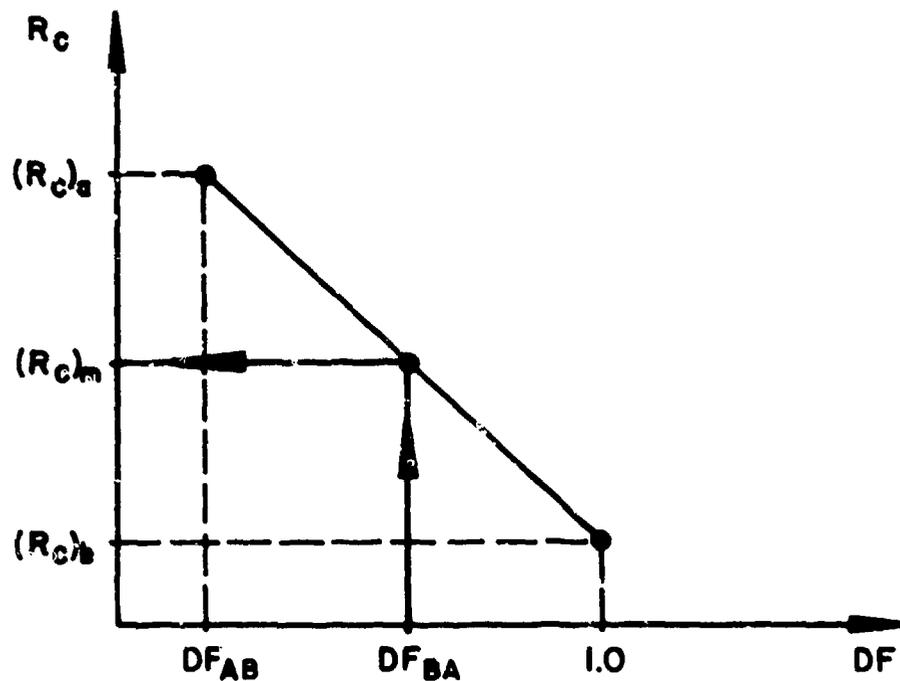


Fig 53 Rotation capacity curves for laterally loaded beam column - elastic restraint one end, other end pinned: distribution factor = 0.750

CASE	DISTRIBUTION FACTORS (DF)		ROTATION CAPACITY FROM CURVES
	END A	END B	
a. BOTH ENDS RESTRAINED	DF_{AB}	DF_{AB}	$(R_c)_a$
b. END A RESTRAINED; END B PINNED	DF_{AB}	1.0	$(R_c)_b$

a.) TABULATION OF DATA



b.) ROTATION CAPACITY vs DF AT END B
FOR CONSTANT DF AT END A

Fig 54 Interpolation of rotation capacity data for member with unequal distribution factors at ends

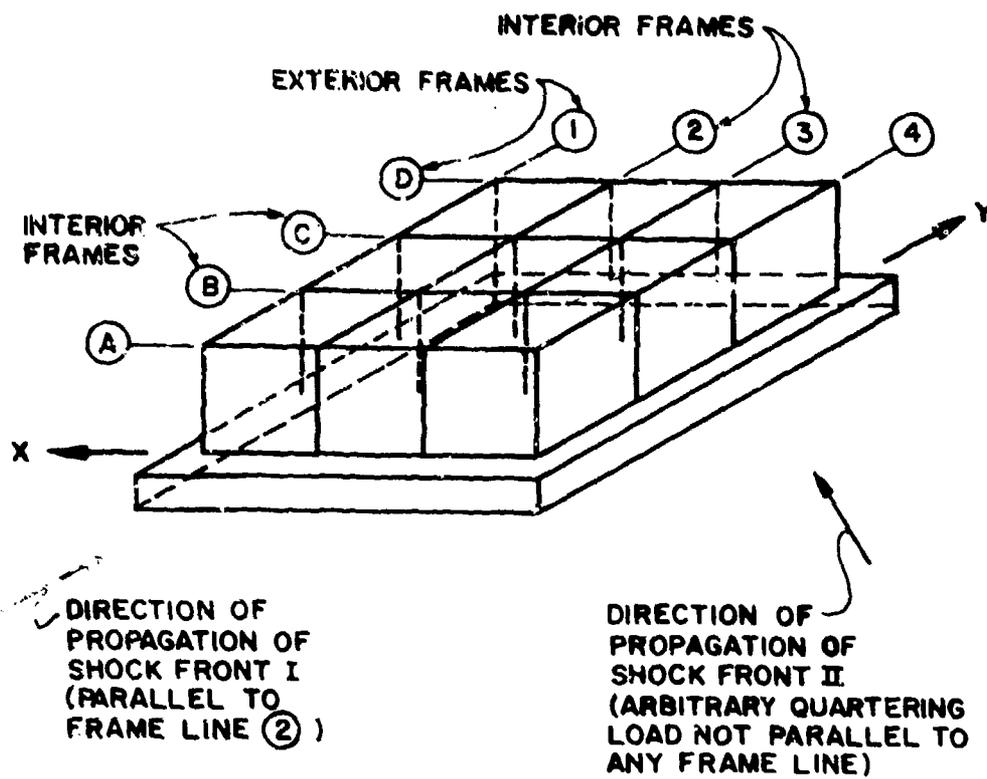


Fig 55 Typical rectangular building rigidly framed in two directions

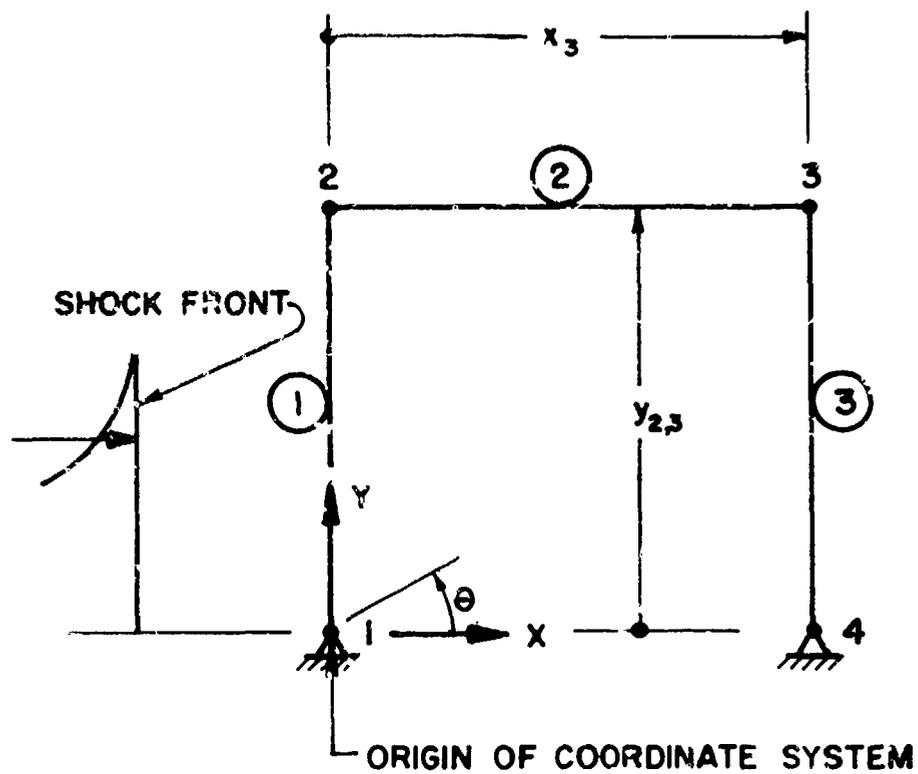
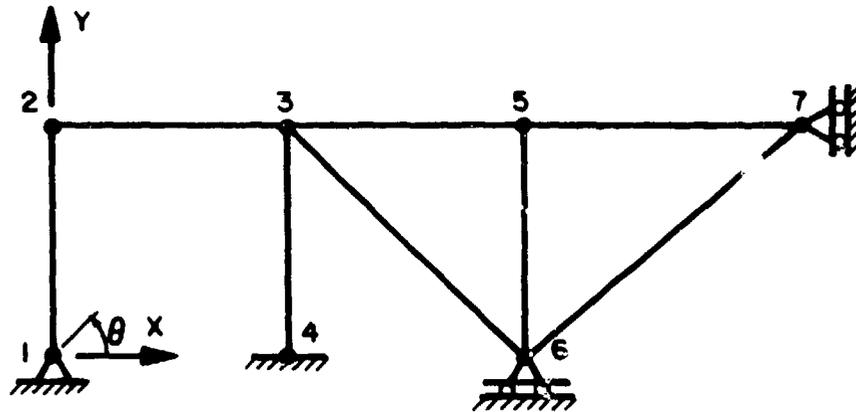


Fig 56 Identification numbers for the nodal points and elements of a typical model



a) MODEL

NODE NO.	r_x^*	r_y^*	r_θ^*
1	1	1	0
4	1	1	1
6	0	1	0
7	1	0	0

* 1 = DISPLACEMENT OR ROTATION RESTRAINED
 0 = NO RESTRAINT

b) RESTRAINTS

Fig 57 Specification of restraints for common types of support conditions

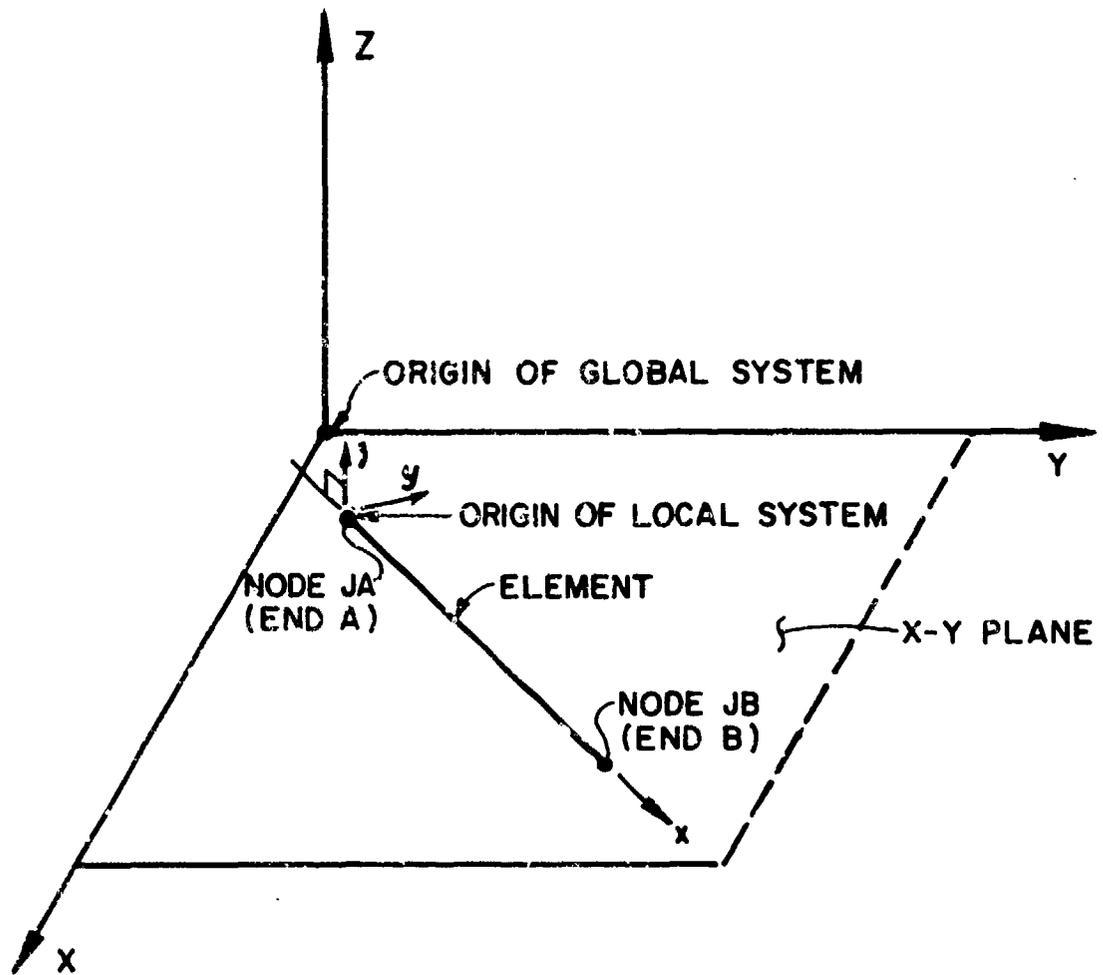
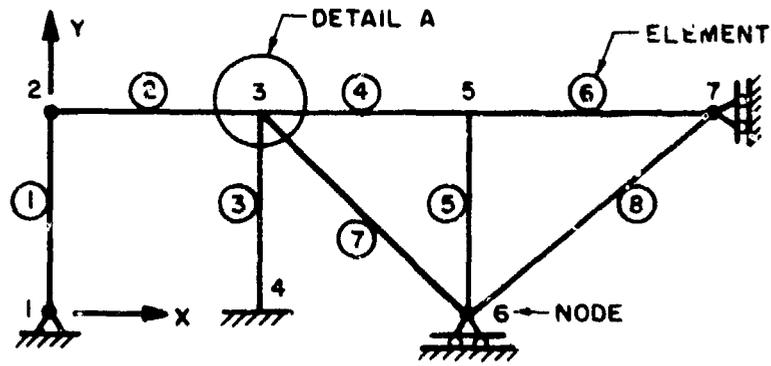
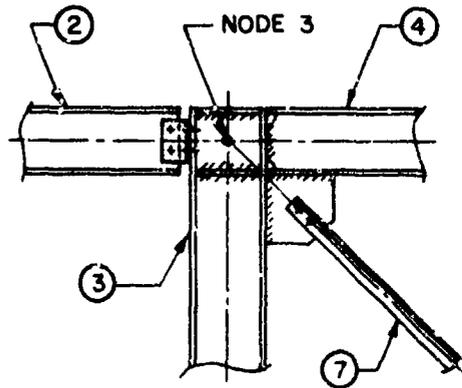


Fig 58 Local coordinate system for element



a) MODEL



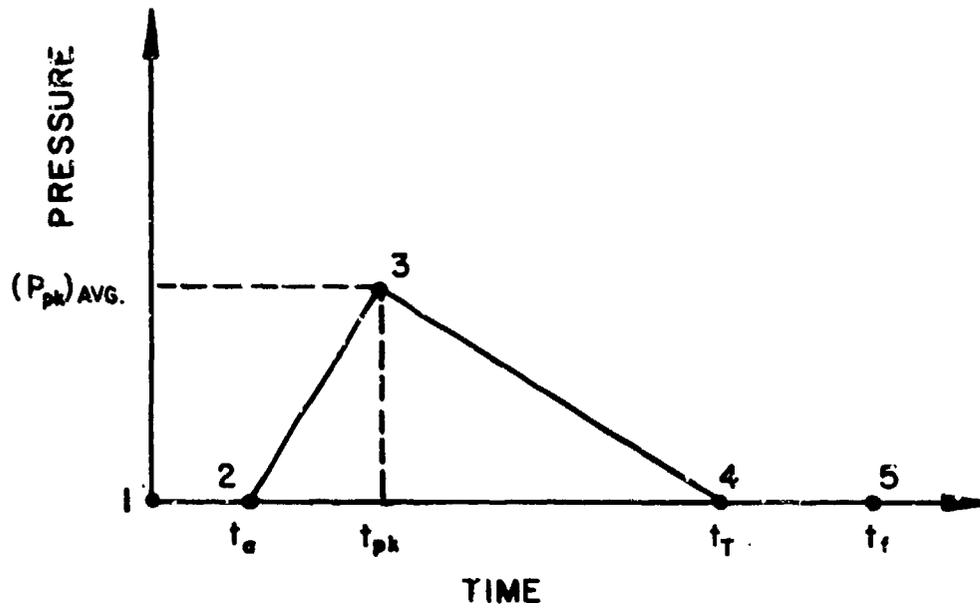
b) DETAIL A

ELEMENT ID. NO.	PIN CODE* AT NODE 3
2	1
3	0
4	0
7	1

* 1 = LOCAL PIN
0 = NO PIN

c) PIN CODES

Fig 59 Specification of pin codes for common types of frame connections

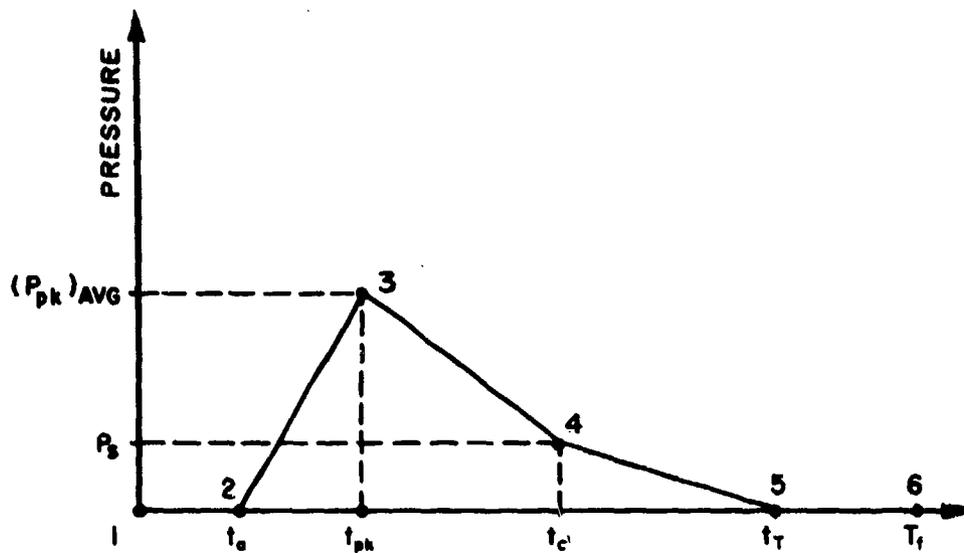


a. TYPICAL PRESSURE WAVEFORM

b. DIGITIZED PRESSURE TIME DATA

<u>POINT</u>	<u>PRESSURE</u>	<u>TIME</u>
1	0.0	0.0
2	0.0	t_o
3	$(P_{pk})_{AVG.}$	t_{pk}
4	0.0	t_T
5	0.0	t_f

Fig 60 Digitized pressure-time data for pressure waveform with linear decay



a) TYPICAL PRESSURE WAVEFORM WITH BI-LINEAR DECAY

<u>POINT</u>	<u>PRESSURE</u>	<u>TIME</u>
1	0.0	0.0
2	0.0	t_0
3	$(P_{pk})_{AVG}$	t_{pk}
4	P_s	t_c
5	0.0	t_T
6	0.0	T_f

b) DIGITIZED PRESSURE TIME DATA

Fig 61 Digitized pressure-time data for reflected pressure waveform with bi-linear decay

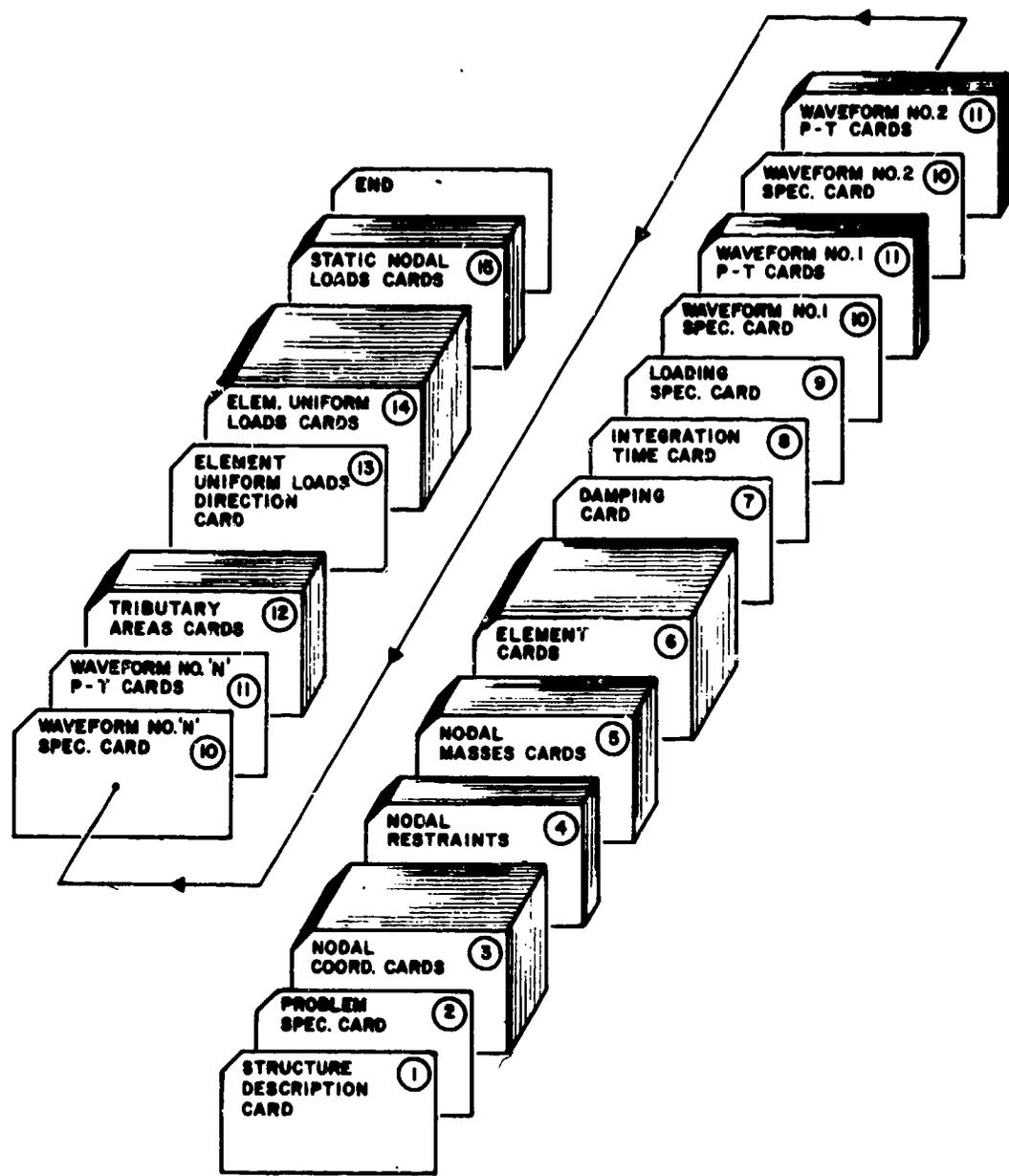
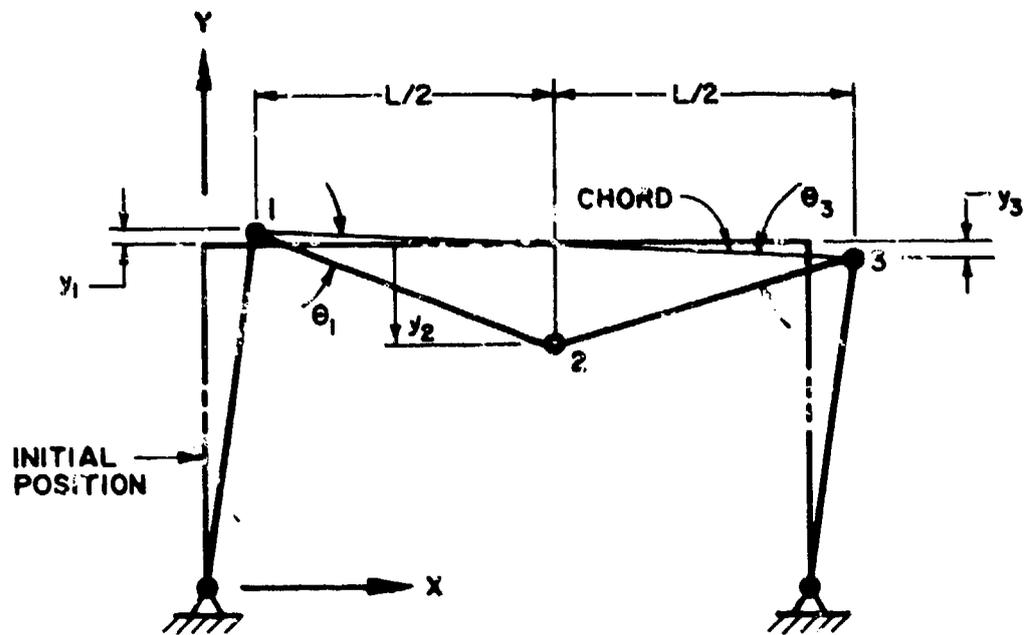


Fig 62 Arrangement of input data deck



Fig 63 Positive directions of element axial loads, shears and bending moments in local coordinate system



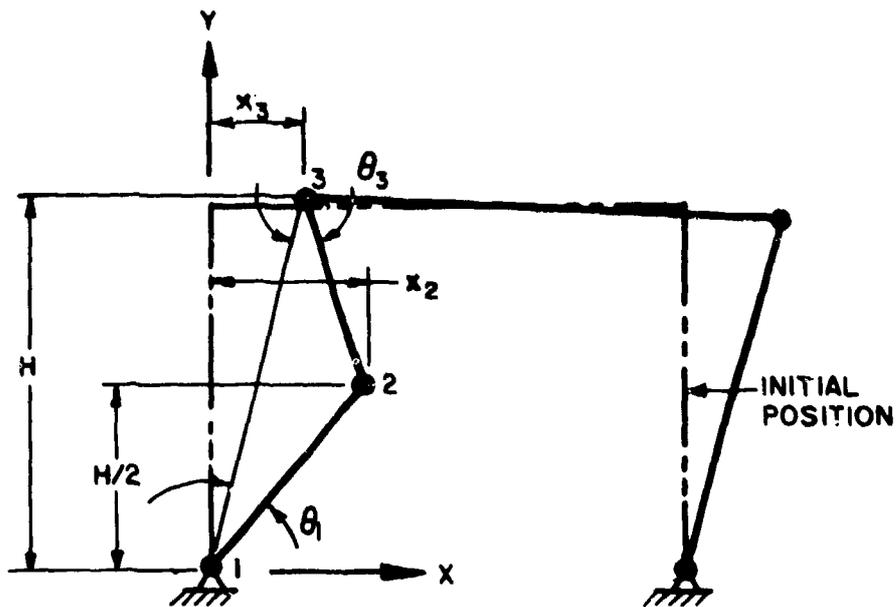
a) DEFORMED CONFIGURATION OF STRUCTURE

$$\theta_1 = \theta_3 = \tan^{-1}(y_2/L/2)$$

y_1 AND y_3 ASSUMED NEGLIGIBLE

b) COMPUTATION OF CHORDAL ANGLES θ_1 & θ_3
FOR GIRDER

Fig 64 Measurement of chordal angles for girder



a) DEFORMED CONFIGURATION OF STRUCTURE

$$\theta_1 = \theta_3 = \frac{(x_2 - x_3 / 2)_{\max.}}{H/2}$$

b) COMPUTATION OF CHORDAL ANGLES θ_1 & θ_3 FOR EXTERIOR COLUMN

Fig 65 Measurement of chordal angles for exterior column

APPENDIX A
ILLUSTRATIVE EXAMPLES

APPENDIX A

ILLUSTRATIVE EXAMPLES

A.1 Introduction

This appendix presents detailed procedures and numerical examples on various topics related to the analysis and design of single-story rigid frame steel structures subjected to blast overpressures. The topics covered are:

1. Preliminary design of interior frames of buildings subjected to normal shock waves.
2. Preliminary design of primary structural frames of buildings subjected to quartering shock waves.
3. Formulation of analytical models for interior frames of buildings subjected to normal shock waves.
4. Formulation of analytical models for representative frames of buildings subjected to quartering shock waves.
5. Use of DYNFA to verify the blast resistance of interior frames of buildings subjected to normal shock waves.
6. Use of DYNFA to verify the blast resistance of primary framing systems of buildings subjected to quartering shock waves.

References are made to the appropriate parts in Sections 1 through 9 of this report and to charts, tables and equations from other design manuals and specifications.

The basic objective of the material presented herein is to illustrate the methods for selecting and verifying the member sizes for the primary framing system of a single-story steel building. As such, little attention was given in the examples to actual framing details since these considerations have little bearing on the manner in which the design procedures are used. In addition, to facilitate the computations, the sizes of the secondary members were standardized throughout the building (i.e., all girts were made the same, all purlins were made the same, etc.). Also, the sizes of the transverse girders that were used in Example A.3 were established on the basis of the design for the girder at the blastward wall, which involved bi-axial bending of the member.

A.2 Preliminary Design of Single-Story Frames

Problem A.1: Preliminary design of an unbraced, interior, rigid frame of a single-story steel building subjected to a normal shock wave.

Procedure:

Step 1. Establish the blast load parameters:

- a. Location of charge
- b. Charge weight, W
- c. Safety factor, SF
- d. Normal distance, R_A , between the center of the charge and the blastward wall of the building

Calculate the effective charge weight,

$$W_E = (1.0 + SF)W.$$

Step 2. Determine the peak pressures, p_v and p_h , for the preliminary design:

- a. Calculate $Z = R_A/W_E^{1/3}$
- b. Enter Figure 4-5 or 4-12 of Reference 3 or Figure 5 of the text with Z and read:

Peak positive incident pressure, P_{s0}

Peak positive normal reflected pressure, P_r .

The data in Figure 6 of the text were derived from the test results reported in Reference 21.

- c. For structures located in high pressure regions, determine q_0 for P_{s0} from Figure 4-66 of Reference 3. Calculate $C_D q_0$ for the roof. Obtain C_D from paragraph 4-74c of Reference 3.
- d. For all cases: $p_h = P_r$.

For cases involving low-to-intermediate pressure levels:

$$P_V = P_{S0}$$

For cases involving high pressure levels:

$$P_V = P_{S0} + C_D q_0$$

Step 3. Establish the design parameters:

- a. Geometry of the frame
- b. Support conditions
- c. Post-explosion condition of the frame (reusable or non-reusable)
- d. Static tensile stress, F_y
- e. Dynamic increase factor, c
- f. Modulus of elasticity, E

Also calculate the dynamic tensile stress,

$$F_{dy} = cF_y.$$

Step 4. Calculate the ratio of the total horizontal-to-vertical peak loadings, α , acting on the frame using Equation (1) of the text. Also determine the dynamic load factors (DLF's) for the beam and panel mechanisms (Table 4 of the text) and compute the corresponding equivalent static loads, w_b and w_p ; tabulate these loads (w_b and w_p) as shown in Table 1.

Step 5. Substitute the values (determined in Step 4) for α and either w_b or w_p (depending upon the type of mechanism) into the appropriate equations of Table 1 of the text. Tabulate, as shown in Table 1 of this appendix, the resulting equations which express M_p in terms of C and/or C_1 .

Step 6. By assuming several sets of values for constants C and C_1 , and using the relationships for M_p determined in Step 5, calculate the values of

the plastic moments, M_p , for the various mechanisms. Tabulate the results as shown in Table 1. For each set of values of C and C_1 , the largest computed value of M_p , from among those computed for all mechanisms, establishes the governing collapse mechanism. The selection of preliminary member sizes is based on several trials in which the values of C and C_1 , and the corresponding maximum value of M_p , are minimized such that an economical design is produced. Here, engineering judgement is required, as several sets of values of C and C_1 may yield similar values for the maximum M_p .

Step 7. Using the moment capacities of each member (determined in Step 6) and Equation (4), calculate the plastic section modulus and select the member size.

Step 8. Establish the minimum thickness requirements for the frame members using Equation (8) and Table 5 of the text and determine whether the member sizes selected meet these requirements. Also, compute the slenderness ratio in the plane of bending for each member and compare this quantity with the limiting value given by Equation (4.1) of Reference 1.

Step 9. For each member selected, calculate the plastic moment capacity and effect the reduction in this moment capacity due to lateral torsional buckling using Equation (5). Check whether this reduced bending capacity, M_{mn} , satisfies the uni-axial bending equation [Eq (7)].

Example A.1: Preliminary design of an unbraced, interior, rigid frame of a single-story steel building subjected to a normal shock wave.

Required: Design an interior frame of the building shown in Figure 1 for the blast loadings produced by a normal shock wave.

Step 1. Given:

- a. Location of charge, surface burst
- b. Charge weight, $W = 1,140$ kg (2,500 lb)

- c. Safety factor SF = 20 percent
(Section 1-5, Ref 3)
- d. Normal distance from center of charge
to blastward wall:

$$R_A = 132 \text{ m (433 ft)}$$

Effective charge weight:

$$W_E = (1.0 + SF)W = 1.2(1,140) \\ = 1,368 \text{ kg (3,000 lb)}$$

Step 2. Determine p_v and p_h for the preliminary design:

$$a. Z = R_A/W_E^{1/3} = 433/3,000^{1/3} = 30 \text{ ft/lb}^{1/3}$$

Note: Since Figure 4-12 (Ref 3) is expressed
in U.S. units, Z is also calculated in
U.S. units.

- b. From Figure 4-12 of Reference 3:

$$P_{S0} = 1.65 \text{ psi or 11.4 kPa}$$

$$P_r = 3.50 \text{ psi or 24.2 kPa}$$

- c. Since the structure is located in a low
pressure region, the drag pressure on the
roof is neglected.
- d. The peak pressures acting on the frame are:

$$p_v = P_{S0} = 11.4 \text{ kPa (1.65 psi)}$$

$$p_h = P_r = 24.2 \text{ kPa (3.50 psi)}$$

Step 3. Given:

- a. Frame geometry (Fig 1)
- b. Pin-ended column supports
- c. Reusable structure
- d. $F_y = 248(10)^3 \text{ kPa (36 ksi)}$

e. $c = 1.1$ (Table 2.1, Ref 1)

f. $E = 207(10)^6 \text{ kPa}$ ($30 \times 10^6 \text{ psi}$)

$$F_{dy} = cF_y = 1.1 \times 248 \times 10^3 \\ = 273 \times 10^3 \text{ kPa} \text{ (39.6 ksi)}$$

Step 4. Calculate α , w_b and w_p :

From Figure 1:

$$b_v = b_h = 6.0 \text{ m}$$

$$q_h = p_h b_h = 24.2 \times 6.0 = 145 \text{ kN/m}$$

$$q_v = p_v b_v = 11.4 \times 6.0 = 68.4 \text{ kN/m}$$

$$\alpha = q_h/q_v = 145/68.4 = 2.12 \text{ [Eq (1)]}$$

For normal shock wave, angle of incidence between shock front and blastward wall is zero; hence,

$$R_{\alpha} \text{ of Table 4 of text} = 0.0, \text{ and}$$

$$DLF_b \text{ (beam mechanism)} = 1.0$$

$$DLF_p \text{ (panel mechanism)} = 0.5$$

$$w_b = 1.0q_v = 1.0(68.4) = 68.4 \text{ kN/m}$$

$$w_p = 0.5q_v = .5(68.4) = 34.2 \text{ kN/m}$$

Step 5. Substitute α and either w_b or w_p into the equations of Table 1 of the text.

Substituting the values of α and w_b or w_p into the appropriate equations (Table 1 of the text) for collapse mechanisms 1 and 5b, yields the following expressions for M_p :

Collapse mechanism 1:

$$M_p = (w_b L^2)/16 = [68.4(6)^2]/16 = 154 \text{ kN-m}$$

Collapse mechanism 5b:

$$M_p = \frac{(3/8)(\alpha w_p H^2)}{C + (n - 1/2)} = \frac{(3/8)(2.12)(34.2)(5.0)^2}{C + (2 - 1/2)}$$

$$= 680/(C + 3/2)$$

Expressions of M_p for the remaining collapse mechanisms are tabulated in Table 1 of this appendix.

Step 6. Calculate M_p for several trial values of C and C_1 . Trial calculations of M_p for collapse mechanisms 1 and 5b are shown below. The calculations are performed for three sets of trial values of C and C_1 . Values of M_p for the remaining collapse mechanisms, which are computed with the same trial values of C and C_1 , are tabulated in Table 1 of this appendix.

Collapse mechanism 1:

For all values of C and C_1 , $M_p = 154$ kN-m

Collapse mechanism 5b:

$$\begin{aligned} \text{For } C = 2 \text{ and } C_1 = 2, M_p &= 680/(2 + 3/2) \\ &= 194 \text{ kN-m} \end{aligned}$$

$$\begin{aligned} \text{For } C = 2.5 \text{ and } C_1 = 2, M_p &= 680/(2.5 + 3/2) \\ &= 170 \text{ kN-m} \end{aligned}$$

$$\begin{aligned} \text{For } C = 1.5 \text{ and } C_1 = 2, M_p &= 680/(1.5 + 3/2) \\ &= 227 \text{ kN-m} \end{aligned}$$

Inspection of the results listed in Table 1 for the three trials investigated, reveals that a value of $M_p = 227$ kN-m (same in all three trials) is the minimum required plastic bending capacity for the girders. Also Trial 3 yields the lowest value of C , thereby minimizing the required capacities of the exterior columns.

Therefore, the required bending capacities are:

For the girder: $(M_p)_x = 227$ kN-m (167 kip-ft)

For the interior column:

$$C_1(M_p)_x = 2.0(227) = 454 \text{ kN-m (334 kip-ft)}$$

For the exterior column:

$$C(M_p)_x = 1.5(227) = 341 \text{ kN-m (251 kip-ft)}$$

Step 7. Calculate plastic section moduli and select member sizes.

Girder: Use Equation (4) for computing plastic modulus, Z_x

$$\begin{aligned} Z_x &= (M_p)_x / F_{dy} = 227 / (273 \times 10^3) \\ &= 0.832 \times 10^{-3} \text{ m}^3 (50.5 \text{ in}^3) \end{aligned}$$

Try W12 x 40:

$$Z_x = 57.5 \text{ in}^3 = 0.942 \times 10^{-3} \text{ m}^3$$

(Ref 14)

The member sizes selected for the columns are listed in Table 2.

Step 8. Establish the minimum thickness requirements for the frame members.

The limiting depth-thickness ratio for the web of each frame member is:

$$d/t_w \leq 29,060 / \sqrt{F_y} \quad [\text{Eq (8)}]$$

$$F_y = 248 \times 10^3 \text{ kPa}$$

$$d/t_w \leq 29,060 / \sqrt{248 \times 10^3} \leq 58.4$$

The limiting width-thickness ratio for the flange of each frame member is:

$$b_f / 2t_f \leq 8.5 \text{ (from Table 5 of text)}$$

Check d/t_w and $b_f / 2t_f$ for the girder.

From the "Properties for designing" tables in Reference 14, the ratios d/t_w and $b_f / 2t_f$ for a W12 x 40 member are:

$$d/t_w = 40.6 < 58.4$$

$$b_f / 2t_f = 7.75 < 8.5$$

Note: Values of M_{mx} and $(M_p)_x/M_{mx}$ for interior and exterior columns are given in Table 2.

Problem A.2: Preliminary design of the primary framing system of a single-story rigid frame steel building subjected to a quartering shock wave.

Procedure:

Step 1. Establish the blast load parameters:

- a. Location of charge
- b. Charge weight, W
- c. Safety factor, SF
- d. Radial distances between center of charge and the following locations:
 1. Nearest corner of building
 2. Farthest point on each blastward wall
- e. Angle of incidence, α_i , between the shock front and each blastward wall, measured at the points listed above.

Calculate the effective charge weight.

$$W_E = (1.0 + SF)W.$$

Step 2. Compute the incident and drag pressures at each location specified in Step 1d:

- a. Calculate $Z = R/W_E^{1/3}$
- b. Enter Figure 4-5, 4-11 or 4-12 of Reference 3 or Figure 6 of the text with Z and read the peak positive incident pressure, P_{S0} .
- c. For structures located in high pressure regions, determine the dynamic pressure, q_0 , for P_{S0} from Figure 4-66 of Reference 3.

Calculate $C_D q_0$ for the roof. Obtain C_D from paragraph 4-14c of Reference 3.

- d. Tabulate all values as shown in Table 3.

Step 3. Determine the peak reflected pressures at the blastward and leeward ends of each blastward wall:

- a. Enter Figure 4-6 with P_{s0} and α_i (from Step 1e) at the point of interest on the wall; read the reflected pressure coefficient, $C_{r\alpha}$.
- b. Calculate $P_{r\alpha} = C_{r\alpha} P_{s0}$
- c. Tabulate all values as shown in Table 3.

Step 4. Using data listed below, make a comparison of the horizontal and vertical blast loads acting on all frames positioned normal to each blastward wall:

1. Peak pressures determined in Steps 2 and 3
 2. Distances between adjacent frames
 3. Secondary framing plan of roof and walls.
- b. On the basis of the comparison of Step 4a, select several representative frames, spanning in each direction, for analysis with DYNFA.
 - c. If analyses of several interior frames, spanning in each direction, are required because of large pressure differentials between the ends of the building, compute the peak pressures at the locations on each frame as specified in Section 2.2.

Step 5. Establish the following design parameters:

- a. Geometry of frames selected for analysis
- b. Orientation of columns
- c. Support conditions for each frame
- d. Post explosion condition of structure (reusable or non-reusable)

- e. Static tensile strength, F_y
- f. Dynamic increase factor, c
- g. Modulus of elasticity, E

Calculate the dynamic tensile stress,

$$F_{dy} = cF_y.$$

- Step 6.
- a. Establish the peak pressures, P_v and P_h for the preliminary design of each frame using the blast pressures determined in Step 2 (and Step 3 when large pressure differentials exist between ends of building).
 - b. Calculate the parameters, α , w_b and w_p for each frame as described in Step 4 of Problem A.1. Tabulate w_b and w_p as shown in Tables 1 and 4.
- Step 7. Perform Step 5 of Problem A.1 for each frame and tabulate the results as shown in Tables 1 and 4.
- Step 8.
- a. Perform Step 6 of Problem A.1 for each frame and tabulate the results as shown in Tables 1 and 4.
 - b. Tabulate the required plastic bending capacities for all primary frame members. When preparing this tabulation, the required bending capacities for the members of one frame can be specified for the members of other frames that have identical configurations, and are subjected to similar loadings.
- Step 9. Calculate the plastic section moduli of the frame members using the plastic bending capacities determined in Step 8b and either Equation (3) (for members designed for bi-axial bending) or Equation (4) (for members designed for uni-axial bending). Select member sizes on the basis of these section moduli.
- Step 10. Establish the minimum thickness requirements for all members using Equation (8) and Table 5 of the text. Determine whether the member sizes

selected meet these requirements. Compute the slenderness ratio(s) in the plane(s) of bending for each member and compare with the limiting slenderness ratio given by Equation (4.1) of Reference 1.

Step 11. Calculate the plastic bending capacities for the members selected. Effect the reduction in these plastic moment capacities due to lateral torsional buckling using Equations (5a) and/or (5b). Check whether the reduced bending capacities satisfy Equation (6) or (7), as the case may be.

Example A.2 Preliminary design of the primary framing system of a single-story rigid frame building subjected to a quartering shock wave.

Required: Design the primary structural frames, of the single-story building shown in Figure 1, for the blast loadings produced by a quartering shock wave.

Step 1. Given:

- a. Location of charge, surface burst, from Figure 2: $H_A = 114.3$ m; $H_B = 66.0$ m
- b. Charge weight, $W = 1,140$ kg (2,500 lb)
- c. Safety factor, $SF = 20\%$
- d. Radial distances between center of charge and the following locations (Fig 2):

1. Nearest corner of building:

$$R_I = \sqrt{(114.3)^2 + (66.0)^2} = 132 \text{ m (433 ft)}$$

2. Furthest point on wall A:

$$\begin{aligned} R_{II} &= \sqrt{(114.3)^2 + [66.0 + 3(6.0)]^2} \\ &= 141.8 \text{ m (465.2 ft)} \end{aligned}$$

3. Furthest point on wall B:

$$\begin{aligned} R_{III} &= \sqrt{[(114.2 + 2(6.0))]^2 + (66.0)^2} \\ &= 142.5 \text{ m (467.5 ft)} \end{aligned}$$

e. Angle of incidence between shock front and (Fig 2):

1. Wall A: $(\alpha_A)_I = 30^\circ$

$$(\alpha_A)_{II} = 36.4^\circ$$

2. Wall B: $(\alpha_B)_I = 60^\circ$

$$(\alpha_B)_{III} = 62.4^\circ$$

$$\text{Effective charge weight, } W_E = (1.0 + SF)W$$

$$= 1.2(1,140)$$

$$= 1,360 \text{ kg (3,000 lb)}$$

Step 2. Determine P_{SO} and C_{DQ_0} at points I, II and III (Fig 2):

For point I:

a. $Z_I = R_I/W_E^{1/3} = 433/(3,000)^{1/3} = 30.03 \text{ ft/lb}^{1/3}$

b. Entering Figure 4-12 with $Z_I = 30 \text{ ft/lb}^{1/3}$

$$P_{SO} = 1.65 \text{ psi (11.4 kPa)}$$

c. Since the structure is located in a low pressure region, the drag pressure on the roof is neglected.

d. Steps 2a, 2b and 2c are repeated for points II and III. The results are tabulated in Table 3.

Step 3. Determine the peak reflected pressures at the blastward and leeward ends of each blastward wall.

For point I on wall A:

a. Enter Figure 4-6 of Reference 3 with $P_{SO} = 1.65 \text{ psi}$ (from Step 2b) and $(\alpha_A)_I = 30^\circ$ (Step 1e).

Note: Lowest value of P_{so} for which data are provided is 3.71 psi; hence, read value of C_{ra} for $(\alpha_A)_I = 30^\circ$ and $P_{so} = 3.71$ psi

$$C_{ra} = 2.11$$

b. $P_{ra} = C_{ra} P_{so}$

$$= 2.11 \times 1.65 = 3.48 \text{ psi (24.0 kPa)}$$

c. Steps 3a and 3b are repeated for point II on wall A and points I and III on wall B. All values are tabulated in Table 3.

Step 4. a. Compare blast loads on frames positioned normal to wall A:

1. Peak pressures at points I and II from Table 3:

Reflected pressures: $(P_{ra})_I = 24.0$ kPa

$$(P_{ra})_{II} = 22.1 \text{ kPa}$$

Incident pressures: $(P_{so})_I = 11.4$ kPa

$$(P_{so})_{II} = 10.3 \text{ kPa}$$

Differences of 7.9% in reflected pressures, and 9.6% in incident pressures between points I and II are negligible.

2. Distances between adjacent frames - frames equally spaced at intervals of 6.0 meters.

3. Secondary framing plan of roof and walls - same for all bays.

4. Conclusions: negligible difference in loads acting on interior frames; negligible difference in loads acting on exterior frames; loads acting on exterior frames approximately one-half of loads acting on interior frames.

- b. On the basis of the above conclusions, the frames on column lines 1 and 2 are selected for analysis with DYNFA.

Repeating Steps 4a and 4b for the frames positioned normal to wall B results in the selection of the frames on column lines A and B for analysis with DYNFA.

It should be noted that since the blast loading on each exterior frame (column lines A and 1) is approximately one half of that on the interior frame adjacent to it (column lines B and 2, respectively), the mechanism analyses of the exterior frames are not required for the preliminary design. The required bending capacities for the members of each such frame can be taken as one half of the bending capacities for the corresponding members of the adjacent interior frame.

Step 5. Given:

- a. Geometry of frames on column lines 1, 2, A and B (Fig 1).
- b. Orientation of columns (Fig 3). The double line around the periphery of the roof plan in Figure 3 is used to indicate that two members are provided along the edges of the roof. One member, with its web in the plane of the exterior wall, is integral with the frame, and designed for uni-axial bending. The other member, with its web in the plane of the roof, is positioned such that it acts independently of the frame. This member is provided to resist the horizontal blast loadings acting on the upper portion of the exterior wall.
- c. Pinned end supports - all frames.
- d. Reusable structure.
- e. $F_y = 248(10)^3$ kPa (36 ksi)
- f. $c = 1.1$ (Table 2.1, Ref 1)

g. $E = 273(10)^6 \text{ kPa} (30 \times 10 \text{ psi})$

$$F_{dy} = cF_y = 1.1 \times 248 \times 10^3$$

$$= 273 \times 10^3 \text{ kPa} (39.6 \text{ ksi})$$

- Step 6. a.** Establish the peak pressures acting on each frame using the pressures determined in Step 2 and tabulated in Table 3.

Pressures acting on all frames normal to wall A:

$$p_h = (P_{r\alpha})_I \text{ on wall A} = 3.47 \text{ psi} (23.94 \text{ kPa})$$

$$p_v = (P_{s\alpha})_I = 1.65 \text{ psi} (11.38 \text{ kPa})$$

Pressures acting on all frames normal to wall B:

$$p_h = (P_{r\alpha})_I \text{ on wall B} = 4.06 \text{ psi} (28.01 \text{ kPa})$$

$$p_v = (P_{s\alpha})_I = 1.65 \text{ psi} (11.38 \text{ kPa})$$

- b. Calculate α , w_b and w_p for each frame:

Frame on column line B:

From Figure 1:

$$b_v = 2.0 \text{ m}; b_h = 6.0 \text{ m}$$

$$q_h = p_h b_h = 28.01(6.0) = 168.1 \text{ kN/m}$$

$$q_v = p_v b_v = 11.38(2.0) = 22.8 \text{ kN/m}$$

$$\alpha = q_h/q_v = 168.1/22.8 = 7.37 \quad [\text{Eq (1)}]$$

The dynamic load factors from Table 4 of the text are:

$$DLF_b (\text{beam mechanism 1}) = 1.0$$

$$DLF_b (\text{beam mechanism 2}) = R_\alpha$$

$$DI \Gamma_p \text{ (panel mechanism)} = 0.5R_\alpha$$

$$R_\alpha = 1.0 - h \sin \alpha_1 / S$$

$$S = 5.0 \text{ m}$$

$$\alpha_1 = (\alpha_B)_1 = 60^\circ$$

$$R_\alpha = 1.0 - 6.0 \sin 60^\circ / 8(5) = 0.87$$

$$\begin{aligned} (w_b)_1 &= 1.0q_v = 1.0(22.8) \\ &= 22.8 \text{ kN/m (beam mechanism 1)} \end{aligned}$$

$$\begin{aligned} (w_b)_2 &= R_\alpha q_v = 0.87(22.8) \\ &= 19.8 \text{ kN/m (beam mechanism 2)} \end{aligned}$$

$$\begin{aligned} w_p &= 0.5R_\alpha q_v = 0.5(0.87)(22.8) \\ &= 9.9 \text{ kN/m (panel mechanism)} \end{aligned}$$

Frame on column line 2:

The calculation of these values for the frame on column line 2 is illustrated in Step 4 of Example A.1. Similar calculations for $(\alpha_A)_1 = 30^\circ$ yield the following results:

$$(w_b)_1 = 68.7 \text{ kN/m (beam mechanism 1)}$$

$$(w_b)_2 = 63.3 \text{ kN/m (beam mechanism 2)}$$

$$w_p = 31.6 \text{ kN/m (panel mechanism)}$$

Step 7. Substitute the values of α , w_b and w_p determined in Step 6b into the equations for the various collapse mechanisms in Table 1 of the text.

For the frame on column line B:

Collapse mechanism 1:

$$M_p = (w_b)_1 L^2 / 16 = [(22.8)(6.0)^2] / 16 = 51.3 \text{ kN-m}$$

Collapse mechanism 5b:

$$M_p = \frac{(3/8)(\alpha_w r_p H^2)}{C + (n - 1/2)} = \frac{(3/8)(7.37)(9.9)(5.0)^2}{C + (3 - 1/2)}$$
$$= 684/(C + 5/2)$$

Expressions for M_p for the remaining collapse mechanisms are given in Table 4.

For the frame on column line 2:

The calculation of the expressions for M_p for the frame on column line 2 is illustrated in Step 5 of Example A.1.

- Step 8. a. For each frame, calculate M_p for several trial values of C and C_1 .

For the frame on column line B:

Collapse mechanism 1:

For all values of C and C_1 , $M_p = 51$ kN-m

Collapse mechanism 5b:

$$\text{For } C = 3 \text{ and } C_1 = 2, M_p = 684/(3 + 5/2)$$
$$= 124 \text{ kN-m}$$

$$\text{For } C = 2.5 \text{ and } C_1 = 2, M_p = 684/(2.5 + 5/2)$$
$$= 137 \text{ kN-m}$$

Inspection of these results and those listed in Table 4, for the two trials investigated, reveals that value of $M_p = 152$ kN-m (same in both trials) is the minimum required plastic bending capacity for the girders of this frame. Also, Trial 2 yields the lowest value of C , thereby minimizing the required capacities of the exterior columns.

Therefore, the required bending capacities are:

For the girder, $(M_p)_x = 152$ kN-m (111 kip-ft)

For the interior column, $C_1(M_p)_x = 2.0(152)$

$$= 304 \text{ kN-m (222 kip-ft)}$$

For the exterior column, $C(M_p)_x = 2.5(175)$

$$= 380 \text{ kN-m (278 kip-ft)}$$

For the frame on line 2:

The calculation of M_p for the frame on column line 2 is illustrated in Step 6 of Example A.1.

The following results were obtained for this frame using the loads determined in Step 6b of this example:

For the girder: $(M_p)_x = 210 \text{ kN-m (155 kip-ft)}$

For the interior column:

$$C_1(M_p)_x = 420 \text{ kN-m (309 kip-ft)}$$

For the exterior column:

$$C(M_p)_x = 315 \text{ kN-m (232 kip-ft)}$$

- b. The required bending capacities for all members of the selected frames are listed in Table 5. The quantities in italics were computed from the mechanism analyses of the frames on column lines 2 and B. The remaining quantities were extrapolated from the results of these analyses as follows:

1. The required bending capacities for the members of the frames on column lines A and C are taken as one-half of those computed for the corresponding members of the frame on column line B. Based on this, the required x-axis bending capacities for members C4 and G2, and the required y-axis bending capacity for column C3 are taken as one-half of the bending capacities for members C2, G1 and C1, respectively.

2. The required bending capacities for the members of the frame on column line 3 are taken as equal to those for the corresponding members of the frame on column line 2.
3. The required bending capacities for the members of the frames on column lines 1 and 4 are taken as one-half of those computed for the corresponding members of the frame on column line 2. Therefore, the required x-axis bending capacity of girder G4 is taken as one-half of the that for girder G3. In addition, the required y-axis bending capacities for columns C2 and C4 are taken as one-half of the required x-axis bending capacity for columns C1 and C3, respectively.

Step 9. Calculate the plastic section moduli of the frame members and select member sizes.

Girder G1:

$$Z_n = (M_p)_n / F_{dy} \quad [\text{Eq (4)}]$$

Member bends about x-axis;

$$(M_p)_x = 152 \text{ kN-m (from Table 5)}$$

$$Z_x = 152 / (273 \times 10^3) = 0.557 \times 10^{-3} \text{ m}^3 \text{ (34.0 in}^3\text{)}$$

Try W12 x 27:

$$Z_x = 38.0 \text{ in}^3 = 0.622 \times 10^{-3} \text{ m}^3$$

Column C1:

$$Z_x = (M_p)_x / F_{dy} \quad [\text{Eq (3a)}]$$

$$Z_y = (M_p)_y / F_{dy} \quad [\text{Eq (3b)}]$$

From Table 5:

$$(M_p)_x = 420 \text{ kN-m; } (M_p)_y = 304 \text{ kN-m}$$

$$Z_x = 420 / (273 \times 10^3) = 1.54 \times 10^{-3} \text{ m}^3 \text{ (93.9 in}^3\text{)}$$

$$Z_y = 304 / (273 \times 10^3) = 1.11 \times 10^{-3} \text{ m}^3 \text{ (67.7 in}^3\text{)}$$

Try W14 x 111:

$$Z_x = 196 \text{ in}^3 = 3.22 \times 10^{-3} \text{ m}^3 > 1.54 \times 10^{-3} \text{ m}^3$$

$$Z_y = 94 \text{ in}^3 = 1.54 \times 10^{-3} \text{ m}^3 > 1.11 \times 10^{-3} \text{ m}^3$$

The member sizes for the other members are determined in a similar manner and listed in Table 5.

Step 10. Establish the minimum thickness requirements and compute the limiting slenderness ratio for all members.

The minimum thickness requirements and limiting slenderness ratio are the same as computed in Step 8 of Example A.1.

$$d/t_w \leq 58.4 \quad [\text{Eq (8)}]$$

$$b_f/2t_f \leq 8.5 \quad (\text{Table 5})$$

$$l/r \leq C_c = 122 \quad [\text{Eq (4.1) of Ref 1}]$$

For girder G1: W12 x 27

$$d/t_w = 50.5 < 58.4$$

$$b_f/2t_f = 8.12 < 8.5$$

$$l = 6.0 \text{ m}$$

$$r = 5.07 \text{ in (0.129 m)}$$

$$l/r = 6.0/0.129 = 46.5 < 122$$

For column C1: W14 x 111

$$d/t_w = 26.6 < 58.4$$

$$b_f/2t_f = 8.37 < 8.5$$

For bending about the x-axis:

$$r_x = 6.23 \text{ in (0.158 m)}$$

$$l = 5.0 \text{ m}$$

$$l/r_x = 5.0/0.158 = 31.6 < 122$$

For bending about the y-axis:

$$r_y = 3.73 \text{ in (0.095 m)}$$

$$l/r_y = 5.0/0.0951 = 52.6 < 122$$

The section dimensions used in the above calculations are obtained from the "Properties for designing" tables of Reference 14.

The values of d/t_w , $b_f/2t_f$ and l/r for the other frame members are listed in Table 6.

Step 11. Compute the plastic moment capacities of the frame members and reduce these moment capacities for lateral torsional buckling; check whether the reduced bending capacities satisfy Equation (6) or (7), as the case may be.

Girder G1: W12 x 27

$$M_{px} = (0.622 \times 10^{-3})(273 \times 10^3) = 169.9 \text{ kN-m}$$

Assuming the compression flange to be laterally braced by the decking:

$$M_{mx} = M_{px} = 169.9 \text{ kN-m}$$

$$(M_p)_n/M_{mn} = (M_p)_x/M_{mx} \quad [\text{Eq (7)}]$$

$$= 175/169.9 = 1.03 < 1.10$$

Column C1: W14 x 111

$$M_{px} = (3.22 \times 10^{-3})(273 \times 10^3) = 876.1 \text{ kN-m}$$

$$M_{py} = (1.54 \times 10^{-3})(273 \times 10^3) = 420.4 \text{ kN-m}$$

$$r_x = 6.231 \text{ in (0.158 m)}$$

$$r_y = 3.73 \text{ in (0.095 m)}$$

$$l = 5.0 \text{ m}$$

$$\begin{aligned}
 M_{mx} &= [1.07 - l/r_y \sqrt{F_{dy}/262,394}] M_{px} \leq M_{px} \quad [\text{Eq (5a)}] \\
 &= [1.07 - (5.0/0.095) \sqrt{273 \times 10^3}/262,394] 876.1 \\
 &= 845.0 \text{ kN-m}
 \end{aligned}$$

Using Equation (5b) of the text,

$$\text{For } l/r_x = 5.0/0.158 = 31.6:$$

$$M_{my} = M_{px} = 420.4 \text{ kN-m}$$

$$(M_p)_x/M_{mx} = 420/845 = 0.5 < 1.1 \quad [\text{Eq (6a)}]$$

$$(M_p)_y/M_{my} = 304/420 = 0.72 < 1.1 \quad [\text{Eq (6b)}]$$

Similar computations are performed for the other columns. The results of these computations are tabulated in Table 5.

A.3 Modeling of Frame Structures for the DYNFA Program

Problem A.3: Construct the analytical model of an unbraced, interior, rigid frame of a single-story steel building subjected to a normal shock wave; prepare the related input data for DYNFA.

Procedure:

Step 1. Establish the design parameters:

- a. Geometry of frame
- b. Sizes of primary frame members; secondary members, decking and siding
- c. Support conditions of frame
- d. Post-explosion condition of structure
- e. Static tensile stress, F_y
- f. Dynamic increase factor, c
- g. Modulus of elasticity, E

Also calculate the dynamic tensile yield stress,
 $F_{dy} = cF_y$.

These data are available from the preliminary design phase (Problem A.1).

- Step 2. Establish the scope of the model on the basis of the guidelines given in Section 4.2. Sketch a line diagram of the frame to be analyzed; designate nodal points at the appropriate locations on the model (Section 4.2). Assign identification numbers to the nodal points and elements (Section 8.3). Specify which nodal points are designated as mass points.
- Step 3. Assign dynamic degrees of freedom to the mass points of the model using the guidelines given in Section 4.3.
- Step 4. Compute the masses of the individual panels of the walls and roof within the tributary strip supported by the frame (Fig 17 and 19 of text). Also compute the masses of the interior columns and primary transverse members within the tributary strip.
- Step 5. Using Figures 17 and 19 of the text (in conjunction with Figures 18 and 20, respectively), and the masses of the individual panels of the walls and roof, determined in Step 4, compute the concentrated masses assigned to the dynamic degrees of freedom on each panel. Tabulate these masses as shown in Table 7. Include the mass of the interior columns and transverse girders in this tabulation.
- Step 6. Determine the cross-sectional properties and capacities of the frame members.

The cross-sectional properties required for the analysis are:

- a. Area of cross-section, A
- b. Moment of inertia about an axis normal to the plane of the frame, I_x or I_y , as the case may be.

The cross-sectional properties required to compute the member capacities are:

- a. Area of cross-section, A
- b. Radii of gyration, r_x and r_y , about both principal axes of the cross-section
- c. Plastic section modulus, Z_x or Z_y , about an axis normal to the plane of the frame.

The capacities required for the analysis are:

- a. Ultimate dynamic load capacity in axial tension, P_p [Eq (13)]
- b. Ultimate dynamic load capacity in axial compression, P_u [Eq (14)]
- c. Ultimate bending capacity in the absence of axial load, M_{mx} or M_{my} , depending upon the axis of bending [Eq (5a) or 5b)]

Some of these data are available from the preliminary design phase.

Tabulate, as shown in Table 8, the nodal connectivities, cross-sectional properties and the axial load and bending capacities for all elements. Indicate the locations of local pins in this tabulation, if any are utilized in the analysis.

Step 7. Using the guidelines provided in Figures 23 and/or 24 of the text, establish the dimensions of the tributary loading areas that are assigned to the mass points on the exterior members of the frame. Compute the tributary areas and tabulate both the dimensions and the areas as shown in Table 9.

Combine adjacent coplanar areas assigned to the one mass point.

Step 8. Establish the following blast loading parameters:

- a. Location of the charge
- b. Charge weight, W
- c. Safety factor, SF

- d. Normal distance, R_A , from the center of the charge to the blastward and leeward walls of the building

Calculate the effective charge weight,
 $W_E = (1.0 + SF)W$.

Most of these data are available from the preliminary design.

Step 9.

- a. Compute the scaled distances from the center of the charge to the blastward and leeward ends of the building, $Z = R_A/W_E^{1/3}$.
- b. Enter Figure 4-5, 4-11 or 4-12 of Reference 3 or Figure 6 of the text with each of the scaled distances determined above, and read from the appropriate curves:

Peak positive incident pressure, P_{S0}

Scaled unit positive impulse, $i_s/W^{1/3}$

Shock front velocity, U

- c. Enter Figure 4-5 or 4-12 of Reference 3 or Figure 6 of the text with the scaled distance to the blastward wall and read from the appropriate curves:

Peak positive normal reflected pressure, P_r

Scaled unit positive normal reflected impulse, $i_r/W^{1/3}$

- d. Tabulate all values as shown in Table 10.

Step 10.

Using the blast wave parameters determined in Steps 9b and 9c, construct the reflected pressure-time history on the blastward wall:

- a. Calculate clearing time t_c :

$$t_c = 3S/U$$

where S = height of blastward wall or one half its width

- b. Calculate fictitious positive phase duration, t_{of} :

$$t_{of} = 2i_s/P_{s0}$$

- c. Determine peak dynamic pressure, q_0 , from Figure 4-66 of Reference 3 for P_{s0} .
- d. Calculate $P_{s0} + C_D q_0$. Obtain C_D from paragraph 4-14b of Reference 3.
- e. Calculate fictitious reflected pressure duration, t_r :
- $$t_r = 2i_r/P_r$$
- f. Construct the reflected pressure-time curves shown in either (a) or (b), Figure 26 of the text. The curve utilized for the analysis is the one which yields the smallest value of the impulse (area under curve).
- g. Tabulate all values as shown in Table 10.

Step 11. Using the blast wave parameters determined in Step 9b, determine the combined incident/drag pressure-time histories at the blastward and leeward end of the roof as follows:

- a. Calculate fictitious positive phase duration, t_{of} :

$$t_{of} = 2i_s/P_{s0}$$

- b. Determine peak dynamic pressure q_0 from Figure 4-66 of Reference 3 for P_{s0} .
- c. Calculate $P_{s0} + C_D q_0$. Obtain C_D from paragraph 4-14c of Reference 3.
- d. Tabulate all values as shown in Table 10.

Step 12. a. Determine the pressure history at the location of each mass point on the roof by linearly interpolating for both the peak pressure and the duration, using the data computed in Step 11 and Equations (15) and (16).

- b. Determine the pressure history for each mass point on the leeward wall using Equations (17) and (18) (where appropriate - see Section 5.3), and the data computed in Step 11.
- c. In a similar manner, interpolate for the shock front velocities at the mass points on the roof, and extrapolate for these quantities at the mass points on the leeward wall.

Tabulate the peak pressures, durations and shock front velocities as shown in Table 11.

If large pressure differentials occur between the blastward and leeward ends of the roof, a more refined interpolation for the pressure-time histories may be required. Refer to Section 5.3 for guidance, if this is necessary.

Step 13. Determine the values of parameters a and D for each tributary area (Section 5.3). Refer to Figures 30, and 32 through 34 of the text for guidance when computing these quantities. Tabulate all values as shown in Table 11.

- Step 14.
- a. Using the pressure histories and shock front velocities determined in Steps 10 and 12, the values for a and D determined in Step 13, and the equations given in Section 5.3, compute the following blast loading parameters for each tributary area:
 1. Travel time, t_a , computed using Equation (19).
 2. Rise time, t_{rt} , computed using Equation (20).
 3. Average peak pressure, $(P_{pk})_{AVG}$, computed using Equation (21).
 4. Time, t_{pk} , at which $(P_{pk})_{AVG}$ occurs, computed using Equation (23).
 5. Duration of the pressure loading on the tributary area, t_{DI} , computed using Equation (25).
 6. Time at which the pressure on the tributary area decays to zero, t_f , computed using

Equation (26). Tabulate these data as shown in Table 11.

- b. Using the parameters determined above, generate the digitized data defining the pressure-time history input for DYNFA as described in Section 8.9 and illustrated in Figures 60 and 61 of the text. Assign an identification number to each pressure waveform entered in the DYNFA input.

Step 15. Compute the dead, and where appropriate, live loads acting on the frame. Distribute these quantities as uniform and concentrated loads as described in Section 8.10. Tabulate all values as shown in Tables 12 and 13.

Step 16. Compute the integration time interval on the basis of either Equation (34) or (35) (whichever governs). In addition, specify the desired duration of the response on the basis of the sideway natural period of the frame as computed using the data in Table 8 of the text and either of Equations (36) or (37) (as the case may be).

Step 17. Locate the origin of the global system for the model; establish the direction of the global axes, (Section 8.4), and specify the nodal coordinates (Fig 4).

Step 18. Transfer the nodal coordinates, together with the tabulated data (as contained in Tables 7 through 13) to punched cards using the input formats for DYNFA, which are specified in Section 8.13.

Example A.3: Construct the analytical model of an unbraced, interior, rigid frame of a single-story steel building subjected to a normal shock wave, and prepare the related input data for DYNFA.

Required: The analytical model, and all related input data for DYNFA, for the rigid frame designed in Example A.1.

Step 1. From Example A.1, given:

- a. Geometry of interior frame on column line 2 (Fig 1)

b. Sizes of primary frame members:

1. Roof girders: W12 x 40
2. Exterior columns: W14 x 53
3. Interior columns: W14 x 61

Sizes of secondary members, decking and siding shown in Figure 1.

Size of primary transverse girders: W12 x 36

c. Pin-ended column supports

d. Reusable structure

e. $F_y = 248(10)^3$ kPa (36 ksi)

f. $c = 1.1$

g. $E = 207(10)^6$ kPa (30×10^6 psi)

$F_{dy} = 273(10)^3$ kPa (39.6 ksi)

Step 2. The interior frame on column line 2 (Fig 1) is modeled for analysis with DYNFA. The model includes only the steel frame, as the foundation response is assumed to have a negligible impact on the frame's response ($M_{FNDN} > 2M_{STEEL}$). A line diagram of the frame is shown in Figure 4. The following locations on the frame are designated as nodal points:

- a. All column base connections: nodes 1, 10 and 18
- b. All column/girder connections: nodes 5, 9 and 14
- c. Three intermediate locations within the span of each exterior member: nodes 2 through 4, 6 through 8, 11 through 13 and 15 through 17. In each group of three intermediate nodal points, two nodes are located at the connections of the secondary members (purlins and girts) to the frame proper. These nodes are

2, 4, 15 and 17 on the exterior columns; and 6, 8, 11 and 13 on the roof girders. The third node in each group is used for the purpose of computing the maximum chordal angle (Section 4.2) at the midspan of the member.

Nodal points at the following locations are designated as mass points (Fig 4).

- a. All column/girder connections
- b. All locations on exterior members corresponding to the connections of secondary members to the frame proper.

Step 3. Assign dynamic degrees of freedom to the mass points of the model.

Consistent with the guidelines given in Section 4.3, the dynamic degrees of freedom for the model are designated as shown in Figure 4b. Note that the mass points at the girder/column intersections (nodes 5, 9 and 14) are assigned two dynamic degrees of freedom (one in the horizontal direction and the other in the vertical direction), while all intermediate mass points (nodes 2, 4, 6, 8, 11, 13, 15 and 17) are assigned one dynamic degree of freedom which is directed normal to the longitudinal axis of the member.

Step 4. Compute the masses of the individual wall, roof and floor panels (if any) within the tributary strip supported by the frame. Also compute the masses of all interior columns and transverse girders within the strip.

Referring to Figure 1, the frames are equally spaced 6.0 meters apart; therefore, the tributary strip is 6.0 meters wide and includes:

- a. Two exterior wall panels, each of which is 6.0 meters wide by 5.0 meters high. Since there are only two girts within each panel, the mass of these members will not be included as a part of the panel mass (see Fig 17 of text, case 2b). Hence, the mass of each wall panel is taken as the sum of the masses of the following structural components:

1. Siding: Type 2 - 20 x 20

Weight = 4.2 psf (20.5 kg/m²)

Mass = 20.5 x 6.0 x 5.0 = 615 kg

2. Exterior column: W14 x 53 (78.9 kg/m)

Length = 5 meters

Mass = 78.9 x 5.0

= 394.5 kg

3. The mass of a wall panel, M_W , is:

$$M_W = 615 + 394.5 = 1,009.5 \text{ kg}$$

b. Two roof panels, each of which is 6.0 meters wide and 6.0 meters long. As was the case with the wall panels, the mass of the secondary framing of the roof is not included as part of the roof panel mass. Hence, the mass of each roof panel is taken as the sum of the masses of the following structural components (Fig 19 of text, case 2b):

1. Decking: Type 2 - 20 x 20

Weight = 4.2 psf (20.5 kg/m²)

Mass = 20.5 x 6.0 x 6.0 = 738 kg

2. Girder: W12 x 40 (59.4 kg/m)

Length = 6.0 m

Mass = 59.4 x 6.0 = 356.4 kg

3. The mass of a roof panel, M_R , is:

$$M_R = 738.0 + 356.4 = 1,094.4 \text{ kg}$$

c. Two girts on each exterior wall, each of which is a W14 x 26 (38.7 kg/m), 6 meters long.

$$m_g = 38.7 \times 6.0 = 232.2 \text{ kg}$$

- d. Two purlins on each roof panel, each of which is a W12 x 19 (28.2 kg/m), 6 meters long.

$$m_p = 28.2 \times 6.0 = 169.2 \text{ kg}$$

- e. Interior column: W14 x 61 (90.8 kg/m)

$$\text{Length} = 5.0 \text{ m}$$

$$\text{Mass} = 90.8 \times 5.0 = 453.8 \text{ kg}$$

- f. Three transverse girders, each of which is a W12 x 36 (53.6 kg/m), 6 meters long.

$$\text{Mass of one transverse girder} = 53.6 \times 6.0$$

$$= 321.6 \text{ kg}$$

- Step 5. Compute the concentrated masses assigned to the dynamic degrees of freedom in each panel.

Using Figure 17, case 2b, in conjunction with Figure 18, both of the text, the concentrated masses for an exterior wall panel are computed as follows:

$$M_W = 1,009.5; m_g = 232.2 \text{ kg}; n_g = 2$$

$$N_{\text{DOFH}} = 2 \text{ from Figure 4b}$$

Therefore:

$$\begin{aligned} (M_{IH})_W &= M_W / (N_{\text{DOFH}} + 1) + m_g \\ &= 1,009.5 / (2 + 1) + 232.2 = 568.7 \text{ kg} \end{aligned}$$

$$\begin{aligned} (M_{EH})_W &= M_W / [2(N_{\text{DOFH}} + 1)] \\ &= 1,009.5 / [2(2 + 1)] = 168.3 \text{ kg} \end{aligned}$$

$$\begin{aligned} (M_{EV})_W &= [M_W + \sum_1^{n_g} (m_{gi})] / 2 \\ &= [1,009.5 + 2(232.2)] / 2 = 737 \text{ kg} \end{aligned}$$

The mass distribution for the blastward column proceeds as follows: $(M_{TH})_W$ is assigned to the horizontal dynamic degrees of freedom at the intermediate mass points (nodes 2 and 4); and $(M_{EH})_W$ and $(M_{EV})_W$ are assigned to the horizontal and vertical dynamic degrees of freedom at the column/girder connection (node 5). Since the column/foundation connection (node 1) is a support point, the portion of the wall mass that would be concentrated there is discarded. The mass distribution on the leeward column is exactly the same as that on the blastward column.

Using Figure 19, case 2b, in conjunction with Figure 20 (both of the text), the concentrated masses for a roof panel are computed as follows:

$$M_R = 1,094.4 \text{ kg}; m_p = 169.2 \text{ kg}; n_g = 2$$

$$N_{DOFV} = 2 \text{ from Figure 4b}$$

Therefore:

$$\begin{aligned} (M_{EH})_R &= [M_R + \sum_1^{n_p} (m_{pi})]/2 \\ &= [1,094.4 + 2(169.2)]/2 = 716.4 \text{ kg} \end{aligned}$$

$$\begin{aligned} (M_{IV})_R &= M_R/(N_{DOFV} + 1) + m_p \\ &= 1,094.4/(2 + 1) + 169.2 = 534 \text{ kg} \end{aligned}$$

$$\begin{aligned} (M_{EV})_R &= M_R/[2(N_{DOFV} + 1)] \\ &= 1,094.4/[2(2 + 1)] = 182.4 \text{ kg} \end{aligned}$$

The mass distribution for the blastward roof girder proceeds as follows: $(M_{IV})_R$ is assigned to the vertical dynamic degrees of freedom at the intermediate mass points (nodes 6 and 8); and $(M_{EH})_R$ and $(M_{EV})_R$ are assigned to the horizontal and vertical dynamic degrees of freedom at the girder/column connections (nodes 5 and 8). The mass distribution for the leeward girder is exactly the same as that for the blastward girder.

The concentrated masses for the wall and roof panels are tabulated in Table 7. The mass of the interior columns and transverse girders are included in this tabulation. Note that there are several contributory masses assigned to the mass points at the girder/column connections. Here, the total mass is the sum of all the contributory masses from several components (wall panels, roof panels, interior columns, transverse girders).

Step 6. Determine the cross-sectional properties and capacities of the frame members.

Girder: W12 x 40

From Step 1: $F_{dy} = 273 \times 10^3$ kPa

$E = 207 \times 10^6$ kPa

From Example A.1: $M_{mx} = 254$ kN-m

$C_c = 122$

The following cross-sectional properties of the member are obtained from the "Properties for designing" tables of Reference 14.

a. $A = 11.9$ in² (76.1 cm²)

b. $I_x = 310$ in⁴ (12,834 cm⁴)

The following additional data is obtained from the same reference to compute P_u :

$r_x = 5.13$ in (0.130 m)

$r_y = 1.94$ in (0.049 m)

Using Equation (13):

$$\begin{aligned} P_p &= F_{dy}A = (273 \times 10^3)(76.1 \times 10^{-4}) \\ &= 2,078 \text{ kN} \end{aligned}$$

According to Equation (14) of the text:

$$P_u = 1.7F_pA$$

Here, F_a is computed using Equation (4.3) of Reference 1. To use this equation, compute the maximum of either l/r_y or Kl/r_x (Table 4.1, Ref 1).

For buckling about the weak axis, l is measured between points of lateral bracing; i.e., purlin/girder connections:

$$l/r_y = 2.0 / (4.93 \times 10^{-2}) = 40.6$$

For buckling about the strong axis, l is measured between the vertical support points, i.e., girder/column connections. Using a value of 0.75 for the effective length factor, K , as recommended in Section 4.4 of Reference 1:

$$Kl/r_x = (0.75 \times 6) / (13.03 \times 10^{-2}) = 34.6 < 40.6$$

Equation (4.3) of Reference 1:

$$F_a = \frac{[1 - \frac{(Kl/r)^2}{2C_c^2}] F_{dy}}{\frac{5}{3} + \frac{3(Kl/r)}{8C_c} - \frac{(Kl/r)^3}{8C_c^3}}$$

$$Kl/r = 40.6$$

$$F_{dy} = 273 \times 10^3$$

$$C_c = 122$$

$$F_a = \frac{[1 - 40.6^2 / (2 \times 122^2)] (273 \times 10^3)}{5/3 + [3(40.6)/8(122)] - [40.6^3/8(122)^3]}$$

$$= 144.3 \times 10^3 \text{ kPa}$$

$$P_u = 1.7(144.3 \times 10^3)(76.1 \times 10^{-4})$$

$$= 1,867 \text{ kPa}$$

Values of P_p , P_u and M_{mx} for all members are tabulated in Table 8 together with the identification numbers and nodal connectivities for the elements representing them.

Step 7. Establish the dimensions of the tributary areas assigned to the mass points on the exterior members.

Since the secondary members are normal to the frame, the dimensions of the tributary areas for the roof and wall panels are established according to the data given in Figure 24 of the text.

For a roof panel:

From Figure 1: $W_T = 6.0$ m

$$W_1 = W_2 = 3.0$$
 m

$$L = 6.0$$
 m

From Figure 4b: $N_M = 2$

For an intermediate mass point (nodes 6, 8, 11 and 13):

Width of tributary area = $W_T = 6.0$ m

Length of tributary area =

$$L/(N_M + 1) = 6.0/(2 + 1) = 2.0$$
 m

$$\text{Area} = 6.0 \times 2.0 = 12.0$$
 m²

For a mass point at the end of the member (nodes 5, 9 and 14):

Width of tributary area = $W_T = 6.0$ m

Length of tributary area =

$$L/2(N_M + 1) = 6/2(2 + 1) = 1.0$$
 m

$$\text{Area} = 6.0 \times 1.0 = 6.0$$
 m²

The dimensions of the tributary areas for the panels of the blastward and leeward walls are established in a similar manner. The dimensions of each tributary area is shown in Figure 5 and tabulated in Table 9. Note that the two contributory coplanar areas from the blastward and leeward roof panels are combined into one area which is assigned to node point 9.

Step 8. From the preliminary design Example A.1, Step 1:

- a. Location of charge, surface burst
- b. Charge weight: $W = 1,140 \text{ kg (2,500 lb)}$
- c. Safety factor: $SF = 20 \text{ percent}$
- d. Normal distance to blastward wall:
 433 ft (132 m)
- e. Normal distance to leeward wall:
 $433 + 39.4 = 472.4 \text{ ft (144 m)}$
- f. Effective charge weight: $W_E = (1.0 + SF)W$
 $= 1.2(1,140) = 1,360 \text{ kg (3,000 lb)}$

Step 9. $W_E = 3,000 \text{ lb}; W_E^{1/3} = 14.42 \text{ lb}^{1/3}$

Point on blastward wall:

- a. $R_A = 433 \text{ f.}$
 $Z = 433/14.42 = 30 \text{ ft/lb}^{1/3}$
- b. Entering Figure 4-12 with $Z = 30 \text{ ft/lb}^{1/3}$:
 $P_{S0} = 1.65 \text{ psi (11.4 kPa)}$
 $i_s/W^{1/3} = 3.4 \text{ psi-ms/lb}^{1/3}$
 $i_s = (3.4)(14.42) = 49.03 \text{ psi-ms}$
 $U = 1.16 \text{ ft/ms (353.6 m/sec)}$

Steps 9a and 9b are repeated for the point on leeward wall.

- c. Entering Figure 4-12 with $Z = 30 \text{ ft/lb}^{1/3}$:
 $P_r = 3.51 \text{ psi (24.2 kPa)}$
 $i_r/W^{1/3} = 6.9 \text{ psi-ms/lb}^{1/3}$
 $i_r = (6.9)(14.42) = 99.5 \text{ psi-ms}$

d. All values are tabulated in Table 10.

Step 10. Determine the reflected pressure-time history for the blastward wall:

a. $t_c = 3S/U$

$$S = 5.0 \text{ m} < 12.0 \text{ m}/2 \text{ (From Fig 1)}$$

$$U = 353.6 \text{ m/sec (From Table 10)}$$

$$t_c = 3(5.0)/353.6 = 0.042 \text{ sec}$$

b. $t_{of} = 2i_s/P_{s0}$

$$\text{From Table 10: } i_s = 49.03 \text{ psi-ms}$$

$$P_{s0} = 1.65 \text{ psi}$$

$$t_{of} = 2(49.03)/1.65$$

$$= 59.4 \text{ ms} = 0.0594 \text{ sec}$$

c. $q_0 = 0.066 \text{ psi (0.45 kPa)}$ for

$$P_{s0} = 1.65 \text{ psi by extrapolating the data in Figure 4-66 of Reference 3.}$$

d. From paragraph 4-14b of Reference 3,

$$C_D = 1.0 \text{ for the blastward wall.}$$

$$\text{Therefore: } P_{s0} + C_D q_0 = 1.65 + 1.0(0.065)$$

$$= 1.72 \text{ psi}$$

e. $t_r = 2i_r/P_r$

$$i_r = 99.5 \text{ psi-ms}$$

$$P_r = 3.51 \text{ psi}$$

$$t_r = 2(99.5)/3.51 = 56.7 \text{ ms}$$

f. The reflected pressure-time histories shown in Figure 6a are constructed using these data.

Note that the curve shown as a dotted line, which was constructed using P_r and t_r , has a higher impulse than the other curve, which is the one used in the analysis.

g. All values are tabulated in Table 10.

Step 11. Determine the combined incident/drag pressure-time history at the blastward end of the roof:

a. $t_{s0} = 2i_s/P_{s0}$

From Step 10b: $t_{of} = 0.0594$ sec

b. From Step 10c: $q_0 = 0.066$ psi (0.48 kPa)

c. From paragraph 4-14c of Reference 3:

$C_D = -0.40$ for the roof and leeward wall.

Therefore, $P_{s0} + C_D q_0 = 1.65 - 0.40(0.065)$

$= 1.62$ psi (11.1 kPa)

d. Steps 11a through 11c are repeated for the point at the leeward end of the building and all values are tabulated in Table 10.

Step 12. a. Determine the pressure-time histories at the locations of all mass points on the roof using Equations (15) and (16), and the data determined in Step 11.

At the blastward end of the frame:

$(P_{pk})_B = P_{s0} + C_D q_0 = 11.1$ kPa

$(t_{dr})_B = t_{of} = 0.0594$ sec

At the leeward end of the frame:

$(P_{pk})_L = P_{s0} + C_D q_0 = 9.9$ kPa

$(t_{dr})_L = t_{of} = 0.0624$ sec

$L_R = 12.0$ m (Fig 1)

For node point 9 on the roof (Fig 5a):

$$z_{gr} = 6.0 \text{ m}$$

Using Equations (15) and (16):

$$(P_{pk})_g = 11.1 - \frac{(11.1 - 9.9)(6.0)}{12.0} \quad [\text{Eq (15)}]$$

$$= 10.5 \text{ kPa}$$

$$(t_{dr})_g = 0.0594 - \frac{(0.0594 - 0.0624)(6.0)}{12.0} \quad [\text{Eq (16)}]$$

$$= 0.0609 \text{ sec}$$

- b. Determine the pressure-time histories at the locations of all mass points on the leeward wall using Equations (17) and (18) and the data determined in Step 11.

For node point 17 on the leeward wall:

$$z_{17z} = 3.33 \text{ m (Fig 5)}$$

$$(P_{pk})_{17} = 9.9 - \frac{(11.1 - 9.9)(3.33)}{12.0} \quad [\text{Eq (17)}]$$

$$= 9.5 \text{ kPa}$$

$$(t_{dr})_{17} = 0.0624 - \frac{(0.0594 - 0.0624)(3.33)}{12.0} \quad [\text{Eq (18)}]$$

$$= 0.0632 \text{ sec}$$

The blast loading parameters defining the pressure waveforms (P_{pk} , t_{dr}) at all of the mass points are tabulated in Table 11.

- c. The shock front velocity remains constant as the wave traverses the building; therefore, no interpolation or extrapolation for this quantity is required in this problem.

Step 13. Determine the values of a and D for the tributary areas.

Based on the dimensions of the tributary areas shown in Figure 5 and using Figure 32 for guidance, the following values of a and D for node 9 on the roof are determined:

$$D = d_2 \text{ and } a = l_2$$

From Figure 5: $d_2 = 5.0 \text{ m}$ and $l_2 = 2.0 \text{ m}$

Using Figure 33 for guidance, the following values of a and D are determined for node 17 on the leeward wall:

$$D = L + h_1 \text{ and } a = h_2$$

From Figure 5:

$$L = 12.0 \text{ m}$$

$$h_1 = 2.5 \text{ m}; h_2 = 1.67 \text{ m}$$

$$D = 12.0 + 2.5 = 14.5 \text{ m}$$

$$a = 1.67 \text{ m}$$

Values of a and D for the other mass points on the model are tabulated in Table 11.

- Step 14. a. Compute the blast loading parameters for modifying the pressure-time histories at each mass point.

For node point 9:

From Step 12a:

$$P_{pk} = 10.5 \text{ kPa}; t_{dr} = 0.0609 \text{ sec}; U = 353.6 \text{ m/sec}$$

From Step 13:

$$a = 2.0 \text{ m}; D = 5.0 \text{ m}$$

$$1. t_a = D/U_{AVG} = 5.0/353.6 = 0.0141 \text{ sec [Eq (19)]}$$

$$2. t_{rt} = a/U_g = 2.0 / 353.6 = 0.0056 \text{ sec [Eq (20)]}$$

$$3. (P_{pk})_{AVG} = P_{pk} [1 - (a/2Ut_{dr})] \quad \text{[Eq (21)]}$$

$$= 10.5[1.0 - 2.0/(2.0)(353.6)(0.0609)]$$

$$= 10.0 \text{ kPa}$$

$$4. \quad t_{pk} = t_a + t_{rt} \quad [\text{Eq (23)}]$$

$$= 0.0141 + 0.0056 = 0.0197 \text{ sec}$$

$$5. \quad t_{DI} = t_{rt}/2 + t_{dr} \quad [\text{Eq (25)}]$$

$$= 0.0056/2 + 0.0609 = 0.0637 \text{ sec}$$

$$6. \quad t_T = t_a + t_{DI} \quad [\text{Eq (26)}]$$

$$= 0.0141 + 0.0637 = 0.0778 \text{ sec}$$

The values of these parameters for the other mass points are computed in a similar manner and all results are tabulated in Table 11.

- b. Generate the digitized data defining the pressure-time history input for DYNFA as described in Section 8.9 and illustrated in Figures 60 and 61 of the text.

For node point 9:

Point	Pressure		Time	
	Parameter	Value (kPa)	Parameter	Value (sec)
1	-	0.0	-	0.000
2	-	0.0	t_a	0.0141
3	$(P_{pk})_{AVG}$	10.0	t_{pk}	0.0197
4	-	0.0	t_T	0.0778
5	-	0.0	T_F	1.000

Waveform identification number = 7 (from Table 11).

Number of points in waveform = 5 (from above).

The digitized data defining the pressure-time histories for the other mass points are given in Appendix B.

Step 15. Compute the dead loads acting on the frame and distribute these loads among the elements and nodal points of the model. In DYNFA, the dead loads are entered in mass units when SI units are utilized in the analysis and the conversion of mass to force is performed by the program. Therefore, in the computations that follow, the dead loads are given in kilograms.

a. Distribution of the weight of a roof panel:

1. The uniform load acting on the girder consists of its own weight:

$$w = 59.4 \text{ kg/m (from Step 4).}$$

This loading is applied to elements 5 through 8, and 10 through 13.

2. The concentrated loads acting along the girder consists of the reactions of the purlins. Each purlin supports its own weight as well as one third the weight of the decking within each roof panel.

From Step 4:

$$\text{Weight of decking} = 738 \text{ kg}$$

$$\text{Weight of one purlin} = 169.2 \text{ kg}$$

$$\text{Concentrated load acting along girder (nodes 6, 8, 11 and 13)} = (1/3)(738) + 169.2$$

$$= 415.2 \text{ kg}$$

The remaining one third of the decking weight is supported by the transverse girders, and therefore is applied as a concentrated load at each girder/column connection.

b. Distribution of the weight of a wall panel:

1. The distributed load acting on the column is its own weight:

$$w = 78.9 \text{ kg/m}$$

This loading is applied to elements 1 through 4, and 14 through 17.

2. One half the weight of the wall (including girts) is applied as a concentrated load at the exterior column/girder connection.

From Step 4:

$$\text{Weight of siding} = 615 \text{ kg}$$

$$\begin{array}{r} \text{Weight of 2 girts} \\ (232.2 \text{ kg/girt}) \end{array} = \frac{464.4 \text{ kg}}{1,079.4 \text{ kg}}$$

Concentrated load acting at each exterior column/girder connection (nodes 5 and 14):

$$(1/2)(1,079.4) = 539.7 \text{ kg}$$

- c. The weight of the transverse girders is applied directly to the girder/column connections (nodes 5, 9 and 14).
- d. The weight of the interior column (90.8 kg/m - from Step 4) is applied as a uniformly distributed load to element no. 9.

The dead loads applied to the elements are tabulated in Table 12; concentrated loads applied to the appropriate nodal points are tabulated in Table 13.

Step 16. Compute the integration time interval and specify the duration of the response.

Integration time interval:

Check the extensional modes of the members:

$$T_a = 2\pi\sqrt{LM_e/AE} \quad \text{[[Eq 34]]}$$

Exterior column: L = 5.0 m (Fig 1)

$$M_e = 1,241.0 \text{ kg (Table 7)}$$

$$= 1.241 \text{ kN-sec}^2/\text{m}$$

$$A = 101.0 \text{ cm}^2 \text{ (Table 8)}$$

$$E = 207 \times 10^6 \text{ kPa (Step 1)}$$

$$T_a = 2\pi\sqrt{5(1.241)/(101 \times 10^{-4})(207 \times 10^6)}$$

$$= 0.011 \text{ sec}$$

Interior column: L = 5.0 m (Fig 1)

$$M_e = 913.3 \text{ kg (Table 7)}$$

$$= 0.9133 \text{ kN-sec}^2/\text{m}$$

$$A = 115.0 \text{ cm}^2 \text{ (Table 8)}$$

$$E = 207 \times 10^6 \text{ kPa}$$

$$T_a = 2\pi\sqrt{5(0.9133)/(115 \times 10^{-4})(207 \times 10^6)}$$

$$= 0.0087 \text{ sec}$$

Roof girder: L = 6.0 m (Fig 1)

$$M_e = 1,206.3 \text{ kg (Table 7)}$$

$$= 1.2063 \text{ kN-sec}^2/\text{m}$$

$$A = 76.1 \text{ cm}^2 \text{ (Table 8)}$$

$$E = 207 \times 10^6 \text{ kPa}$$

$$T_a = 2\pi\sqrt{6.0(1.2063)/(76.1 \times 10^{-4})(207 \times 10^6)}$$

$$= 0.0135 \text{ sec}$$

Check the second bending mode of vibration of the exterior column and the roof girder:

$$T_b = (1/C)\sqrt{M_1 L^3/EI} \quad \text{[[Eq (35)]]}$$

Exterior column: $C = 7.95$ (Table 7 of text)

$$\begin{aligned}M_i &= 2 \times 568.7 \\ &= 1,137.4 \text{ kg (Table 7)} \\ &= 1.1374 \text{ kN-sec}^2/\text{m}\end{aligned}$$

$$I = 22,550 \text{ cm}^4 \text{ (Table 8)}$$

$$T_b = \sqrt{\frac{(1.1374)(5.0)^3 / (207 \times 10^6)(22,550 \times 10^{-8})}{7.95}}$$

$$= 0.0069 \text{ sec}$$

Roof girder: $C = 9.82$ (Table 7 of text)

$$\begin{aligned}M_i &= 2 \times 534 = 1,068 \text{ kg (Table 7)} \\ &= 1.068 \text{ kN-sec}^2/\text{m}\end{aligned}$$

$$I = 12,834 \text{ (Table 8)}$$

$$T_b = \sqrt{\frac{(1.068)(6.0)^3 / (207 \times 10^6)(12,834 \times 10^{-8})}{9.82}}$$

$$= 0.0095 \text{ sec}$$

Inspection of these results indicates that the second bending mode of vibration of the exterior column has the shortest period (highest frequency); hence, the integration time increment used is $1/20$ of this period:

$$\Delta t = 0.0069/20 = 0.000345 \text{ sec}$$

$$\text{Use } \Delta t = 0.00035 \text{ sec}$$

Duration of response and number of integration time increments:

Compute sidesway natural period of frame:

$$T_s = 2\pi\sqrt{m_e / KK_L} \quad [\text{Eq (36)}]$$

Here, m_e is the total mass of the roof panels, purlins and transverse girders plus one third the mass of the wall panels, girts and interior columns:

From Step 4:

Mass of 2 roof panels = 2,188.8 kg
(1,094.4 kg/panel)

Mass of 4 purlins = 676.8 kg
(169.2 kg/purlin)

Mass of 3 transverse girders = 964.8 kg
(321.6 kg/girder)

Total = 3,830.4 kg

Mass of 2 wall panels = 2,019.0 kg
(1,009.5 kg/panel)

Mass of 4 girts = 928.8 kg
(232.2 kg/girt)

Mass of interior column = 453.8 kg

Total = 3,401.6 kg

$$m_e = 3,830.4 + (1/3)(3,401.6) = 4,964.3 \text{ kg}$$

$$= 4.964 \text{ kN-sec}^2/\text{m}$$

$$K = (EI_{ca} C_2 / H^3) [1 + (0.7 - 0.1\beta)(n - 1)]$$

(Table 8 of text)

$$E = 207 \times 10^6 \text{ kPa}$$

$$I_{ca} = \Sigma I_c / (n + 1) \quad (\text{Table 8 of text})$$

From Table 8 of Appendix A:

$$I \text{ for exterior column} = 22,550 \text{ cm}^4$$

$$I \text{ for interior column} = 26,670 \text{ cm}^4$$

$$I_{ca} = \frac{2(22,550) + 26,670}{3} = 23,923 \text{ cm}^4$$

$$H = 5.0 \text{ m (Fig 1)}$$

$$\beta = 0.0 \text{ (Table 8 of text)}$$

C_2 is a function of D and β :

$$D = (I_g/L)/[I_{ca}(0.75 + 0.25 \beta)/H] \quad (\text{Table 8 of text})$$

From Table 8 of Appendix A: $I_g = 12,834 \text{ cm}^4$

$$D = \frac{12,834/6.0}{23,923(0.75)/5.0} = 0.6$$

For $D = 0.6$ and $\beta = 0.0$: $C_2 = 4.93$

therefore:

$$K = \frac{(207 \times 10^6)(23,923 \times 10^{-8})(4.93)[1 + 0.7(2 - 1)]}{5^3}$$
$$= 3,320 \text{ kN/m}$$

$$K_L = 0.55(1 - 0.25\beta); \beta = 0$$
$$= 0.55$$

$$T_S = 2\pi\sqrt{4.964/[3,320(0.55)]}$$
$$= 0.33 \text{ sec}$$

Here, the response will be computed for a duration, T_f , of $(1/2)T_S$; therefore, the number of integration time increments, NDT, is:

$$\text{NDT} = 1 + T_f/\Delta t \quad [\text{Eq (39)}]$$

$$T_f = (1/2)T_S = 0.165 \text{ sec}$$

$$\Delta t = 0.00035 \text{ sec}$$

$$\text{NDT} = 1 + 0.165/0.00035 = 471$$

Use 500 increments in the analysis.

Step 17. The origin of the global coordinate system is located at nodal point 1 (Fig 4), which is the lower left-hand support point of the model. The nodal coordinates are specified on the sketch of the model (Fig 4a).

Step 18. The nodal coordinates together with the data contained in Tables 7 through 13 are punched on input data cards according to the format specifications in Section 8.13. A listing of the input data deck for this problem is given in Appendix B. Note that the blast loads on the leeward walls and roof act in the negative directions of the global X and Y axes, respectively. Therefore, the tributary areas on these surfaces are entered as negative quantities in the input to DYNFA. In addition, the dead loads act in the negative direction of the global Y axis. Therefore, in the DYNFA input, -Y is entered as the direction of the element uniform loads and the concentrated nodal loads in Table 13 are entered as negative quantities.

Problem A.4: Construct the analytical models of several representative frames, spanning in each direction, of a single-story rigid frame steel building subjected to a quartering shock wave; prepare all related input data for DYNFA.

Procedure:

- Step 1. Construct the analytical model of each frame (selected for analysis with DYNFA) as outlined through Step 7. in Steps 1 through 3 of Problem A.3. Compute the following data for each model as instructed in Steps 4 through 7 of the same problem:
- a. Masses of the individual panels of the walls and roof supported by frame.
 - b. Concentrated masses assigned to the dynamic degrees of freedom.
 - c. Cross-sectional properties and capacities of frame members; prepare a tabulation containing the element nodal connectivities, and their cross-sectional properties and capacities.
 - d. Dimensions of tributary loading areas assigned to the mass points on the exterior members.

Step 8. Establish the following blast loading parameters:

- a. Location of charge

- b. Charge weight, W
- c. Safety factor, SF ; calculate effective charge weight, $W_E = (1.0 + SF)W$.
- d. Angles of incidence, α_A and α_B , between the shock front and each blastward wall at location of frames.
- e. Radial distances from the center of the charge to the blastward and leeward ends of each frame selected for analysis.

Some of these data are available from the preliminary design.

- Step 9.
- a. Compute the scaled distances from the center of the charge to the blastward and leeward ends of each frame, $Z = R/W_E^{1/3}$.
 - b. Enter Figure 4-5, 4-11 or 4-12 of Reference 3 or Figure 6 of the text, with each of the scaled distances determined above and read from the appropriate curves:

Peak positive incident pressure, P_{S0}

Scaled unit positive impulse, $i_s/W^{1/3}$

Shock front velocity, U

- Step 10. Using the blast loading parameters determined in Steps 8 and 9, construct the reflected pressure-time histories at the location of each frame:

- a. Read the reflected pressure coefficient, $C_{r\alpha}$, from Figure 4-6 of Reference 3 for P_{S0} determined in Step 9b and α determined in Step 8d.
- b. Calculate $P_{r\alpha} = C_{r\alpha}P_{S0}$.
- c. Calculate clearing time t_c :

$$t_c = 3S/U$$

where S , in cases involving quartering loads, equals the minimum of:

1. Height of blastward wall
 2. Horizontal distance from the frame to the leeward end of the wall
 3. Sum of the horizontal distance from the frame to the blastward end of the wall, plus the length of the adjacent blastward wall.
- d. Calculate fictitious positive phase duration, t_{of} :

$$t_{of} = 2i_s/P_{s0}.$$
 - e. Determine peak dynamic pressure, q_0 , from Figure 4-66 of Reference 3 for P_{s0} . Calculate $P_{s0} + C_D q_0$. Obtain C_D from paragraph 4-14b of Reference 3.
 - f. Enter Figure 4-5 or 4-12 of Reference 3 or Figure 6 of the text, and read $i_r/W^{1/3}$ for $P_{r\alpha}$ determined in 10b.
 - g. Calculate fictitious reflected pressure duration, t_r :

$$t_r = 2i_r/P_{r\alpha}.$$
 - h. Construct the reflected pressure curves shown in either (a) or (b), Figure 26 of the text. The curve utilized for the analysis is the one which yields the smallest value of the impulse (area under curve).
 - i. Tabulate all values as shown in Table 17.

Step 11. Using the blast wave parameters determined in Step 9b, construct the combined incident/drag pressure-time histories at the blastward and leeward walls of each frame as instructed in Step 11 of Problem A.3. Tabulate all values as shown in Table 17.

- Step 12. Determine the pressure-time histories and shock front velocities at all mass points on the roof and leeward walls of each frame as instructed in Step 12 of Problem A.3. Tabulate all values as shown in Tables 18 and 19.
- Step 13. Determine the values of parameters a and D (Section 5.3) for the tributary areas assigned to the mass points of each model. Refer to Figures 31, 32, 33 and 35 of the text for guidance in accomplishing this task. Tabulate all values as shown in Tables 18 and 19.
- Step 14. Modify the pressure-time histories determined in Steps 10 and 12, and prepare the digitized input of pressure versus time for DYNFA as instructed in Step 14 of Problem A.3. Tabulate the parameters t_a , t_{rt} , $(P_{pk})_{AVG}$, t_{pk} , P_s , t_c , t_{DI} and t_T for all loaded mass points of each frame as shown in Tables 18 and 19.
- Step 15. Compute and distribute the dead and live loads (where appropriate) acting on each frame as instructed in Section 8.10. Tabulate as shown in Tables 12, 13, 20 and 21.
- Step 16. Compute Δt , NDT and NSKIP (Section 8.8) for the DYNFA analysis of each frame as instructed in Step 16 of Problem A.3.
- Step 17. For each model, locate the origin of the global coordinate system, establish the directions of the global axes and specify nodal coordinates.
- Step 18. Prepare input data decks for each model as instructed in Step 18 of Problem A.3. Enter fictitiously high values for the bending capacities of elements representing members or portions of members designed for bi-axial bending (Section 7.3).

Example A.4: Construct the analytical models of several representative frames, spanning in each direction, of a single-story rigid frame steel building subjected to a quartering shock wave; prepare all related input data for DYNFA.

Required: The analytical models and all related input data for the frames on column lines 1, 2, A and B of the building shown in Figure 1.

Step 1. From Example A.2, given:

a. Geometry of frames on column lines 1, 2, A and B, Figure 1.

b. Sizes of primary frame members given in Table 5.

Sizes of secondary members, decking and siding shown in Figure 1.

c. Pin-ended column supports all frames.

d. Reusable structure

e. $F_y = 248(10)^3$ kPa (36 ksi)

f. $c = 1.1$

g. $E = 207(10)^6$ kPa (30×10^6) psi

$F_{dy} = 273(10)^3$ kPa

Step 2. The frames on column lines 1, 2, A and B are modeled for analysis with DYNFA. Each model includes the members of the steel frame only. The foundation is not included as its response is assumed to have a negligible impact on the frame response ($M_{FNDN} \geq 2M_{STEEL}$). This presentation is directed towards illustrating the modeling process and data preparation for the frames on column lines B and 2. Construction of the models and preparation of the input data for the other two frames proceed in exactly the same manner and, therefore, their illustration here would be superfluous.

Model of frame on column line B:

A line diagram of the frame is shown in Figure 7. The following locations on the frame are designated as nodal points:

- a. All column base connections: nodes 1, 10, 15 and 23
- b. All column/girder connections: nodes 5, 9, 14 and 19
- c. Three intermediate points within the span of each roof girder: nodes 6 through 8, 11 through 13 and 16 through 18. In each group of three intermediate nodal points, one node is located at the midspan of the member and each of the remaining two is located a distance $L/4$ from its end, where L is the length of the member.
- d. Three intermediate points within the span of each exterior column: nodes 2 through 4 and 20 through 22. In each such group of three intermediate nodal points, one node is located at the midspan of the member (for the purpose of measuring the member's response at midspan) and the other two are located at the connections of the girts to the frame.

Nodal points at the following locations are designated as mass points:

- a. All column/girder connections
- b. All intermediate locations on roof girders
- c. All connections of girts to exterior columns.

Model of frame on column line 2:

A line diagram of the frame is shown in Figure 4. The placement of nodal points and designation of mass points for this model is given in Step 2 of Example A.3.

Step 3. Assign dynamic degrees of freedom to the mass points of each model.

Model of frame on column line B:

The dynamic degrees of freedom for the model of the frame on column line B are shown in Figure 7b.

Model of frame on column line 2:

The dynamic degrees of freedom for the model of the frame on column line 2 are shown in Figure 4b.

- Step 4. Compute the masses of the wall and roof panels, and interior columns within the tributary strip supported by each frame.

Model of frame on column line B:

- a. External wall panel: 6.0 meters wide by 5.0 meters high; secondary framing corresponds to that shown in Figure 18b of the text, number of girts, $n_g = 2$; therefore, the mass of the panel, M_W , consists of the mass of the following (Fig 17 of text, case 2b):
1. Siding: Type 2 - 20 x 20 (Figure 1)
Weight = 4.2 psf (20.5 kg/m²)
Mass = 20.5 x 6.0 x 5.0 = 615.0 kg
 2. Exterior column: W14 x 78 (116.1 kg/m)
(Table 5)
Length = 5 meters
Mass = 116.1 x 5 = 580.5 kg
 3. $M_W = 615.0 + 580.5 = 1,195.5$ kg
- b. Girts on exterior wall: W14 x 26 (38.7 kg/m)
(Fig 1)
Length = 6.0 meters
Mass, $m_g = 38.7 \times 6.0$
= 232.2 kg
- c. Roof panel: 6.0 meters wide by 6.0 meters long; secondary framing corresponds to that shown in Figure 20a of the text; therefore, the mass of the panel, M_R , consists of the mass of the following (Fig 19 of text, case 1):

1. Decking: Type 2 - 20 x 20 (Fig 1)
 Weight = 4.2 psf (20.5 kg/m²)
 Mass = 20.5 x 6.0 x 6.0 = 738.0 kg
 2. Girder: W12 x 27 (40.2 kg/m) (Table 5)
 Length = 6.0 meters
 Mass = 40.2 x 6.0 = 240.9 kg
 3. Two purlins: W12 x 19 (28.3 kg/m) (Fig 1)
 Length = 6.0 meters
 Mass = 2 x 28.3 x 6.0 = 339.6 kg
 4. $M_R = 738.0 + 240.9 + 339.6 = 1,318.5$ kg
- d. Mass within panel area $(W_1' + W_2') \times L$ on roof:
- $W_1' = W_2' = 2.0$ meters (Fig 1)
- $L = 6.0$ meters (Fig 1)
- Mass of girder = 240.9 kg (from above)
- One half mass of decking within $(W_1' + W_2') \times L =$
- $(1/2)(20.5 \times 4.0 \times 6.0) = 246.0$ kg
- $M_R' = 246.0 + 240.9 = 486.9$ kg
- (Fig 19 of text, case 1)
- e. Interior column: W14 x 111 (165.2 kg/m) (Table 5)
 Length = 5.0 meters
 Mass = 165.2 x 5.0 = 826.0 kg
 - f. Exterior transverse girders - wall B and opposite wall:

1. Frame girder - W12 x 22 (32.7 kg/m) (Table 5)

Length = 6.0 meters

Mass = $32.7 \times 6.0 = 196.2$ kg

2. Wall support - W8 x 10 (14.9 kg/m) (Fig 3)

Length = 6.0 meters

Mass = $14.9 \times 6.0 = 89.4$ kg

Total = $196.2 + 89.4 = 285.6$ kg

g. Interior transverse girder: W12 x 36 (53.6 kg/m)

Length = 6.0 meters

Mass = $53.6 \times 6.0 = 321.6$ kg

Model of frame on column line 2:

The mass of the wall and roof panels, girts, purlins, transverse girders and the interior column are computed as shown in Step 4 of Example A.3. It should be noted, however, that the mass quantities for this problem will be greater than those in Example A.3 because of the larger columns used.

Step 5. Compute the concentrated masses assigned to the dynamic degrees of freedom of the wall and roof panels of each model.

Frame on column line B:

a. Exterior wall panel: Figure 17 of text, case 2b

$M_W = 1,195.5$ kg (Step 4a)

$m_g = 232.2$ kg (Step 4b)

$n_g = 2$ (Fig 1)

$N_{DOFH} = 2$ (Fig 7b)

$$\begin{aligned}(M_{IH})_W &= M_W / (N_{DOFH} + 1) + m_g \\ &= 1,195.5 / (2 + 1) + 232.2 = 630.7 \text{ kg}\end{aligned}$$

$$(M_{EH})_W = M_W / [2(N_{DOFH} + 1)] = 1,195.5 / [2(2 + 1)]$$

$$= 199.3 \text{ kg}$$

$$(M_{EV})_W = (M_W + \sum_1^{ng} m_g) / 2 = [1,195.5 + 2(232.2)] / 2$$

$$= 830.0 \text{ kg}$$

b. Roof panel: Figure 19 of text, case 1

$$M_R = 1,318.5 \text{ kg} \quad (\text{Step 4c})$$

$$M_R^I = 486.9 \text{ kg} \quad (\text{Step 4d})$$

$$N_{DOFV} = 3 \quad (\text{Fig 7b})$$

$$(M_{EH})_R = M_R / 2 = 1,318.5 / 2 = 659.3 \text{ kg}$$

$$(M_{IV})_R = M_R^I / (N_{DOFV} + 1) = 486.9 / (3 + 1)$$

$$= 121.7 \text{ kg}$$

$$(M_{EV})_R = (M_R - N_{DOFV} M_{IV}) / 2 = [1,318.2 - 3(121.7)] / 2$$

$$= 476.6 \text{ kg}$$

All concentrated masses for the model are tabulated in Table 14.

Frame on column line 2:

The concentrated masses assigned to the dynamic degrees of freedom on the wall and roof panels are computed as shown in Step 5 of Example A.3. A tabulation of all masses for the model is prepared as shown in Table 7.

Step 6. Determine the cross-sectional properties and capacities of the members of each frame.

Frame on column line B:

From Step 1:

$$F_{dy} = 273 \times 10^3 \text{ kPa}$$

$$E = 207 \times 10^6 \text{ kPa}$$

a. Girder G1: W12 x 27 (Table 5)

From Example A.2

$$M_{mx} = 170.0 \text{ kN-m (Table 5)}$$

$$C_c = 122 \text{ (Step 11)}$$

From "Properties for designing" tables,
Reference 14:

1. $A = 7.95 \text{ in}^2 (51.2 \text{ cm}^2)$

2. $I_x = 204.0 \text{ in}^4 (8,491 \text{ cm}^4)$

3. $r_x = 5.07 \text{ in (0.129 m)}$

4. $r_y = 1.52 \text{ in (0.039 m)}$

$$P_p = F_{dy}A = (273 \times 10^3)(51.2 \times 10^{-4}) \quad [\text{Eq (13)}]$$
$$= 1,398.0 \text{ kN}$$

$$P_u = 1.7 F_a A \quad [\text{Eq (14)}]$$

$$F_a = \frac{[1 - \frac{(Kl/r)^2}{2C_c^2}]F_{dy}}{\frac{5}{3} + \frac{3(Kl/r)}{8C_c} - \frac{(Kl/r)^3}{8C_c^3}} \quad [\text{Eq (4.3), Ref 1}]$$

The member is considered to be braced against buckling around the weak axis; hence, F_a is computed for buckling about the strong axis only.

$$l = 6.0 \text{ meters (Fig 1)}$$

$$r_x = 0.129 \text{ meter (from above)}$$

$$K = 0.75 \text{ (Section 4.4, Ref 1)}$$

$$C_c = 122 \text{ (from Step 1)}$$

$$Kl/r_x = 0.75 \times 6.0/0.129 = 34.9$$

$$F_a = \frac{[1.0 - (34.9)^2/(2 \times 122^2)](273 \times 10^3)}{5 + [3(34.9)/8(122)] - [(34.9)^3/8(122)^3]}$$
$$= 147.8 \times 10^3 \text{ kPa}$$

$$P_u = 1.7(147.8 \times 10^3)(51.2 \times 10^{-4})$$
$$= 1,286.5 \text{ kN}$$

b. Column C1: W14 x 111 (Table 5)

From Example A.2:

$$M_{my} = 421.9 \text{ kN-m (Note member bends about y-axis, Fig 3)}$$

$$C_c = 122$$

From "Properties for designing" tables,
Reference 14:

1. $A = 32.7 \text{ in}^2 (211.0 \text{ cm}^2)$

2. $I_y = 455.0 \text{ in}^4 (18,938.0 \text{ cm}^4)$

3. $r_x = 6.23 \text{ in } (0.158 \text{ m})$

4. $r_y = 3.73 \text{ in } (0.095 \text{ m})$

$$P_p = F_{dy}A = (273 \times 10^3)(211.0 \times 10^{-4}) \quad [\text{Eq (13)}]$$
$$= 5,760 \text{ kN}$$

$$P_u = 1.7 F_a A \quad [\text{Eq (14)}]$$

According to Section 4.3 of Reference 1, F_a is computed for the maximum of Kl/r_x or Kl/r_y .

Use: $K = 1.5$ (from Section 4.4, Ref 1)

$$L = 5.0 \text{ m (from Fig 1)}$$

By inspection, Kl/r_y will be the larger of the two ratios:

$$Kl/r_y = (1.5 \times 5.0)/0.095 = 78.9$$

$$\begin{aligned}
 F_a &= \frac{[1.0 - (78.9)^2/(2 \times 122^2)](273 \times 10^3)}{5 + [3(78.9)/8(122)] - [(78.9)^3/8(122)^3]} \\
 &= 115.1 \times 10^3 \text{ kPa} \\
 P_u &= 1.7(115.1 \times 10^3)(211.0 \times 10^{-4}) \\
 &= 4,128.6 \text{ kN}
 \end{aligned}$$

The values of P_p , P_u and M_m for all members of the frame on column line B are listed in Table 15 together with the identification numbers and nodal connectivities for the elements representing them.

Frame on column line 2:

The element properties and capacities for this frame are computed and tabulated in a similar manner. The tabulation is not shown for brevity.

Step 7. Establish the dimensions of the tributary areas assigned to the mass points on the exterior members.

Frame on column line B:

Roof Panel: Use Figure 23 of the text to determine dimensions of tributary areas.

From Figure 1: $W_T = 6.0$ meters

$$W_1 = W_2 = 3.0 \text{ meters}$$

$$W_1' = W_2' = 2.0 \text{ meters}$$

$$L = 6.0 \text{ meters}$$

From Figure 7b: $N_M = 3$

Dimensions of areas for intermediate mass points (nodes 6 through 8, 11 through 13 and 16 through 18) on panel.

From Figure 23 of text:

$$\begin{aligned}\text{Width of tributary area} &= (W_1' + W_2')/2 \\ &= (2.0 + 2.0)/2 = 2.0 \text{ m}\end{aligned}$$

$$\begin{aligned}\text{Length of tributary area} &= L/(N_M + 1) \\ &= 6.0/(3 + 1) = 1.5 \text{ m}\end{aligned}$$

$$\text{Area} = 2.0 \times 1.5 = 3.0 \text{ m}^2$$

The dimensions of the areas for mass points at the ends of the panel are determined as shown in Figure 8.

$$\begin{aligned}\text{From Figure 8: Area} &= (6.0 \times 3.0) - (2.0 \times 2.25) \\ &= 13.5 \text{ m}^2\end{aligned}$$

The dimensions of the tributary areas for all mass points on the blastward wall and roof are shown in Figure 9. The dimensions of the tributary areas on the leeward walls are the same as those on the blastward wall. All dimensions and areas are tabulated in Table 16.

Frame on column line 2:

The tributary areas for the mass points are shown in Figure 5 and tabulated in Table 9.

Step 8. From Example A.2:

a. Location of charge: surface burst; from Figure 2:

$$H_A = 114.3 \text{ m}$$

$$H_B = 66.0 \text{ m}$$

b. Charge weight: $W = 1,140 \text{ kg (2,500 lb)}$

c. Safety factor: 20%

d. Angle of incidence between shock front (Fig 2) and:

$$1. \text{ Wall A: } (\alpha_A)_2 = \tan^{-1}[(66.0 + 6.0)/114.3] \\ = 32.2^\circ$$

$$2. \text{ Wall B: } (\alpha_B)_B = \tan^{-1}[(114.3 + 6.0)/66.0] \\ = 61.2^\circ$$

$$\text{Effective charge weight: } W_E = (1.0 + SF)W \\ = 1.2(1,140) \\ = 1,350 \text{ kg (3,000 lb)}$$

e. Referring to Figure 2, radial distances from center of charge to:

1. Blastward end of column line B:

$$R = \sqrt{(114.3 + 6.0)^2 + (66.0)^2} \\ = 137.2 \text{ m (450.1 ft)}$$

2. Leeward end of column line B:

$$R = \sqrt{(114.3 + 6.0)^2 + [66.0 + 3(6.0)]^2} \\ = 146.7 \text{ m (481.0 ft)}$$

3. Blastward end of column line 2:

$$R = \sqrt{(114.3)^2 + (66.0 + 6.0)^2} \\ = 135.1 \text{ m (443.2 ft)}$$

4. Leeward end of column line 2:

$$R = \sqrt{[114.3 + 2(6.0)]^2 + (66.0 + 6.0)^2} \\ = 145.4 \text{ m (477 ft)}$$

Step 9. Compute Z , P_{SO} , i_S and U at the blastward and leeward ends of each frame.

Point at blastward end of column line B:

a. Compute Z.

$$R = 450.1 \text{ ft (Step 8e)}$$

$$Z = R/W_E^{1/3} = 450.1/(3,000)^{1/3} = 31.2 \text{ ft/lb}^{1/3}$$

b. Entering Figure 4-12 of Reference 3 with
 $Z = 31.2 \text{ ft/lb}^{1/3}$:

$$P_{SO} = 1.56 \text{ psi (10.75 kPa)}$$

$$i_s/W^{1/3} = 3.25 \text{ psi-ms/lb}^{1/3}$$

$$i_s = (3.25)(3,000)^{1/3} = 46.9 \text{ psi-ms}$$

$$U = 1.16 \text{ ft/ms} = 353.6 \text{ m/sec}$$

Repeat steps a and b above for point at leeward end of column line B and tabulate all values in Table 17.

Point at blastward end of column line 2:

a. Compute Z.

$$R = 443.2 \text{ ft (Step 8e)}$$

$$Z = R/W_E^{1/3} = 443.2/(3,000)^{1/3} = 30.7 \text{ ft/lb}^{1/3}$$

b. Entering Figure 4-12 of Reference 3 with
 $Z = 30.7 \text{ ft/lb}^{1/3}$:

$$P_{SO} = 1.60 \text{ psi (11.0 kPa)}$$

$$i_s/W^{1/3} = 3.3 \text{ psi-ms/lb}^{1/3}$$

$$i_s = (3.3)(3,000)^{1/3} = 47.6 \text{ psi-ms}$$

$$U = 1.16 \text{ ft/ms} = 353.6 \text{ m/sec}$$

Repeat steps a and b above for point at leeward end of column line 2 and tabulate all values in Table 17.

Step 10. Construct the reflected pressure-time histories at the location of each frame.

Frame on column line B:

- a. Enter Figure 4-6 of Reference 3 with $P_{S0} = 1.56$ psi and $(\alpha_B)_B = 61.2^\circ$.

Note: Lowest value of P_{S0} for which data are provided is 3.71 psi; hence, read value of $C_{r\alpha}$ for $(\alpha_B)_B = 61.2^\circ$ and $P_{S0} = 3.71$ psi.

$$C_{r\alpha} = 2.40$$

- b. $P_{r\alpha} = C_{r\alpha}P_{S0}$
 $= 2.40 (1.56) = 3.74$ psi (25.8 kPa)

- c. Calculate t_c :

$$t_c = 3S/U$$

where S is the minimum of:

1. Height of building: 5.0 meters
2. Distance to leeward end of wall B: 6.0 meters
3. Distance to blastward end of wall B plus length of blastward wall A:

$$6.0 \text{ m} + 18.0 \text{ m} = 24.0 \text{ meters}$$

$$S = 5.0 \text{ meters}$$

$$t_c = 3(5.0)/353.6 = 0.0424 \text{ second}$$

- d. Calculate t_{of} :

$$t_{of} = 2i_s/P_{S0} = 2(46.9)/1.56$$

$$= 60.1 \text{ ms} = 0.0601 \text{ second}$$

- e. Determine q_0 from Figure 4-66 of Reference 3 for $P_{S0} = 1.56$ psi (extrapolate data in figure):

$$q_0 = 0.056 \text{ psi (0.39 kPa)}$$

$$C_D = 1.0 \text{ (from paragraph 4-14b, Ref 3)}$$

$$P_{S0} + C_D q_0 = 10.75 + 0.39 = 11.14 \text{ kPa}$$

- f. Read $i_r/W^{1/3}$ in Figure 4-12 of Reference 3 for $P_{r\alpha} = 3.74$ psi:

$$i_r/W^{1/3} = 7.2 \text{ psi-ms/lb}^{1/3}$$

$$i_r = 7.2(3,000)^{1/3} = 103.8 \text{ psi-ms}$$

- g. Calculate t_r :

$$t_r = 2i_r/P_{r\alpha} = 2(103.8)/3.74$$

$$= 55.5 \text{ ms} = 0.0555 \text{ sec}$$

- h. The reflected pressure-time histories shown in Figure 10a are constructed using this data. The dotted pressure-time history, constructed using $P_{r\alpha}$ and t_r , has a higher impulse than the other curve, which is the one used for the analysis.

- i. Tabulate all values in Table 17.

Frame on column line 2:

Repeat Steps 10a through 10h to obtain the reflected pressure-time histories shown in Figure 11a. Tabulate all values in Table 17.

Step 11. By following the instructions in Step 11 of Problem A.3, the combined incident/drag pressure histories shown in (b) and (c), Figure 10, are computed for the frame on column line B and those shown in (b) and (c), Figure 11, are computed for the frame on column line 2. All values are tabulated in Table 17.

Step 12. a. Determine the pressure-time histories and shock front velocities at all mass points on the roof using the roof loading parameters listed in Table 17.

Frame on column line B:

Blastward end:

$$(P_{pk})_B = P_{s0} + C_D q_0 = 10.6 \text{ kPa}$$

$$(t_{dr})_B = t_{of} = 0.0601 \text{ second}$$

Leeward end:

$$(P_{pk})_L = P_{so} + C_D q_o = 9.66 \text{ kPa}$$

$$(t_{dr})_L = t_{of} = 0.0629 \text{ second}$$

$$L_R = 18.0 \text{ meters}$$

For node point 11 on the roof (Fig 7a):

$$z_{11r} = 7.5 \text{ meters}$$

Using Equation (15):

$$\begin{aligned} (P_{pk})_{11} &= 10.60 - \frac{(10.60 - 9.66)(7.5)}{18.0} \\ &= 10.21 \text{ kPa} \end{aligned}$$

Using Equation (16):

$$\begin{aligned} (t_{dr})_{11} &= \frac{0.0601 - (0.0601 - 0.0629)(7.5)}{18.0} \\ &= 0.0613 \text{ second} \end{aligned}$$

The shock front velocity, U , is constant across the roof; therefore:

$$U_{11} = 353.6 \text{ m/sec}$$

All values are tabulated in Table 18.

- b. Determine the pressure-time history at all mass points on the leeward wall:

Based on the measurements shown in Figure 12, the spillover from the roof engulfs the leeward wall area in about the same time as the spillover from wall A. Hence, the leeward wall will be taken as an extension of the roof.

For node point 22 on the leeward column (Fig 7a):

$$z_{22l} = 3.75 \text{ meters}$$

$$(P_{pk})_{22} = \frac{9.66 - (10.6 - 9.66)(3.75)}{18.0} \quad [\text{Eq (17)}]$$

$$= 9.46 \text{ kPa}$$

$$(t_{dr})_{22} = 0.0629 - \frac{(0.0601 - 0.0629)(3.75)}{18.0} \quad [\text{Eq (18)}]$$

$$= 0.0635 \text{ sec}$$

The shock front velocity is constant across the area; therefore:

$$U_{22} = 353.6 \text{ m/sec}$$

All values are tabulated in Table 18.

Frame on column line 2:

- a. The pressure-time histories and shock front velocities at all mass points on the roof are determined as illustrated in Step 12a of Example A.3. All values are tabulated in Table 19.
- b. Based on the measurements shown in Figure 12, the spillover from roof produces the more rapid loading on the leeward column; hence, the pressure-history and shock front velocity for all mass points on the leeward wall are computed as illustrated in Step 12 of Example A.3. All values are tabulated in Table 19.

Step 13. Determine the values of a and D for the tributary areas assigned to mass points of each model.

Frame on column line B:

Blastward wall, values of a and D for all mass points from Figure 31 of text:

$$D = 0.0$$

$$a = W_T \sin \alpha$$

From Figure 9: $W_T = 6.0$ meters

From Figure 2: $(\alpha_B)_I = 60^\circ$

$$a = 6.0 \sin 60^\circ = 5.2 \text{ meters}$$

Roof: node 9

From Figure 32b of text:

$$D = d \cos \alpha + (1/2)(W_T - w) \sin \alpha$$

$$a = w \sin \alpha + l \cos \alpha$$

From Figure 9:

$$d = 3.0 \text{ meters}$$

$$w = 6.0 \text{ meters}$$

$$W_T = 6.0 \text{ meters}$$

$$l = 6.0 \text{ meters}$$

$$\begin{aligned} D_9 &= 3.0 \cos 60^\circ + (1/2)(6.0 - 6.0) \sin 60^\circ \\ &= 1.5 \text{ meters} \end{aligned}$$

$$\begin{aligned} a_9 &= 6.0 \sin 60^\circ + 6.0 \cos 60^\circ \\ &= 8.2 \text{ meters} \end{aligned}$$

Roof: node 13

From Figure 9:

$$d = 9.75 \text{ meters}$$

$$w = 2.0 \text{ meters}$$

$$l = 1.5 \text{ meters}$$

$$\begin{aligned} D_{13} &= 9.75 \cos 60^\circ + (1/2)(6.0 - 2.0) \sin 60^\circ \\ &= 6.61 \text{ meters} \end{aligned}$$

$$\begin{aligned} a_{13} &= 2.0 \sin 60^\circ + 1.5 \cos 60^\circ \\ &= 2.48 \text{ meters} \end{aligned}$$

Leeward wall: node 22

From Figure 33b of text:

$$D = (L + h_1) \cos \alpha + (1/2)(W_T - w) \sin \alpha$$

$$a = h_2 \cos \alpha + w \sin \alpha$$

From Figure 9:

$$L = 18.0 \text{ meters}$$

$$h_1 = 2.5 \text{ meters}$$

$$h_2 = 1.67 \text{ meters}$$

$$w = 6.0 \text{ meters}$$

$$\begin{aligned} D_{22} &= (18.0 + 2.50) \cos 60^\circ + (1/2)(6.0 - 6.0) \sin 60^\circ \\ &= 10.25 \text{ meters} \end{aligned}$$

$$\begin{aligned} a_{22} &= 1.67 \cos 60^\circ + 6.0 \sin 60^\circ \\ &= 5.03 \text{ meters} \end{aligned}$$

The measurement of these values of a and D is shown in Figure 13. All values of a and D are tabulated in Table 18.

Frame on column line 2:

Blastward wall, values of a and D for all mass points; from Figure 31 of text:

$$D = 0.0$$

$$a = W_T \sin \alpha$$

$$\text{From Figure 5: } W_T = 6.0 \text{ meters}$$

$$\text{From Figure 2: } (\alpha_A)_I = 30^\circ$$

$$a = 6.0 \sin 30^\circ = 3.0 \text{ meters}$$

Roof: node 9

From Figure 32b of text:

$$D = d \cos\alpha + (1/2) (W_T - w) \sin\alpha$$

$$a = w \sin\alpha + z \cos\alpha$$

From Figure 5:

$$d = 5.0 \text{ meters}$$

$$w = 6.0 \text{ meters}$$

$$W_T = 6.0 \text{ meters}$$

$$z = 2.0 \text{ meters}$$

$$\begin{aligned} D_g &= 5.0 \cos 30^\circ + (1/2)(6.0 - 6.0) \sin 30^\circ \\ &= 4.33 \text{ meters} \end{aligned}$$

$$\begin{aligned} a_g &= 6.0 \sin 30^\circ + 2.0 \cos 30^\circ \\ &= 4.73 \text{ meters} \end{aligned}$$

Leeward wall: node 17

From Figure 33b of text:

$$D = (L + h_1) \cos\alpha + (1/2)(W_T - w) \sin\alpha$$

$$a = h_2 \cos\alpha + w \sin\alpha$$

From Figure 5:

$$L = 12.0 \text{ meters}$$

$$h_1 = 2.50 \text{ meters}$$

$$h_2 = 1.67 \text{ meters}$$

$$w = 6.0 \text{ meters}$$

$$\begin{aligned} D_{17} &= (12.0 + 2.5) \cos 30^\circ + (1/2)(6.0 - 6.0) \sin 30^\circ \\ &= 12.56 \text{ meters} \end{aligned}$$

$$a_{17} = 1.67 \cos 30^\circ + 6.0 \sin 30^\circ = 4.45 \text{ meters}$$

All values of a and D for the model of the frame on column line 2 are tabulated in Table 19.

- Step 14. a. Compute the blast loading parameters for modifying the pressure-time histories at each mass point:

Frame on column line B:

Blastward wall, all mass points:

From Step 10a:

$$P_{pk} = \gamma_{ra} = 25.8 \text{ kPa}$$

$$t_{dr} = t_{of} = 0.0601 \text{ sec}$$

$$t_c = 0.0424 \text{ second}$$

$$U = 353.6 \text{ m/sec (Step 9)}$$

$$P_s = 3.28 \text{ kPa (Fig 10a)}$$

From Step 13:

$$D = 0.0$$

$$a = 5.2 \text{ meters}$$

$$1. \quad t_a = D/U_{AVG} = 0.0/353.6 = 0.0 \quad [\text{Eq (19)}]$$

$$2. \quad t_{rt} = a/U = 5.2/353.6 = 0.015 \text{ sec} \quad [\text{Eq (20)}]$$

$$3. \quad t_R = t_c / (1 - P_s/P_r) \quad [\text{Eq (22)}]$$

$$= 0.0424 / (1 - 3.28/25.8)$$

$$= 0.0486 \text{ second}$$

$$4. \quad (P_{pk})_{AVG} = P_{pk} [1 - (a/2Ut_{dr})] \quad [\text{Eq (21)}]$$

$$\text{Here } t_{dr} = t_R = 0.0486 \text{ second}$$

$$= 25.8(1 - [5.2/2(353.6)(0.0486)])$$

$$= 21.90 \text{ kPa}$$

$$5. \quad t_{pk} = t_a + t_{rt} = 0.0 + 0.015 \text{ sec} \quad [\text{Eq (23)}]$$

$$= 0.015 \text{ second}$$

$$6. \quad t_{c'} = t_a + t_{rt}/2 + t_c \quad [\text{Eq (24)}]$$

$$= 0.0 + 0.015/2 + 0.0424$$

$$= 0.0499 \text{ sec}$$

$$7. \quad t_{DI} = t_{rt}/2 + t_{dr} \quad [\text{Eq (25)}]$$

$$\text{Here } t_{dr} = t_{nf} = 0.0601 \text{ sec}$$

$$t_{DI} = 0.015/2 + 0.0601 = 0.0676 \text{ sec}$$

$$8. \quad t_T = t_a + t_{DI} \quad [\text{Eq (26)}]$$

$$= 0.0 + 0.0676 \text{ sec} = 0.0676 \text{ sec}$$

The values of these parameters for the other mass points are computed in a similar manner and tabulated in Table 18.

- b. Generate the digitized data defining the pressure-time history input for DYNFA as described in Section 8.9 and illustrated in Figures C0 and 61 of the text.

For mass points on blastward column of frame on column line B:

Point	Pressure		Time	
	Parameter	Value (kPa)	Parameter	Value (sec)
1*	-	0.00	t_a	0.0
2	$(P_{pk})_{AVG}$	21.90	t_{pk}	0.0150
3	P_s	3.28	$t_{c'}$	0.0499
4	-	0.00	t_T	0.0676
5	-	0.00	T_F	1.0

*Note: Point 1 corresponds to t_a since $t_a = 0.0$ second in this example.

Frame on column line 2:

Repeat Steps 14a and 14b for the frame on column line 2. The blast loading parameters [t_a , t_c , $(P_{pk})_{AVG}$, etc.] are tabulated in Table 19.

Step 15. Compute and distribute the dead loads acting on each frame according to the method prescribed in Section 8.10.

Frame on column line B:

a. Distribution of weight of roof panel:

1. Uniform load acting on girder:

From Step 4c:

Weight of girder =	240.9 kg
Weight of decking supported by girder =	<u>246.0 kg</u>
Total	486.9 kg

Length of girder = 6.0 meters

$w = 486.9/6.0 = 81.2 \text{ kg/m}$

Apply to elements 5 through 8, 10 through 13 and 15 through 18.

2. Concentrated loads applied to intermediate node points: none.

3. Remainder of weight applied as concentrated loads at girder/column intersections:

Weight of roof panel =	1,318.5 kg
Weight supported by girder =	<u>486.9 kg</u>
Remainder	831.6 kg

Apply: $(1/2)(831.6) = 415.8 \text{ kg}$ at each girder/column intersection (nodes 5, 9, 14 and 19).

b. Distribution of weight of wall panel:

1. Uniform load acting on column:

Weight of column from Step 4a = 116.1 kg/m

Apply to elements 1 through 4 and 19 through 22.

2. Concentrated loads applied to intermediate node points: none.

3. One-half of weight of siding and girts applied as concentrated load at exterior column/girder connection:

From Step 4a:

Weight of siding = 615.0 kg

From Step 4b:

Weight of girt = 232.2 kg

Total = $615.0 + 2(232.2) = 1,079.4$ kg

Apply: $(1/2)(1,079.4) = 539.7$ kg at each exterior column/girder intersection (nodes 5 and 19).

c. Weight of transverse girders applied directly to girder/column connections.

d. Weight of interior column applied as uniform load to elements 9 and 14.

The element uniform loads are tabulated in Table 20; the concentrated nodal point loads are tabulated in Table 21.

Frame on column line 2:

The distribution of the dead loads is illustrated in Step 15 of Example A.3. The element and nodal point loads are tabulated as shown in Tables 12 and 13. Note the corresponding loads for this problem will be somewhat greater than those computed for Example A.3 since heavier columns are required for the bi-axial bending design.

Step 16. Compute Δt , NDT and NSKIP for each frame.

By following the instructions in Problem A.3 and the computations in Step 16 of Example A.3, the following quantities are computed for the frame on column line B:

Periods of extensional modes of frame members:

$$T_a = 2\pi\sqrt{LM_e/AE} \quad [\text{Eq (34)}]$$

<u>Member</u>	<u>Period (sec)</u>
Exterior column	0.0101
Interior column	0.0087
<u>Roof girder</u>	<u>0.0160</u>

Period of highest bending modes of each exterior member:

$$T_b = (1/C)\sqrt{M_1L^3/EI} \quad [\text{Eq (35)}]$$

<u>Member</u>	<u>Highest mode</u>	<u>Period (sec)</u>
Exterior column	2	0.0058
<u>Girder</u>	<u>3</u>	<u>0.0035</u>

Minimum period = 0.0035 sec for third mode of girder

$$\Delta t = 0.0035/20 = 0.00017 \text{ sec}$$

Use $\Delta t = 0.00020 \text{ sec}$.

Duration of response:

The period of the sideway mode of the frame is computed using Equation (36) together with the data in Table 8 of the text. The computed values of T_s is 0.350 second and the response will be computed for one-half this period:

$$T_f = (1/2)T_s = 0.175 \text{ sec}$$

$$NDT = 1 + T_f/\Delta t$$

$$= 1 + 0.175/0.0002 = 876 \quad [\text{Eq (39)}]$$

Use 875 increments in the analysis.

Step 17. Establish the global coordinate system for each model.

Model of frame on column line B.

The origin of the global coordinate system is located at nodal point 1, as shown in Figure 7. The nodal coordinates are specified on the sketch of the model (Fig 7a).

Model of frame on column line 2:

The global coordinate system is established in a similar manner for this model, as shown in (a) and (b) Figure 4.

Step 18. The input data decks for the frames on column lines 1, 2, A and B are prepared utilizing the input data formats specified in Section 8. For the first analysis of each frame, fictitiously high values are entered for the bending capacities of all members designed for bi-axial bending (Section 7.3).

A.4 Utilization of the DYNFA Output in the Design of Frame Structures

Problem A.5: Using DYNFA, compute the blast-induced response of an unbraced, interior, rigid frame of a single-story steel building subjected to a normal shock wave. Determine the adequacy of all frame members by comparing the frame response computed by DYNFA with the frame design criteria of Section 6.

Procedure:

Step 1. With member sizes for the frame established on the basis of the preliminary design (Problem A.1),

and input data deck for the model of the frame prepared as described in Problem A.3, run the analysis of the structure using the DYNFA program on the CDC 6600 computer. For analyzing frames modeled with 15 to 20 elements, specify a running time of at least 10 minutes on the Job Control Card. For models with more than 20 elements, add 1-1/2 minutes of running time for each additional element.

Step 2. Inspect the appropriate portions of the printed output of DYNFA to determine whether the following have occurred:

- a. A successful numerical integration. An unsuccessful integration is generally characterized by extreme fluctuations in the axial load-time histories of some elements accompanied by excessively large axial loads and moments. As a result, excessive deflections at some or all of the nodes occur and the execution of the program may be aborted. Such results are usually caused by an integration-time interval which is too large. Therefore, first check the calculation of the integration-time interval (Section 8.8); recompute the quantity, if necessary, and rerun the job. If the integration-time interval is correct, then check the input data for errors.

The absence of grossly distorted results indicates that a successful integration has been achieved.

- b. A reasonable approximation of the structure's response. There are no exact methods for measuring the accuracy of the structure's response as computed by DYNFA. In general, the deflected shape of a single-story rigid frame subjected to high intensity, short duration blast loadings should resemble that produced by static loads similar to the blast loads. Review the time histories of the nodal displacements to determine the deflected shape of the structure. The occurrence of unexpected or distorted structural behavior may indicate that errors exist in the input data deck. When such behavior occurs, check the input data. Generally, such input errors will affect the entire response history but will not cause a premature termination of the analysis.

- c. The peak sideways response of the frame. Inspect the tabulation of the maximum and minimum nodal displacements that is printed at the end of the output. If each roof node achieves its first peak horizontal displacement at the last integration-time station, T_i , then the frame has not achieved its peak sideways response. If this occurs, specify a 50 percent increase in the number of integration-time stations, NDT, and rerun the job. Before rerunning the job, however, the member sizes should be verified, as described herein, to whatever extent possible using the results available from the unsuccessful run.

On the other hand, if the peak horizontal displacements on the roof occur prior to the last integration-time station, the frame has achieved its peak sideways response.

Step 3. Extract the following data from the printed tabulation of the maxima and minima of the nodal displacements:

- a. The maximum horizontal displacements of all nodal points on the roof girders. Inspect these values. If they are roughly equal, the largest of these values is designated as the peak sideways deflection of the frame, δ . Compare δ with the sideways criteria of Section 6.1. When significant differences occur between the horizontal displacements of the roof nodes (25 percent of the maximum), the results of the analysis are not considered acceptable and the job must be rerun with the appropriate revision in the input data to eliminate these distortions. Such distortions in the analysis are produced by spurious extensional vibrations of the girder in the inelastic response range in which the axial stiffness of the member is only 5 percent of its elastic axial stiffness. Although these vibrations always occur (see Fig 10 of text), they rarely distort the response of the frame. However, in cases involving girders with unusually long spans, these vibrations may produce large axial deformations of the member which could distort the overall

sidesway response of the of the frame. To eliminate these distortions, increase the inelastic stiffness of each element (see Section 3.3) to approximately 7.5 to 10 percent of its elastic stiffness. This is accomplished by entering a value of between 0.075 and 0.10 for the parameter YFACT on the Problem Specification Card (Card Type 2 of the input data deck, see Section 8.13).

- b. The maximum vertical displacement of the nodal point at the midspan of each roof girder. Using these values, compute the maximum chordal angle, θ , for each girder as illustrated in Figure 64 of the text. Tabulate these values for all girders, together with the appropriate chordal angle criteria (Section 6.1), as shown in Table 22.

Step 4. Using the printed output of the nodal displacement-time histories (Section 9.5), determine the maximum differential displacement at the midspan of each exterior column (Fig 65 of text). Using these values, compute the maximum chordal angle, θ , for each exterior column as illustrated in Figure 65 of the text. Tabulate these values for both exterior columns as shown in Table 22.

Step 5. Prepare a tabulation of the maximum ductility ratios occurring at all nodal points on each frame member. Include the following data in this tabulation:

- a. The response time at which each maximum occurs.
- b. The applied axial load to tensile capacity ratio, P/P_U , which occurs when each maximum is achieved. Include also the nature of the applied axial load (either tension or compression).

The above data are contained in the printed tabulation of the element end rotations. Tabulate these values for all frame members as shown in Table 23.

Step 6. Select the appropriate ductility ratio criteria for each frame member. As discussed in Section 6, the selection of the criteria is based on the nature of the axial load on the member when each maximum

ductility ratio is achieved. If several maximum ductility ratios are computed at various locations on a member, the member's response in the inelastic response range is limited by the appropriate ductility ratio criteria for each maximum ductility ratio computed.

The ductility ratio criteria for members subjected to tensile loads is given in Section 6.3. The ductility ratio criteria for members subjected to compressive loads is determined from the rotation capacity data given in Figures 41, 42 and 44 through 53 of the ext. Consult Section 6.4 ("Use of the Rotation Capacity Curves") for guidance on using these figures.

Step 7. Extract from the printed output of the element load-time histories (axial load, shear and bending moment) the maximum shear, V_{max} , acting on each frame member and compare this value with the shear yield capacity of the member given by Equation (3.11) of Reference 1.

- Step 8.
- a. If each of the response quantities determined in Steps 3, 4 and 5 is within the limits specified in Section 6, the member sizes are considered acceptable. If one or more of the limiting values are exceeded by more than a few percent, revise the member sizes as required and rerun the analysis with DYNFA.
 - b. If the maximum shear on a member exceeds its shear yield capacity, increase the size of the member as required to provide the needed shear yield capacity.

Step 9. From the printed output of the element load-time histories, determine the design loads for the connections (Section 9.5). Design the connections according to the provisions given in Chapter 6 of Reference 1.

Example A.5 Using DYNFA, compute the blast-induced response of an unbraced, interior, rigid frame of a single-story steel building subjected to a normal shock wave. Determine the adequacy of the frame members by comparing the frame response computed by DYNFA with the design criteria given in Section 6.

Required: Verification of the blast resistance of the unbraced, interior, rigid frame designed in Example A.1.

Step 1. Given:

- a. Member sizes for the frame, determined in Example A.1.
- b. Input data deck for the model of the frame, prepared in Example A.3.

The frame is analyzed with DYNFA on the CDC 6600 computer. Extracts from the printed output are presented in Appendix B.

Step 2. Inspection of the printed output of the response history indicates that a successful integration has been achieved. There are no gross distortions in the results. In addition, the results depict the expected behavior of the structure.

Inspection of the printed tabulation of the maximum and minimum nodal displacements (Ref Appendix B) reveals that all of the nodal points on the roof girder (nodes 5 through 9 and 11 through 14) achieve their maximum positive horizontal displacements during the time interval beginning at .08015 second and ending at .08890 second. Referring to Step 16 of Example A.3, $\Delta t = 0.00035$ second and $NDT = 500$ integration time stations. The last integration time station, T_f , is:

$$T_f = NDT \times \Delta T = 500(0.00035) = 0.175 \text{ second}$$

Therefore, the frame has attained its peak sideways response.

Step 3. Determine the maximum sideways deflection of the frame and the maximum chordal angles for the roof girders using the printed tabulation of the maximum and minimum nodal displacements.

The following data are extracted from the printed output of the maximum and minimum nodal displacements:

- a. The maximum horizontal $[X(+)]$ displacement of all nodal points on the roof girders:

<u>Node</u>	<u>Displacement (mm)</u>
5	80.8
6	80.3
7	80.3
8	80.3
9	78.7
11	80.3
12	80.3
13	80.4
14	73.5

The difference between the maximum at node 13 and the minimum at node 14 is 9 percent of the displacement at node 13.

Therefore: $\delta = 80.4 \text{ mm}$

$H = 5.0 \text{ m (Fig 1)}$

$$\delta/H = (80.4 \times 10^{-3})/5 = 1/62$$

From Section 6.1 for a reusable structure:

$$\text{maximum } \delta/H \leq 1/50 > 1/62$$

- b. The maximum vertical [Y(-)] displacement at the midspan of each roof girder:

<u>Node</u>	<u>Displacement (mm)</u>
7	-12.7
12	-18.5

The maximum chordal angle for the girder at the leeward end of the roof is computed as follows (see Fig 64 of text):

$$L = 6.0 \text{ m (from Fig 1)}$$

$$y_{12} = -18.5 \text{ mm (from above)}$$

$$\begin{aligned}\theta &= \tan^{-1}[(18.5 \times 10^{-3})/(6.0/2)] \\ &= 0.353^\circ\end{aligned}$$

From Section 6.1, $\theta_{\max} = 1.0^\circ > 0.353^\circ$

The maximum chordal angle for the girder at the blastward end of the roof is computed in a similar manner and tabulated in Table 22.

Step 4. Using the output of the nodal displacement-time histories, determine the maximum chordal angles for the exterior columns.

From the printed output of the nodal displacement-time histories, the maximum differential displacement at the midspan of the blastward column occurs at a response time of 0.0175 second. The following horizontal displacements of nodes 3 and 5 are read from the output at this time station:

$$x_3 = 29.34 \text{ mm}$$

$$x_5 = 9.94 \text{ mm}$$

Referring to Figure 65 of the text, the maximum differential displacement at the midspan of the column is:

$$x_3 - x_5/2 = 29.34 - 9.94 = 24.37 \text{ mm}$$

$$H/2 = 5.0/2 = 2.5 \text{ m}$$

and the maximum chordal angle is:

$$= \tan^{-1}[(24.37 \times 10^{-3})/2.5]$$

$$= 0.56^\circ < 1.0^\circ (\theta_{\max} \text{ from Section 6.1})$$

The maximum chordal angle for the leeward column is computed in a similar manner and tabulated in Table 22.

- Step 5. Tabulate the maximum ductility ratios and related data at all nodal points on each frame member.

The maximum ductility ratios (and related data) for the frame members are tabulated in Table 23. The data tabulated are extracted from the printed output of the maxima and minima of the element end rotations (Appendix B). A ductility ratio of 1.0 indicates that the member remained elastic at the location of the nodal point.

- Step 6. Select the appropriate ductility ratio criteria for each frame member.

Here, most members yield when subjected to compressive axial loads. Therefore, the rotation capacity data given in Figures 41, 42 and 44 through 53 of the text are used to determine most of the ductility ratio criteria.

Referring to Section 6.4, the first step in using these data consists of sketching the bending moment diagrams for the frame members at the instant that the maximum ductility ratios occur. The moments at the nodal points on the frame members are extracted from the printed output of the element load-time histories. Plots of the moment diagrams are presented in Figure 14 for the blastward column at a response time of 0.020 second; in Figure 15 for the girder at the blastward end of the roof at a response time of 0.014; and in Figure 16 for the girder at the leeward end of the roof at response times of 0.055 and 0.070 second.

Since the blastward column is pin-ended and the moment diagram plotted in Figure 14 is similar to that shown for case 2 in Figure 43 of the text, the ductility ratio criteria for the member is determined from the rotation capacity curves in Figures 49 through 53 of the text. Three parameters are required to enter these curves. They are:

- a. The applied axial load to tensile capacity ratio which occurs when the maximum ductility is achieved:

$$P/P_p = 0.064 \text{ (from Table 23)}$$

- b. The slenderness ratio:

$$L/r_x = 33 \text{ (from Table 2)}$$

- c. The distribution factor for the column at the column/girder connection which is computed as follows:

$$DF_{AB} = K_{AB}/\Sigma K \quad [\text{Eq (31) of text}]$$

For the column: $K_C = (3/4)I/L$

$$I = 22,550 \text{ cm}^4 \text{ (from Table 8)}$$

$$L = 5.0 \text{ m} \quad \text{(from Fig 1)}$$

The factor 3/4 is applied because the member is pin-ended.

$$K_C = (3/4)[(22,550 \times 10^{-8})/5] = 3.38 \times 10^{-5}$$

For the girder: $K_g = I/L$

$$I = 12,834 \text{ cm}^4 \text{ (from Table 8)}$$

$$L = 6.0 \text{ m} \quad \text{(from Fig 1)}$$

$$K_g = (12,834 \times 10^{-8})/6 \\ = 2.14 \times 10^{-5}$$

Substituting into Equation (31) of the text:

$$K_{AB} = K_C = 3.38 \times 10^{-5}$$

$$\Sigma K = K_C + K_g = (3.38 + 2.14)10^{-5} \\ = 5.52 \times 10^{-5}$$

$$DF_{AB} = 3.38/5.52 = 0.61$$

Interpolation is required for L/r_x and DF_{AB} . For the values of P/P_p , L/r_x and DF_{AB} in this example, an adequate interpolation can be accomplished using a portion of the applicable rotation capacity data; namely, the curves provided for distribution factors of 0.25, 0.50 and 0.75 (Fig 51, 52 and 53 of text, respectively).

Entering Figures 51 through 53 of the text with $P/P_p = 0.064$, the values of R_c are read from the curves for slenderness ratios of 20, 40 and 60. These values are tabulated in Table 24 and plotted in Figure 17a as curves of R_c versus L/r_x for constant values of DF_{AB} .

Entering the curves of Figure 17a with $L/r_x = 33$, values of R_c are read for values of $DF_{AB} = 0.25$, 0.50 and 0.75. These latter values of R_c are then plotted versus DF_{AB} as shown in Figure 17b, and a value of $R_c = 2.60$ for the member is read off this curve for a corresponding value of $DF_{AB} = 0.61$. The ductility criteria for the member is then computed as shown below:

$$\mu_{max} = 1.0 + R_c/4 \text{ (from Section 6.5 for reusable structure)}$$

$$R_c = 2.6 \text{ (from Fig 17b), therefore:}$$

$$\mu_{max} = 1.65 < 2.567 \text{ (maximum } \mu \text{ computed at node 2 on element 1 from Table 23)}$$

The moment diagram plotted for the blastward girder (Fig 15) is a combination of the moment diagram for case 2 in Figure 40 of the text with that of case 1 in Figure 43 of the text. Although the major portion of each peak ductility at the member's end appears to be caused by applied end moments (Case 2, Fig 40 of text), the rotation capacity data for the lateral load case (Fig 43 of text, Case 1) will be used for the design verification as these data represent a lower bound on the ductility criteria for the member. Interpolation of the data in Figures 44 through 48 of the text using the guidelines provided in Section 6.4 yields the following rotation capacities for the blastward girder:

at $t = 0.014$ sec: $P/P_p = 0.143$

$$R_C = 6.8$$

$$\mu_{\max} = 1 + R_C/4 = 1 + 6.8/4 = 2.7$$

At $t = 0.055$ second, the moment diagram (Fig 16) for the leeward girder is a combination of the moment diagram for case 2, Figure 40, with that of case 1, Figure 43 (both of text). Hence, the ductility criteria for the member are extracted from Figures 44 through 48 of the text.

At $t = 0.070$ second, the moment diagram for the member corresponds to that for case 2, Figure 40 of the text. Therefore, the ductility criteria are extracted from Figure 41 of the text.

Summarizing the ductility criteria for the girder:

$$l/r_x = 46.1$$

For $t = 0.055$ second: $P/P_p = 0.022$

$$R_C = 7.5 \text{ by inspection of Figure 47 of text}$$

$$\mu_{\max} = 1.0 + 7.5/4 = 2.88$$

For $t = 0.070$ second: $P/P_p = 0.114$

$$R_C > 15 \text{ by inspection of Figure 41 of text}$$

Therefore: $\mu_{\max} = 3.0$

Values of μ_{\max} for all frame members are tabulated in Table 23.

Step 7. Extract from the printed output of the element load-time histories, the maximum shear for each member and compare this value with the shear yield capacity of the member.

For the leeward girder:

$$V_{\max} = 193 \text{ kN at the leeward girder/column connection (element 13, node 14) at } t = 0.0511 \text{ sec.}$$

Using Equation (3.11) of Reference 1, the shear yield capacity of the member is computed as shown below:

$$V_p = F_{dv} A_w$$

$$F_{dv} = 0.55 F_{dy} \quad [\text{Eq (2.3) of Ref 1}]$$

$$F_{dy} = 273 \times 10^3 \text{ kPa} \quad (\text{Step 3, Example A.1})$$

$$F_{dv} = (0.55)(273 \times 10^3) = 150 \times 10^3 \text{ kPa}$$

$$A_w = t_w(d - 2t_f) \quad (\text{Section 3.3.3 of Ref 1})$$

From the "Properties for designing" tables of Reference 14 for a W12 x 40:

$$d = 11.940 \text{ in (303.3 mm)}$$

$$t_w = 0.294 \text{ in (7.46 mm)}$$

$$t_f = 0.516 \text{ in (13.11 mm)}$$

$$A_w = 7.46 [303.3 - (2 \times 13.11)] = 2,067 \text{ mm}^2$$

$$V_p = (150 \times 10^3)(2,067 \times 10^{-6})$$

$$= 310.1 \text{ kN} > 193 \text{ kN}$$

A tabulation of the maximum applied shear and the shear yield capacity is provided in Table 25 for each frame member.

Step 8. A comparison of each of the response quantities determined in Steps 3, 4 and 5 with the limits specified in Section 6, indicates that the blastward column, with a ductility ratio of 2.57 at node point 2 of element 1, should be increased in size and the remaining numbers should remain the same since the frame response approaches the limiting sideways deflection of $\delta/H = 1/50$.

Step 9. Once the final member sizes are established and verified, determine the connection design loads from the printed output as described in Section 9.5, and design the connections in accordance with the provisions given in Chapter 6 of Reference 1.

Problem A.6: Using DYNFA, compute the blast-induced responses of several representative frames spanning in each direction of a single-story rigid frame steel building subjected to a quartering shock wave. Determine the adequacy of all frame members by comparing the frame responses computed by DYNFA with the frame design criteria of Section 6.

Procedure:

- Step 1. Compute the response of each frame by running DYNFA on the CDC 6600 computer. Refer to Step 1 of Problem A.5 for guidance in specifying running times for this computer.
- Step 2. Inspect the results of each analysis as instructed in Step 2 of Problem A.5.
- Step 3. Compute the following quantities as instructed in Step 3 of Problem A.5:
 - a. Peak sidesway deflection of each frame.
 - b. Maximum chordal angle for each girder.
- Step 4. Compute the maximum chordal angle for each exterior column as instructed in Step 4 of Problem A.5.
- Step 5. Tabulate the maximum ductility ratios for each girder and determine the appropriate ductility criteria as instructed in Steps 5 and 6 of Problem A.5.
- Step 6. If each of the response quantities determined in Steps 3, 4 and 5 is within 90 percent of the limit specified for it in Section 6, proceed with the next stage of the design. If one of these response quantities exceeds 90 percent of the limit specified for it, increase the sizes of the appropriate members and rerun the analysis of the frame.
- Step 7. a. Extract from printed tabulations of the maximum and minimum element end loads, the peak bending moments at all nodal points on each column analyzed.

- b. Prepare a tabulation of the maximum moments about both axes of bending at all nodal points of each column (Section 7.3).
- c. Using the ultimate bending capacities determined in Step 11 of Problem A.2 and the tabulated moments from above, compute the nodal reduction factors and reduced bending capacities for each column using Equations (32) and (33). Tabulate all values as shown in Table 26.

Step 8. Rerun all analyses with DYNFA using the reduced bending capacities for the columns that were determined in Step 7.

- Step 9.
- a. Review the results of the second group of analyses as instructed in Steps 2 through 5.
 - b. Tabulate the maximum ductility ratios for each column.
 - c. Determine the appropriate ductility criteria for each column as instructed in Step 6 of Problem A.5.

Step 10. Step 7 of Problem A.5.

- Step 11.
- a. Step 8a of Problem A.5.
 - b. Step 8b of Problem A.5.

Step 12. Step 9 of Problem A.5.

Example A.6: Using DYNFA, compute the blast-induced responses of several representative frames spanning in each direction of a single-story rigid frame steel building subjected to a quartering shock wave. Determine the adequacy of all frame members by comparing the frame responses computed by DYNFA with the frame design criteria of Section 6.

Required: Verification of the blast resistance of the primary structural framing system designed in Example A.2.

Step 1. Given:

- a. Member sizes for the frames on column lines A, B, 1 and 2, determined in Example A.2.
- b. Input data decks for these frames, prepared as illustrated in Example A.4.

Each frame is analyzed with SYNFA on the CDC 6600 computer.

Step 2. Inspection of the printed output for each frame indicates that a successful integration has been achieved in each analysis and each frame has achieved a peak sidesway response prior to the final integration time step.

Step 3. a. The peak sidesway deflections for each frame are listed below:

<u>Frame</u>	<u>Peak sidesway deflection (mm)</u>	<u>$\delta/H \leq 1/50$</u>
Column line A	47	1/106
Column line B	63	1/79
Column line 1	40	1/125
Column line 2	49	1/102

b. The maximum chordal angles for each girder are:

<u>Girder</u>	<u>θ_{max}</u>
G1	0.20°
G2	0.17°
G3	0.31°
G4	0.32°

Step 4. The maximum chordal angles for the exterior columns are:

<u>Frame</u>	<u>Column</u>	<u>θ_{max}</u>
Column line A	blastward	0.15°
Column line B	blastward	0.33°
Column line 1	blastward	0.51°
Column line 2	blastward	0.40°

Step 5. The maximum ductility ratios and the corresponding ductility criteria for the girders are:

<u>Frame</u>	<u>Girder</u>	<u>Maximum computed ductility ratio, μ</u>	<u>Ductility criteria, μ_{max}</u>
Column line A	ali	1.00	-
Column line B	blastward	1.03	2.2
Column line B	middle	1.00	-
Column line B	leeward	1.33	3.0
Column line 1	blastward	2.69	3.0
Column line 1	leeward	1.20	3.0
Column line 2	blastward	1.70	2.0
Column line 2	leeward	1.71	1.75

The ductility criteria listed above were determined using the data provided in Figures 41, 42 and 44 through 53 of the text.

Step 6. Comparisons of each of the response quantities, determined in Steps 3, 4 and 5 with the limits specified for them in Section 6, indicate that the responses of the four frames are within acceptable limits.

Step 7. A tabulation of the bending moments at all nodal points on column B-1 is shown in Table 26. Inspection of the computed nodal reduction factors listed in the table, reveals that the member is oversized; hence, the member is decreased in size from a W14 x 78 to a W14 x 61. The computation of the nodal reduction factors for the new member is shown in Table 27.

Similar calculations for columns A-1 and A-4 produce maximum nodal reduction factors of 1.44 and 1.28, respectively. Hence, these members remain the same. The maximum nodal reductions for columns B-2 and B-3 are 1.0 and 1.08, respectively. The next smallest size that meets all of the minimum thickness criteria of Section 2.4 is a W14 x 84, which does not have adequate strength. Hence, columns B-2 and B-3 also remain the same.

Calculation of nodal reduction factors for columns A-2, A-3, C-2 and C-3 reveals these members to be oversized; therefore, they are reduced to W14 x 61. Reduced bending capacities for these members are computed in the manner illustrated in Table 27.

Step 8. All frames are re-analyzed using the reduced bending capacities for the columns determined in Step 7.

Step 9. a. 1. Inspection of the printed output for the second group of analyses indicates that each of these runs was successful and each frame attains a peak sideways response prior to the final integration time step.

2. The results of the second group of analyses are listed as follows:

Maximum sideways deflections of frames

<u>Frame</u>	<u>Peak sideways deflection (mm)</u>	<u>$\delta/H \leq 1/50$</u>
Column line A	52	1/96
Column line B	65	1/77
Column line 1	43	1/116
Column line 2	53	1/94

Maximum chordal angles for girders

<u>Girder</u>	<u>θ_{max}</u>
G1	0.21°
G2	0.18°
G3	0.32°
G4	0.33°

Maximum chordal angles for exterior columns

<u>Frame</u>	<u>Column</u>	<u>θ_{max}</u>
Column line A	blastward	0.31°
Column line B	blastward	0.45°
Column line 1	blastward	0.88°
Column line 2	blastward	0.54°

Maximum ductility ratios
and
corresponding ductility criteria for girders

<u>Frame</u>	<u>Girder</u>	<u>Maximum computed ductility ratio, μ</u>	<u>Ductility criteria μ_{max}</u>
Column line A	all	1.00	-
Column line B	blastward	1.30	2.0
Column line B	middle	1.00	-
Column line B	leeward	1.03	3.0
Column line 1	blastward	2.73	3.0
Column line 1	leeward	1.10	3.0
Column line 2	blastward	1.54	3.0
Column line 2	leeward	1.44	1.65

The ductility criteria listed above were determined using the data provided in Figures 41, 42 and 44 through 53 of the text.

b. The maximum ductility ratios for each column are listed below:

<u>Frame</u>	<u>Column</u>	<u>Maximum computed ductility ratio, μ</u>
Column line A	blastward	3.0
Column line A	leeward	1.1
Column line B	blastward	1.5
Column line B	leeward	1.8
Column line 1	blastward	2.0
Column line 1	leeward	1.0
Column line 2	blastward	2.0
Column line 2	leeward	1.7

c. The corresponding ductility criteria for the above ductility ratios are listed below:

<u>Frame</u>	<u>Column</u>	<u>Ductility criteria μ_{max}</u>
Column line A	blastward	3.00
Column line A	leeward	3.00
Column line B	blastward	1.50
Column line B	leeward	3.00
Column line 1	blastward	3.00
Column line 1	leeward	3.00
Column line 2	blastward	2.2
Column line 2	leeward	2.2

The ductility criteria listed above were determined using the data in Figures 41, 42 and 44 through 53 of the text.

- Step 10. The maximum member shears are extracted from the printed output and the shear capacity of each member is computed as described in Section 3.3.3 of Reference 1. All members have adequate shear capacity. The computations are omitted for brevity.
- Step 11. All response quantities computed in Step 9 are within the frame design criteria specified in Section 6. Hence, the member sizes selected are acceptable.
- Step 12. Step 9 of Example A.5.

Table 1
Trial selections of C , C_1 and M_p for frame on column line 2, Example A.1

Collapse mechanism	w (kN/m)	M_p (kN-m)	M_p (kN-m)		
			Trial 1 $C_1=2, C=2$	Trial 2 $C_1=2, C=2.5$	Trial 3 $C_1=2, C=1.5$
1	68.4	154	154	154	154
2	68.4	$906/(2C+1)$	181	151	227
3a	34.2	$906/(2+C_1)$ ($C_1 \leq 2$)	227	227	227
3b	34.2	227 ($C_1 \geq 2$)	227	227	227
4	34.2	190	190	190	190
5a	34.2	$680/(C + 1/2 + C_1/2)$ ($C_1 \leq 2$)	194	170	227
5b	34.2	$680/(C + 3/2)$ ($C_1 \geq 2$)	194	170	227
6	34.2	$834/(C + 5/2)$	185	167	209

Table 2
Member sizes and capacities for frame on column line 2, Example A.1

Member	Member Size	Tabulation of moments (kN-m)			$\frac{(M_p)_x}{M_{mix}}$	$\frac{d^*}{t_w} \leq 58.4$	$\frac{b_f^*}{2t_f} \leq 8.5$	$\frac{L}{r} \leq 122$
		$(M_p)_x$	M_{px}	M_{mix}				
Girder	W12 x 40	227	258	254	0.90	40.6	7.75	46
Interior column	W14 x 61	454	457	415	1.09	36.8	7.78	33
Exterior column	W14 x 53	341	390	390	0.87	37.7	6.73	33

* d/t_w and $b_f/2t_f$ ratios for these members given in "Properties for designing" tables of Reference 14.

Table 3
Tabulation of peak pressures
for
preliminary design, Example A.2

Wall	Point	R (ft)	Z (ft/lb ^{1/3})	P _{so} (psi)	P _{so} (kPa)	α _f (deg)	C _{rα}	P _{rα} =C _{rα} P _{so} (kPa)
A	I	433	30.0	1.65	11.4	30	2.11	24.0
	II	465	32.3	1.50	10.3	36.3	2.15	22.1
B	I	433	30.0	1.65	11.4	60	2.46	28.0
	III	467	32.4	1.50	10.3	62.4	2.35	24.2

Table 4
Trial selections of C, C₁ and M_p for frame of column line B, Example A.2

Collapse mechanism	W (kN-m)	M _p (kN-m)	M _p (kN-m)	
			Trial 1 C ₁ = 2.0, C = 3.0	Trial 2 C ₁ = 2.0, C = 2.5
1	22.8	51	51	51
2	19.8	912/(2C + 1)	130	152
3a	9.9	456/(1 + C ₁) C ₁ ≤ 2	152	152
3b	9.9	152 C ₁ ≥ 2	152	152
4	9.9	98	98	98
5a	9.9	684/(C + 1/2 + C ₁) C ₁ ≤ 2	98	98
5b	9.9	684/(C + 5/2) C ₁ ≥ 2	124	137
6	9.9	733/(C + 9/2)	103	110

Table 5

Required bending capacities for frame members, Example A.2

Member	$(M_p)_x$ (kn-M)	$(M_p)_y$ (kN-m)	Size	M_{px} (kN-m)	M_{mx} (kN-m)	M_{py} (kN-m)	M_{my} (kN-m)	$\frac{(M_p)_x}{M_{mx}}$	$\frac{(M_p)_y}{M_{my}}$
G1	152	-	W12 x 27	170.0	170.0	-	-	0.89	-
G2	76	-	W12 x 16.5	92.2	92.2	-	-	0.82	-
G3	210	-	W12 x 36	231.0	223.4	-	-	0.94	-
G4	105	-	W12 x 22	131.5	116.4	-	-	0.90	-
C1	420	304	W14 x 111	876.8	845.6	421.9	421.9	0.50	0.72
C2	380	210	W14 x 78	599.5	599.5	234.4	234.4	0.63	0.90
C3	315	152	W14 x 68	514.5	514.5	164.6	164.6	0.61	0.92
C4	190	158	W14 x 68	514.5	514.5	164.6	164.6	0.37	0.96



DESIGNATION OF AXES OF BENDING

Table 6
 Tabulation of d/t_w , $b/2t_f$ and slenderness ratios
 for
 frame members, Example A.2

Member	Size	$d/t_w^* \leq 58.4$	$b_f/2t_f^* \leq 8.5$	l/r_x	l/r_y
G1	W12 x 27	50.5	8.12	46.6	-
G2	W12 x 16.5	52.2	7.43	50.8	-
G3	W12 x 36	40.1	6.08	45.9	-
G4	W12 x 22	47.3	4.75	48.1	-
C1	W14 x 111	26.6	8.37	31.6	52.6
C2	W14 x 78	32.9	8.36	32.3	21.9
C3	W14 x 68	33.6	6.99	32.7	26.7
C4	W14 x 68	36.8	6.99	10.9	26.7

* d/t_w and $b_f/2t_f$ for these members given in "Properties for designing tables of Reference 14.

Table 7

Tabulation of concentrated masses at mass points of model
of frame on column line 2, Example A.3

Node number of mass point	Horizontal dynamic degrees of freedom		Vertical dynamic degrees of freedom	
	Item	Mass (kg)	Item	Mass (kg)
2, 4, 15, 17	Exterior wall panel (M_{IH}^W)	568.7	-	-
5, 14	Exterior wall panel (M_{EH}^W)	168.3	Exterior wall panel (M_{EV}^W)	737.0
	Roof panel (M_{EH}^R)	716.4	Roof panel (M_{EV}^R)	182.4
	Transverse girder	321.6	Transverse girder	321.6
	Total	1,206.3	Total	1,241.6
6, 8, 11, 13	-	-	Roof panel (M_{IV}^R)	534.0
9	Roof panel (M_{EH}^R)	716.4	Roof panel (M_{EV}^R)	182.4
	Roof panel (M_{EH}^R)	716.4	Roof panel (M_{EV}^R)	182.4
	1/2 Interior column	226.9	1/2 Interior column	226.9
	Transverse girder	321.6	Transverse girder	321.6
	Total	1,981.3	Total	913.3

Table 8

Input data for elements of model of frame on column line 2, Example A.3

Member	Size	Element data										
		I.D. No.		Nodal connectivities		Pin codes		A (cm ²)	I (cm ⁴)	M _m [*] (kN-m)	P _P (kN)	P _U (kN)
		End A	End B	End A	End B	End A	End B					
Exterior column - blastward	W14x53	1	2	none	none	101.0	22,550	390	2,750	2,370		
		2	2	none	none	101.0	22,550	390	2,750	2,370		
		3	4	none	none	101.0	22,550	390	2,750	2,370		
		4	5	none	none	101.0	22,550	390	2,750	2,370		
Roof girder - blastward end of roof	W12x40	5	6	none	none	76.1	12,834	254	2,078	1,867		
		6	7	none	none	76.1	12,834	254	2,078	1,867		
		7	8	none	none	76.1	12,834	254	2,078	1,867		
		8	9	none	none	76.1	12,834	254	2,078	1,867		
Interior column	W14x61	9	10	none	none	115.0	26,670	415	3,155	2,222		
Roof girder - leeward end of roof	W12x40	9	11	none	none	76.1	12,834	254	2,078	1,867		
		11	12	none	none	76.1	12,834	254	2,078	1,867		
		12	13	none	none	76.1	12,834	254	2,078	1,867		
		13	14	none	none	76.1	12,834	254	2,078	1,867		
Exterior column - leeward	W14x53	14	14	none	none	101.0	22,550	390	2,750	2,370		
		15	15	none	none	101.0	22,550	390	2,750	2,370		
		16	16	none	none	101.0	22,550	390	2,750	2,370		
		17	17	none	none	101.0	22,550	390	2,750	2,370		

*M_m determined in preliminary design of frame, Example A.1.

Table 9
Tributary areas for mass points of model of frame on column line 2, Example A.3

Node no. of mass point	Tributary areas*					
	For horizontal loads			For vertical loads		
	Loaded surface	Dimensions of area (m)	Area (m ²)	Loaded surface	Dimensions of area (m)	Area (m ²)
2, 4	Blastward wall	6.0 x 1.670	10.0	-	-	-
5	Blastward wall	6.0 x 0.835	5.0	Roof	6.0 x 1.0	6.0
6, 8, 11, 13	-	-	-	Roof	6.0 x 2.0	12.0
9	-	-	-	Roof: blastward panel	5.0 x 1.0	6.0
	-	-	-	Roof: leeward panel	6.0 x 1.0	6.0
				Totals	6.0 x 2.0	12.0
14	Leeward wall	6.0 x 0.835	5.0	Roof: leeward panel	6.0 x 1.0	6.0
15, 17	Leeward wall	6.0 x 1.670	10.0	-	-	-

*Reference: Figure 5

Table 10

Blast loading parameters, Example A.3

Location	R_A (ft)	Z (ft/16 ^{1/3})	P_{50} (psf)	$i_s^{1/3}$ (psf-ms/16 ^{1/3})	i_s (psf-ms)	U (ft-ms)	P_r (psf)	P_r (kPa)	$i_r^{1/3}$ (psf-ms/16 ^{1/3})
Blastward wall	433.0	30.00	1.65	3.40	49.03	1.16	3.51	24.2	6.90
Roof-blastward end	433.0	30.00	1.65	3.40	49.03	1.16	-	-	-
Roof-leeward end	472.4	32.75	1.46	3.16	45.55	1.16	-	-	-

i_r (psf-ms)	t_c (sec)	t_{of} (sec)	t_r (sec)	q_0 (psf)	C_p	$P_{50} + C_p q_0$ (psf)	$P_{50} + C_p q_0$ (kPa)	Location
99.5	0.0424	0.0594	0.0567	0.066	1.0	1.72	11.8	Blastward wall
-	-	0.0594	-	0.066	-0.40	1.62	11.1	Roof-blastward end
-	-	0.0624	-	0.052	-0.40	1.42	9.9	Roof-leeward end

Table 17
Blast loading data for DYNFA analysis of frame on column line 2, Example A.3

Mode no. of mass point	Direction of load	Blast loading parameters at mass points					Parameters a and D for tributary areas			Parameters used for modifying pressure waveforms						
		p_a (kPa)	t_{dr}^a (sec)	p_s (kPa)	t_c^a (sec)	U (m/sec)	a (m)	D (m)	t_a^b (sec)	t_{rt} (sec)	(P_{pk}/V_s) (kPa)	t_{pk}^b (sec)	t_{DI} (sec)	t_f^b (sec)	Waveform I.D. no.	
2	Horizontal	24.2	0.0594	3.5	0.042	-	0.00	0.00	0.0000	0.0028	10.60	0.0028	0.0608		1	
4	Horizontal	24.2	0.0594	3.5	0.042	-	0.00	0.00	0.0000	0.0028	10.40	0.0028	0.0627		1	
5	Horizontal	24.2	0.0594	3.5	0.042	-	0.00	0.00	0.0000	0.0028	10.20	0.0028	0.0632		1	
5	Vertical	11.1	0.0594	-	-	353.6	1.00	0.00	0.0000	0.0028	10.60	0.0028	0.0608		2	
6	Vertical	10.9	0.0599	-	-	353.6	2.00	1.00	0.0028	0.0056	10.40	0.0084	0.0627		3	
8	Vertical	10.7	0.0604	-	-	353.6	2.00	3.00	0.0085	0.0056	10.20	0.0141	0.0632		4	
9	Vertical	10.5	0.0609	-	-	353.6	2.00	5.00	0.0141	0.0056	10.00	0.0197	0.0637		5	
11	Vertical	10.3	0.0614	-	-	353.6	2.00	7.00	0.0197	0.0056	9.83	0.0253	0.0642		6	
13	Vertical	10.1	0.0619	-	-	353.6	2.00	9.00	0.0254	0.0056	9.64	0.0310	0.0647		7	
14	Vertical	9.9	0.0624	-	-	353.6	1.00	11.00	0.0311	0.0028	9.67	0.0379	0.0638		8	
14	Horizontal	9.9	0.0624	-	-	353.6	0.83	12.00	0.0339	0.0023	9.71	0.0362	0.0636		9	
15	Horizontal	9.7	0.0628	-	-	353.6	1.67	12.83	0.0363	0.0047	9.34	0.0410	0.0652		10	
17	Horizontal	9.5	0.0632	-	-	353.6	1.67	14.50	0.0410	0.0047	9.15	0.0457	0.0656		11	

^aParameters used to generate pressure-time input for horizontal loading applied to mass points on blastward wall (nodes 2, 4 and 5).

^bParameters used to generate pressure-time input for loading applied to mass points on roof and leeward walls.

Table 12

Dead loads applied to elements of model
of frame on column line 2 Example A.3

Element number	Uniform load (kg/m)	Element number	Uniform load (kg/m)
1	78.9	10	59.4
2	78.9	11	59.4
3	78.9	12	59.4
4	78.9	13	59.4
5	59.4	14	78.9
6	59.4	15	78.9
7	59.4	16	78.9
8	59.4	17	78.9
9	90.8		

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Table 13

Dead loads applied at nodal points of model
of frame on column line 2, Example A.3

Nodal point	Item	Load (kg)
5, 14	Roof decking	123.0
	Wall panel	539.7
	Transverse girder	<u>321.6</u>
	Total	984.3
6, 8, 11, 12	Roof panel - purlin load	415.2
9	Roof decking:	
	blastward	123.0
	leeward	123.0
	Transverse girder	<u>321.6</u>
	Total	567.6

Table 14
 Tabulation of concentrated masses at mass points
 of
 model of frame on column line B, Example A.4

Node number of mass point	Horizontal dynamic degrees of freedom		Vertical dynamic degrees of freedom	
	Item	Mass (kg)	Item	Mass (kg)
2, 4, 20, 22	Exterior wall panel (M_{IH}^W)	630.7	-	-
5, 19	Exterior wall panel (M_{EH}^W)	199.3	Exterior wall panel (M_{EV}^W)	830.0
	Roof panel (M_{EH}^R)	659.3	Roof panel (M_{EV}^R)	476.6
	Transverse girder	285.6	Transverse girder	285.6
	Total	1,144.2	Total	1,592.2
6, 7, 8, 11, 12, 13, 16, 17, 18	-	-	Roof panel (M_{IV}^R)	121.7
9, 14	Roof panel (M_{EH}^R)	659.3	Roof panel (M_{EV}^R)	476.6
	Roof panel (M_{EH}^R)	650.3	Roof panel (M_{EV}^R)	476.6
	1/2 interior column	413.0	1/2 interior column	413.0
	Transverse girder	321.6	Transverse girder	321.6
	Total	2,053.2	Total	1,687.8

Table 15

Input data for elements of model of frame on column line B, Example A.4

Member	Size	Element data									
		Nodal connectivities				Pin codes		Cross-section properties and capacities			
		I.D. No.	End A	End B	PA	PB	A (cm ²)	I (cm ⁴)	M _m (kN-m)	P _p (kN)	P _u (kN)
Exterior column - blastward	W14x78	1	2	0	0	0	147.7	35,421	1.0x10 ⁶	4,033	3,654
		2	3	0	0	0	147.7	35,421	1.0x10 ⁶	4,033	3,654
		3	4	0	0	0	147.7	35,421	1.0x10 ⁶	4,033	3,654
		4	5	0	0	0	147.7	35,421	1.0x10 ⁶	4,033	3,654
Roof girder - blastward end of frame	W12x27	5	6	0	0	0	51.2	8,491	170.0	1,398	1,287
		6	7	0	0	0	51.2	8,491	170.0	1,398	1,287
		7	8	0	0	0	51.2	8,491	170.0	1,398	1,287
		8	9	0	0	0	51.2	8,491	170.0	1,398	1,287
Interior column	W14x111	9	10	9	0	0	211.0	18,938	1.0x10 ⁶	5,760	4,120
Roof girder - center span	W12x27	10	11	11	0	0	51.2	8,491	170.0	1,398	1,287
		11	12	12	0	0	51.2	8,491	170.0	1,398	1,287
		12	13	13	0	0	51.2	8,491	170.0	1,398	1,287
		13	14	14	0	0	51.2	8,491	170.0	1,398	1,287
Roof girder - leeward end of frame	W12x27	14	15	16	0	0	51.2	8,491	170.0	1,398	1,287
		15	16	17	0	0	51.2	8,491	170.0	1,398	1,287
		16	17	18	0	0	51.2	8,491	170.0	1,398	1,287
		17	18	19	0	0	51.2	8,491	170.0	1,398	1,287
Exterior column - leeward	W14x78	18	19	19	0	0	147.7	35,421	1.0x10 ⁶	4,033	3,654
		19	20	20	0	0	147.7	35,421	1.0x10 ⁶	4,033	3,654
		20	21	21	0	0	147.7	35,421	1.0x10 ⁶	4,033	3,654
		21	22	22	0	0	147.7	35,421	1.0x10 ⁶	4,033	3,654

Table 16
Tributary areas for mass points of model of frame on column line B, Example A.4

Node no. of mass point	Tributary areas ^a					
	For horizontal loads			For vertical loads		
	Loaded surface	Dimensions of area (m)	Area (m ²)	Loaded surface	Dimensions of area (m)	Area (m ²)
2, 4	Blastward wall	6.0 x 1.670	10.0	-	-	-
5	Blastward wall	6.0 x 0.835	5.0	Roof ^b	6.0 x 3.0 - 2.0 x 2.25	13.5
6, 7, 8, 11, 12, 13, 16, 17, 18	-	-	-	Roof ^b	2.0 x 1.5	13.5
9, 14	-	-	-	Roof ^b	6.0 x 3.0 - 2.0 x 2.25 6.0 x 3.0 - 2.0 x 2.25 Total	13.5 13.5 27.0
19	Leeward wall	6.0 x 0.835	5.0	Roof ^b	6.0 x 3.0 - 2.0 x 2.25	13.5
20, 23	Leeward wall	6.0 x 1.670	10.0	-	-	-

^aReference Figure 9

^bReference Figure 8

Table 17

Blast loading parameters for Example A.4

Column Line	Location	R (ft)	Z (ft/lb ^{1/3})	P _{so} (psf)	$1_s/M^{1/3}$ (psl-ms/lb ^{1/3})	1_s (ft/ms)	U (ft/ms)	α_1 (deg)	C _{pc}	P _{ru} (psf)	P _{ru} (kPa)
B	Blastward wall	450.1	31.2	1.56	3.25	46.9	1.16	61.2	2.40	3.74	25.8
	Roof: Blastward end	450.1	31.2	1.56	3.25	46.9	1.16	-	-	-	-
	Leeward end	481.4	33.4	1.42	3.10	44.7	1.16	-	-	-	-
2	Blastward wall	443.2	30.7	1.60	3.30	47.6	1.16	32.2	2.11	3.38	28.3
	Roof: Blastward end	443.2	30.7	1.60	3.30	47.6	1.16	-	-	-	-
	Leeward end	477.0	33.1	1.44	3.10	44.7	1.16	-	-	-	-

$1_r/M^{1/3}$ (psl-ms/ft ^{1/3})	1_r (psl-ms)	t _c (sec)	t _{of} (sec)	t _r (sec)	q _o (psf)	C _D	P _{so} + C _D q _o (psf)	P _{so} + C _D q _o (kPa)	Location	Column Line
7.2	103.8	0.0424	0.0601	0.0555	0.056	1.0	1.62	11.14	Blastward wall	B
-	-	-	0.0601	-	0.056	-0.4	1.54	10.60	Roof: Blastward end	
-	-	-	0.0629	-	0.046	-0.4	1.40	9.66	Leeward end	
6.6	95.2	0.0424	0.0595	0.0560	0.060	1.0	1.66	11.44	Blastward wall	2
-	-	-	0.0595	-	0.060	-0.4	1.58	10.86	Roof: Blastward end	
-	-	-	0.0621	-	0.046	-0.4	1.42	9.82	Leeward end	

Table 18

Blast loading data for DYNFA analysis of frame on column line B, Example A.4

Node no. of mass point	Direction of load	Blast loading parameters at mass points					Parameters a and D for tributary areas					Parameters used for modifying pressure waveforms						
		P _{pk} (kPa)	t _{dr} (sec)	P _s (kPa)	t _c (sec)	U (m/sec)	a (m)	D (m)	t _g (sec)	t _{pt} (sec)	(P _{pk}) _{AVG} (kPa)	t _{pk} (sec)	P _s (kPa)	t _c (sec)	t _{DI} (sec)	t _g (sec)	Waveform I.D. no.	
2	Horizontal	25.8	0.0601	3.28	0.0424	353.6	5.20	0.00	0.0000	0.0150	21.90	0.0150	3.28	0.0499	0.0676	0.0676	1	
4	Horizontal	25.8	0.0601	3.28	0.0424	353.6	5.20	0.00	0.0000	0.0150	21.90	0.0150	3.28	0.0499	0.0676	0.0676	1	
5	Horizontal	25.8	0.0601	3.28	0.0424	353.6	5.20	0.00	0.0000	0.0150	21.90	0.015	3.28	0.0499	0.0676	0.0676	1	
5	Vertical	10.6	0.0601	-	-	353.6	6.70	0.00	0.0000	0.0189	8.93	0.0189	-	-	0.0696	0.0696	2	
6	Vertical	10.5	0.0603	-	-	353.6	2.48	2.10	0.0059	0.0070	9.91	0.0129	-	-	0.0639	0.0698	3	
7	Vertical	10.4	0.0606	-	-	353.6	2.48	2.86	0.0081	0.0070	9.84	0.0151	-	-	0.0641	0.0722	4	
8	Vertical	10.4	0.0608	-	-	353.6	2.48	3.61	0.0102	0.0070	9.77	0.0172	-	-	0.0643	0.0745	5	
9	Vertical	10.3	0.0610	-	-	353.6	8.20	1.50	0.0042	0.0232	8.33	0.0274	-	-	0.0726	0.0768	6	
11	Vertical	10.2	0.0613	-	-	353.6	2.48	5.11	0.0145	0.0070	9.63	0.0215	-	-	0.0648	0.0793	7	
12	Vertical	10.1	0.0615	-	-	353.6	2.48	5.86	0.0166	0.0070	9.55	0.0236	-	-	0.0650	0.0816	8	
13	Vertical	10.1	0.0617	-	-	353.6	2.48	6.61	0.0187	0.0070	9.48	0.0257	-	-	0.0652	0.0839	9	
14	Vertical	10.0	0.0620	-	-	353.6	8.20	4.50	0.0127	0.0232	8.10	0.0359	-	-	0.0736	0.0863	10	
16	Vertical	9.9	0.0622	-	-	353.6	2.48	8.11	0.0229	0.0070	9.34	0.0299	-	-	0.0657	0.0886	11	
17	Vertical	9.8	0.0624	-	-	353.6	2.48	8.86	0.0251	0.0070	9.27	0.0321	-	-	0.0659	0.0910	12	
18	Vertical	9.7	0.0627	-	-	353.6	2.48	9.61	0.0272	0.0070	9.20	0.0342	-	-	0.0662	0.0934	13	
19	Vertical	9.7	0.0629	-	-	353.6	6.70	7.50	0.0212	0.0189	8.21	0.0401	-	-	0.0724	0.0936	14	
19	Horizontal	9.7	0.0629	-	-	353.6	5.62	9.00	0.0254	0.0160	8.43	0.0414	-	-	0.0709	0.0963	15	
20	Horizontal	9.6	0.0631	-	-	353.6	6.03	9.42	0.0266	0.0171	8.30	0.0437	-	-	0.0717	0.0983	16	
22	Horizontal	9.5	0.0635	-	-	353.6	6.02	10.25	0.0290	0.0171	8.19	0.0461	-	-	0.0721	0.1011	17	

*Parameters used to generate pressure-time input for loading mass points.

Table 19
Blast loading data for DYNFA analysis of frame on column line 2, Example A.4

Mode no. of mass point	Direction of load	Blast loading parameters at mass points					Parameters used for modifying pressure waveforms					Waveform I.D. No.					
		P _{pk} (kPa)	t _{dr} (sec)	P _s (kPa)	t _c (sec)	U (m/sec)	a, D (m)	a, D (m)	t _a [*] (sec)	t _{rt} (sec)	(P _{pk}) [*] / AVG (kPa)		t _{pk} [*] (sec)	P _s (kPa)	t _c [*] (sec)	t _{DI} (sec)	t _{tt} (sec)
2	Horizontal	23.3	0.0595	3.29	0.0424	353.6	3.00	0.00	0.0000	0.0085	21.30	0.0085	3.29	0.0467	0.0638	0.0680	1
4	Horizontal	23.3	0.0595	3.29	0.0424	353.6	3.00	0.00	0.0000	0.0085	21.30	0.0085	3.29	0.0467	0.0638	0.0680	1
5	Horizontal	23.3	0.0595	3.29	0.0424	353.6	3.00	0.00	0.0000	0.0085	21.30	0.0085	3.29	0.0467	0.0638	0.0680	1
5	Vertical	10.9	0.0595	-	-	353.6	3.87	0.00	0.0000	0.0109	9.87	0.0109	-	-	0.0650	0.0650	2
6	Vertical	10.7	0.0599	-	-	353.6	4.73	0.87	0.0025	0.0134	9.49	0.0159	-	-	0.0666	0.0691	3
8	Vertical	10.5	0.0604	-	-	353.6	4.73	2.60	0.0074	0.0134	9.34	0.0208	-	-	0.0671	0.0745	4
9	Vertical	10.3	0.0608	-	-	353.6	4.73	4.33	0.0122	0.0134	9.19	0.0256	-	-	0.0675	0.0797	5
11	Vertical	10.2	0.0612	-	-	353.6	4.73	6.06	0.0171	0.0134	9.04	0.0305	-	-	0.0679	0.0850	6
13	Vertical	10.0	0.0617	-	-	353.6	4.73	7.79	0.0220	0.0134	8.90	0.0354	-	-	0.0684	0.0904	7
14	Vertical	9.8	0.0621	-	-	353.6	4.73	9.53	0.0270	0.0134	8.74	0.0404	-	-	0.0688	0.0958	8
14	Horizontal	9.8	0.0621	-	-	353.6	3.72	10.36	0.0292	0.0105	8.97	0.0397	-	-	0.0674	0.0966	9
15	Horizontal	9.7	0.0625	-	-	353.6	4.45	11.11	0.0314	0.0126	8.68	0.0440	-	-	0.0688	0.1002	10
17	Horizontal	9.5	0.0628	-	-	353.6	4.45	12.56	0.0355	0.0126	8.56	0.0481	-	-	0.0691	0.1046	11

*Parameters used to generate pressure-time input for loading mass points.

Table 20

Dead loads applied to elements of model
of frame on column line B, Example A.4

Element number	Uniform load (kg/m)	Element number	Uniform load (kg/m)
1	116.1	12	81.2
2	116.1	13	81.2
3	116.1	14	165.2
4	116.1	15	81.2
5	81.2	16	81.2
6	81.2	17	81.2
7	81.2	18	81.2
8	81.2	19	116.1
9	165.2	20	116.1
10	81.2	21	116.1
11	81.2	22	116.1

Table 21

Dead loads applied at nodal points of model
of frame on column line B, Example A.4

Nodal point	Item	Load (kg)
5, 19	Roof decking	415.8
	Wall panel	539.7
	Transverse girder	<u>285.9</u>
	Total	1,241.4
9, 14	Roof decking: 2 panels	831.6
	Transfer girder	<u>321.6</u>
	Total	1,153.2

Table 22
 Tabulation of maximum chordal angles for exterior members, Example A.5

Member	Maximum differential displacement at midspan (mm)	Length of member (m)	Maximum chordal angle (deg)	Chordal angle criteria θ_{max} (deg)
Blastward column	24.37	5.0	0.560	1.0
Blastward end roof girder	12.70	6.0	0.243	1.0
Leeward end roof girder	18.50	6.0	0.359	1.0
Leeward column	11.70	5.0	0.268	1.0

Table 23
Tabulation of maximum ductility ratios
for
members of frame on column line 2, Example A.5

Member	Node	Element	Max. computed ductility ratio (μ)	P/P _p	Nature of applied axial load	Response time (sec)	Ductility criteria (μ_{max})	Applicable figure of text
Blastward column	1	1	1.000	-	-	-	-	-
	2	1	2.567	0.064	compression	0.020	1.65	43
	3	2	2.080	0.064	compression	0.020	1.65	43
	4	3 or 4	1.000	-	-	-	-	-
	5	4	1.000	-	-	-	-	-
Girder at blastward end of frame	5	5	1.539	0.143	compression	0.014	2.70	43
	6	5 or 6	1.000	-	-	-	-	-
	7	6 or 7	1.000	-	-	-	-	-
	8	7 or 8	1.000	-	-	-	-	-
	9	8	1.558	0.011	tension	0.046	3.00*	-
Interior column	9	9	1.000	-	-	-	-	-
	10	9	1.000	-	-	-	-	-
Girder at leeward end of frame	9	10	1.493	0.114	compression	0.070	3.00	40
	11	10 or 11	1.000	-	-	-	-	-
	12	11 or 12	1.000	-	-	-	-	-
	13	12 or 13	1.000	-	-	-	-	-
	14	13	2.212	0.022	compression	0.055	2.88	43
Leeward column	14	14	1.000	-	-	-	-	-
	15	14 or 15	1.000	-	-	-	-	-
	16	15 or 16	1.000	-	-	-	-	-
	17	16 or 17	1.000	-	-	-	-	-
	18	17	1.000	-	-	-	-	-

* μ_{max} for this case given in Section 6.3.

Table 24

Tabulation of R_c for $P/P_D = 0.064$ and several values of DF_{AB} and l/r_x for blastward column, Example A.5

Figure	Distribution factor, DF_{AB}	Rotation capacities		
		$l/r_x = 20$	$l/r_x = 40$	$l/r_x = 60$
51	0.25	15.0	9.3	4.6
52	0.50	6.7	3.5	2.0
53	0.75	2.3	1.4	1.0

Table 25

Tabulation of maximum applied shears
and
shear yield capacities for frame members, Example A.5

Member	Size	Section dimensions*			Web area (mm ²)	V _p (kN)	V _{max} (kN)	Element	Node
		d (mm)	t _f (mm)	t _w (mm)					
Blastward column	W14 x 53	354.1	16.7	9.4	3,015	452.3	35	4	5
Blastward end girder	W12 x 40	303.3	13.1	7.5	2,067	310.1	158	8	9
Leeward end girder	W12 x 40	303.3	13.1	7.5	2,067	310.1	193	13	14
Leeward column	W14 x 53	354.1	16.7	9.4	3,015	452.3	210	14	14
Interior column	W14 x 61	353.3	16.3	9.6	3,079	461.9	84	9	10

* d, t_f and t_w for frame members given in "Properties for designing" tables of Reference 14.

Table 26
 Tabulation of bi-axial bending moments
 and
 calculation of nodal reduction factors
 for
 column B-1, Example A.6

Frame	Element	Node	$ M_x $	M_{mx}	$ M_x /M_{mx}$
Column line B	1	2	351.8	599.5	0.59
	2	3	319.9	599.5	0.53
	3	4	288.1	599.5	0.48
	4	5	159.4	599.5	0.27

Frame	Element	Node	$ M_y $	M_{my}	$ M_y /M_{my}$	R_η [Eq (32)]
Column line 1	9	-	$1/3 M_y _9$	234.4	0.17	0.76
		-	$1/2 M_y _9$	234.4	0.26	0.79
		-	$2/3 M_y _9$	234.4	0.34	0.82
		9	120.3	234.4	0.51	0.78

Table 27

Calculation of nodal reduction factors and reduced bi-axial bending capacities for column B-1: W14x61

Frame	Element	Node	$ M_x $	M_{mx}	$ M_x /M_{mx}$
Column line B	1	2	351.8	456.3	0.77
	2	3	319.9	456.3	0.70
	3	4	288.1	456.3	0.63
	4	5	159.4	456.3	0.35

Frame	Element	Node	$ M_y $	M_{my}	$ M_y /M_{my}$	R_n [Eq (32)]
Column line 1	9	-	$1/3 M_y _g$	146.3	0.27	1.04
		-	$1/2 M_y _g$	146.3	0.41	1.11
		-	$2/3 M_y _g$	146.3	0.55	1.18
		9	120.3	146.3	0.82	1.17

Frame	Element	Node	R_n	M_{mx} [Eq (33a)]
Column line B	1	1	-	-
		2	1.04	338.3
	2	2	1.04	338.3
		3	1.11	288.2
	3	3	1.11	288.2
		4	1.18	244.1
	4	5	1.18	244.1
		5	1.17	136.2

Frame	Element	Node	R_n	M_{my} [Eq (33b)]
Column line 1	9	10	-	-
		9	1.17	102.8

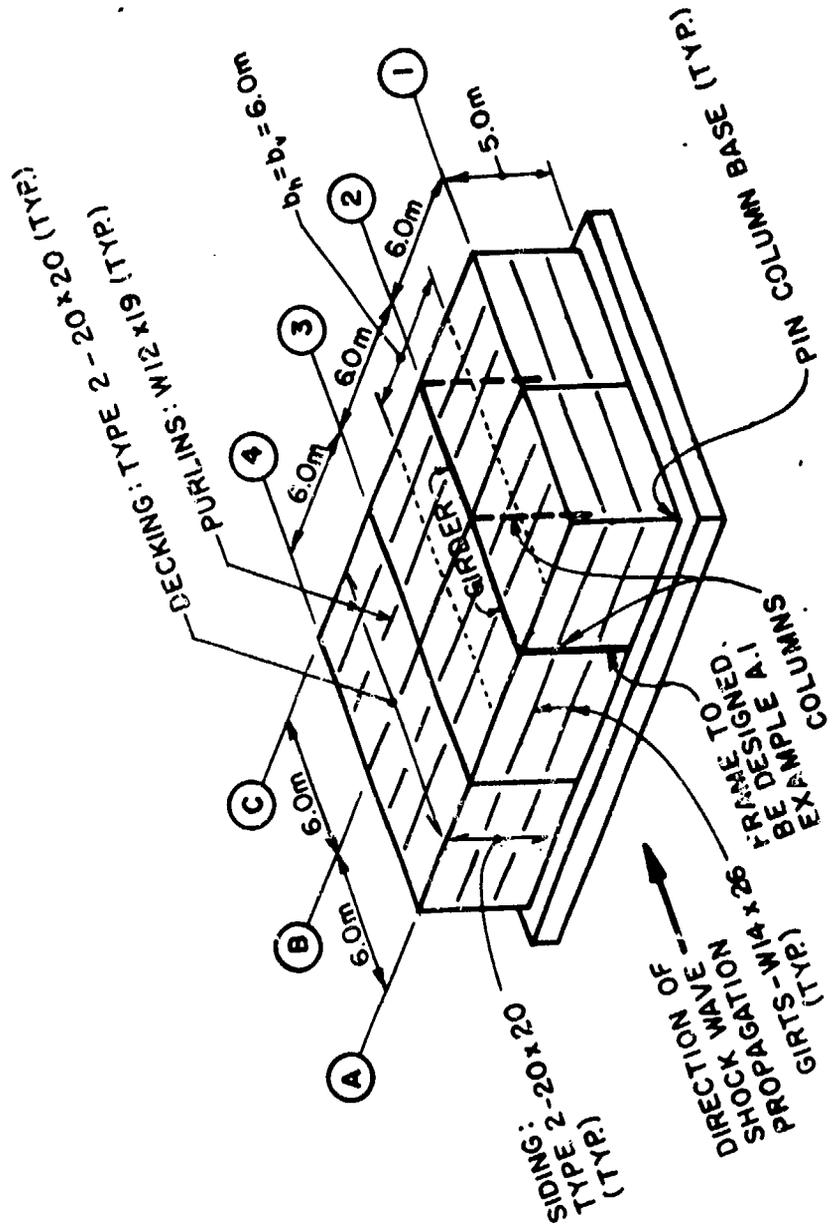


Fig 1 Building designed in Examples A.1 through A.6

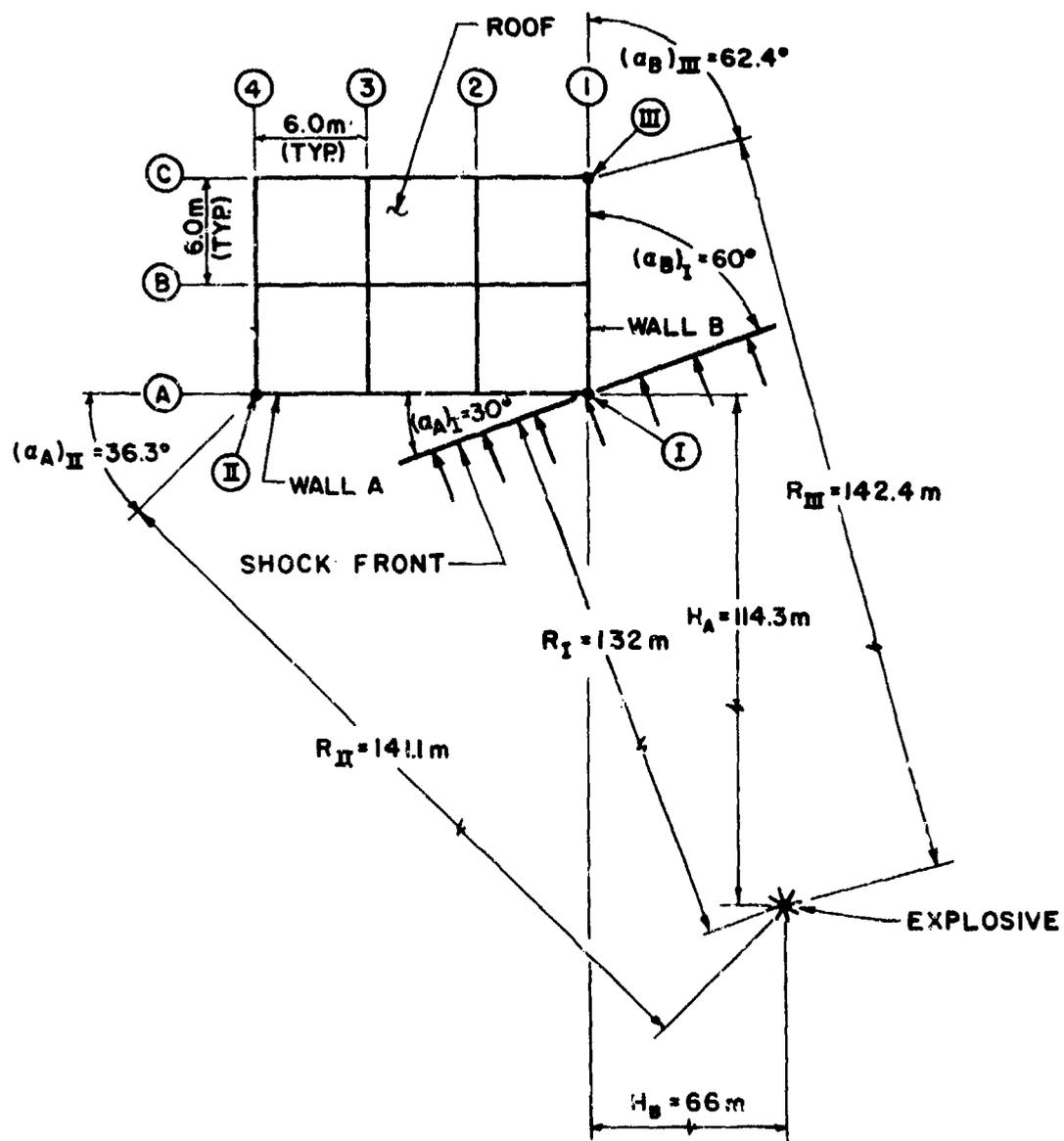


Fig 2 Location of charge: quartering load, Example A.2

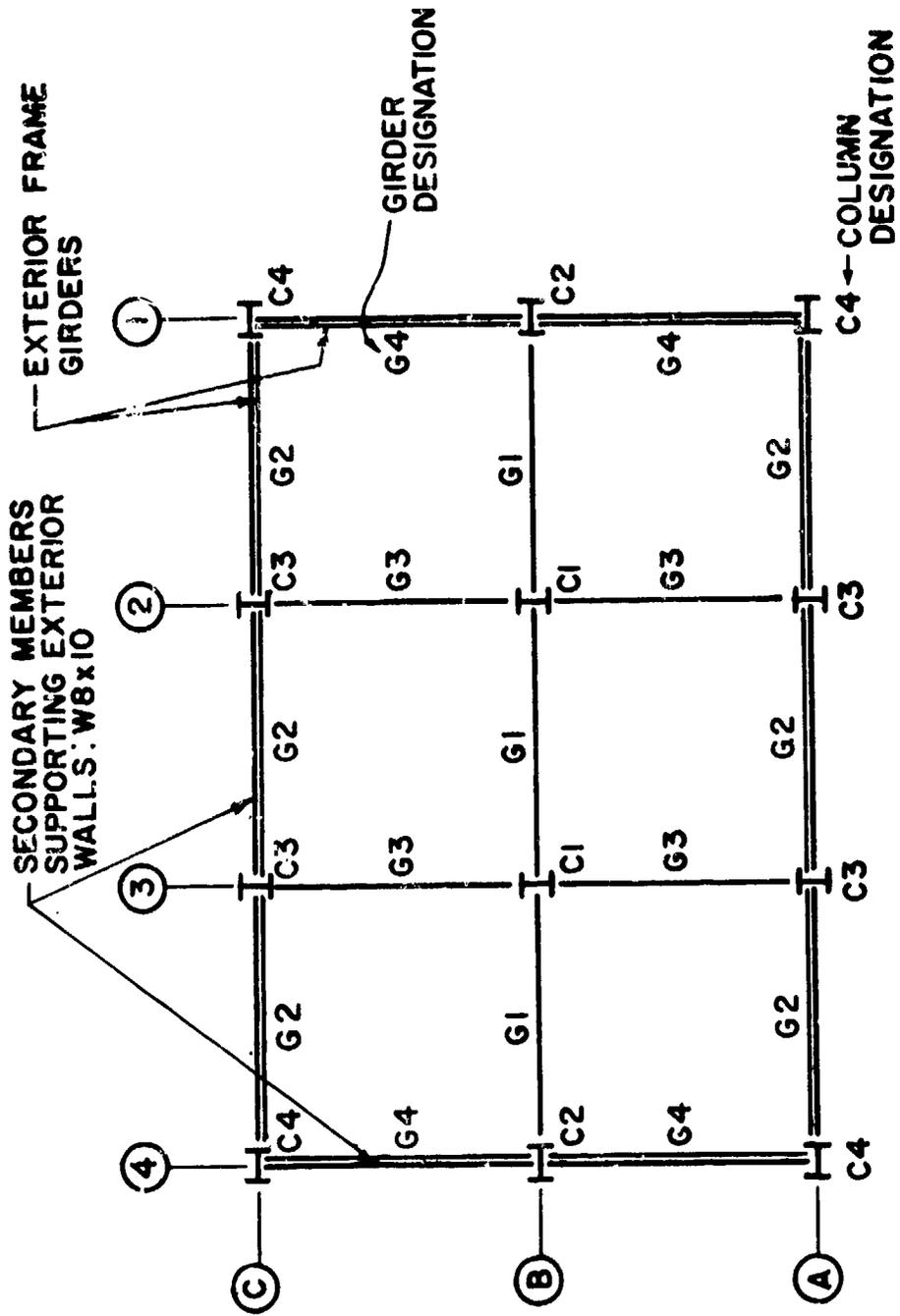
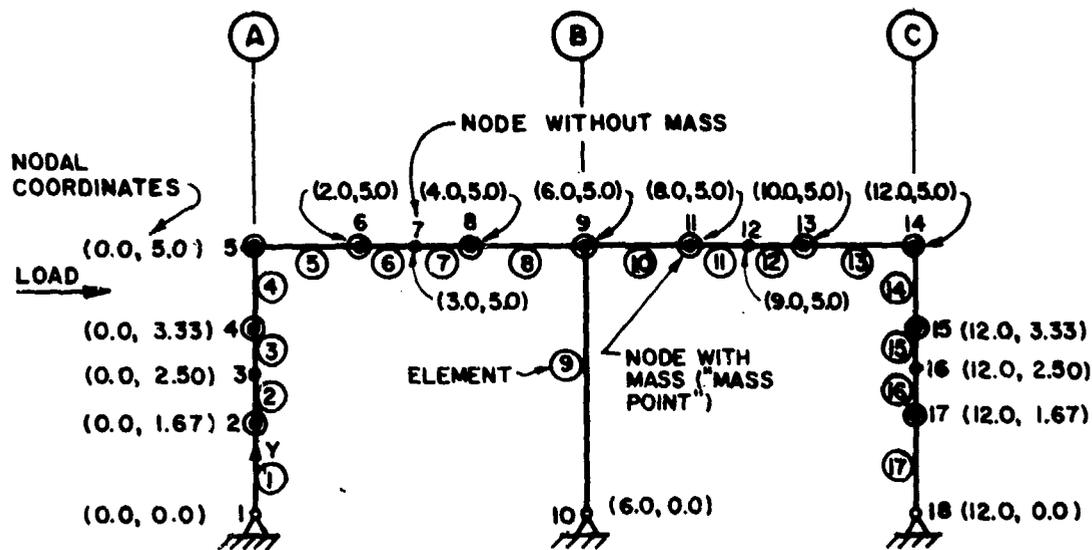
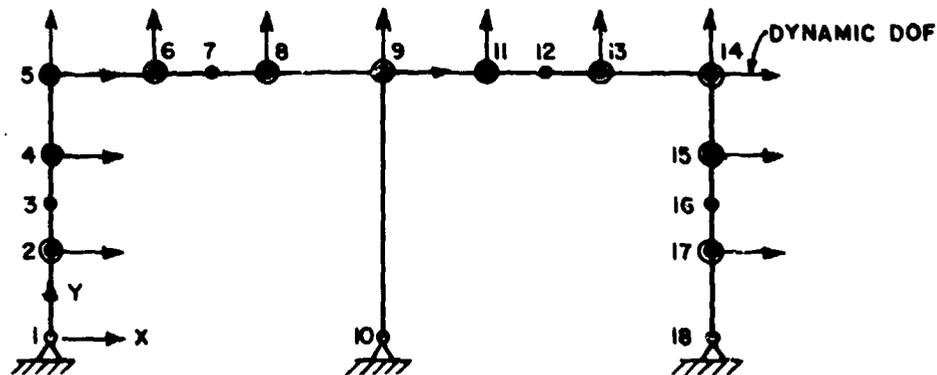


Fig 3 Primary framing plan of building



a) NODAL POINT AND ELEMENT ID NUMBERS AND NODAL COORDINATES FOR MODEL



b) DYNAMIC DOFS FOR MODEL

Fig 4 Analytical model of frame on column line 2

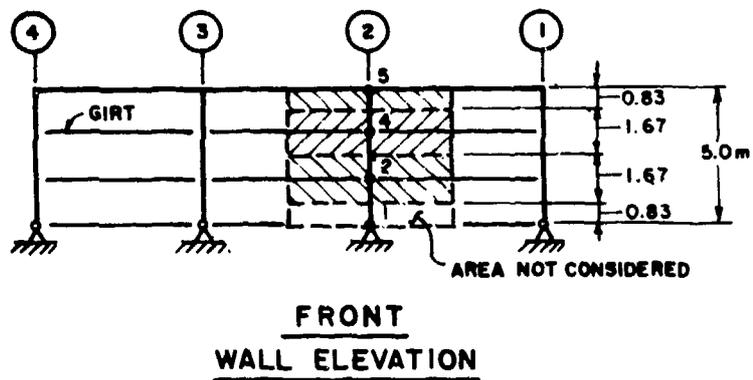
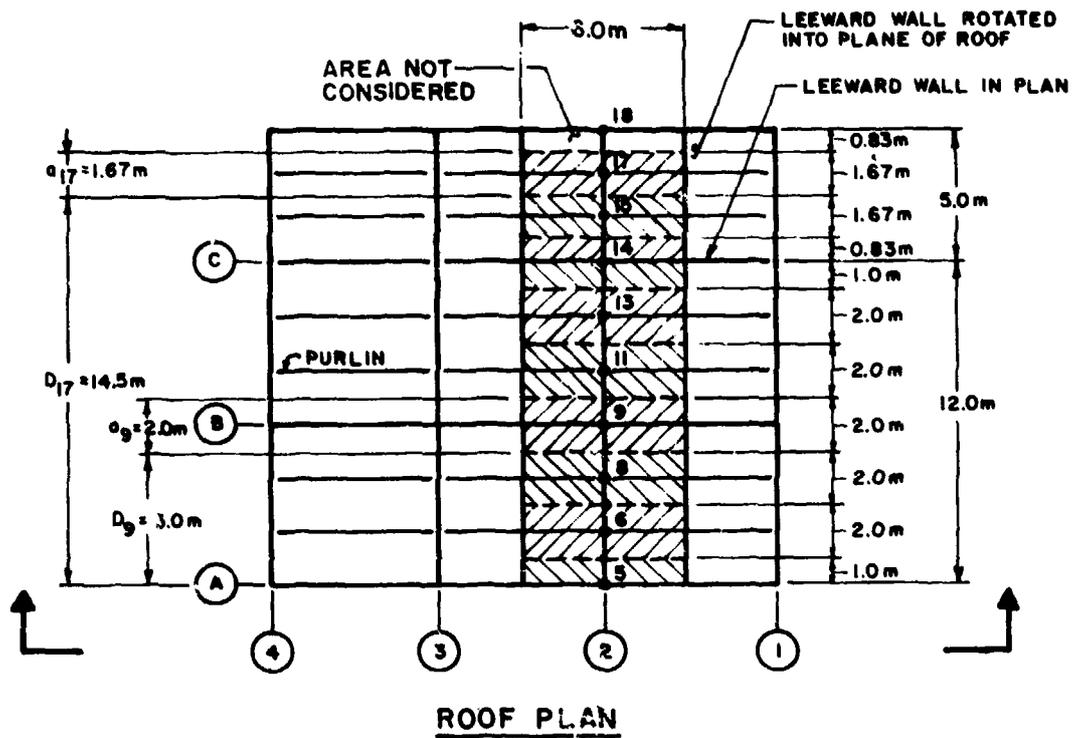
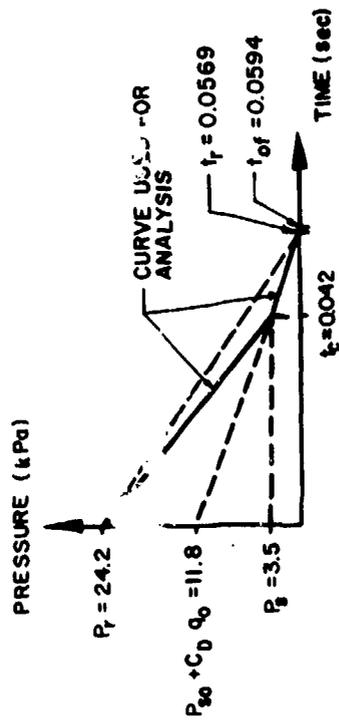
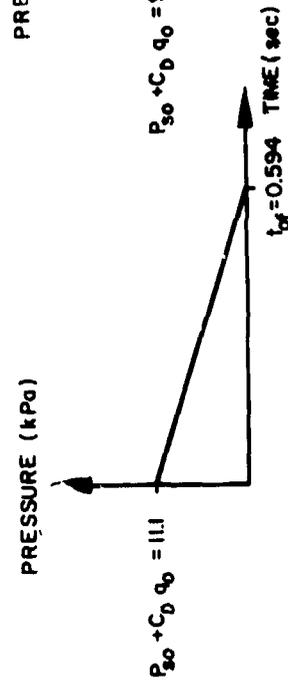


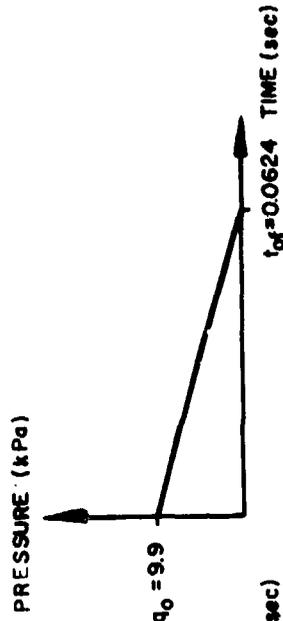
Fig 5 Tributary areas for model of frame on column line 2



a) BLASTWARD WALL

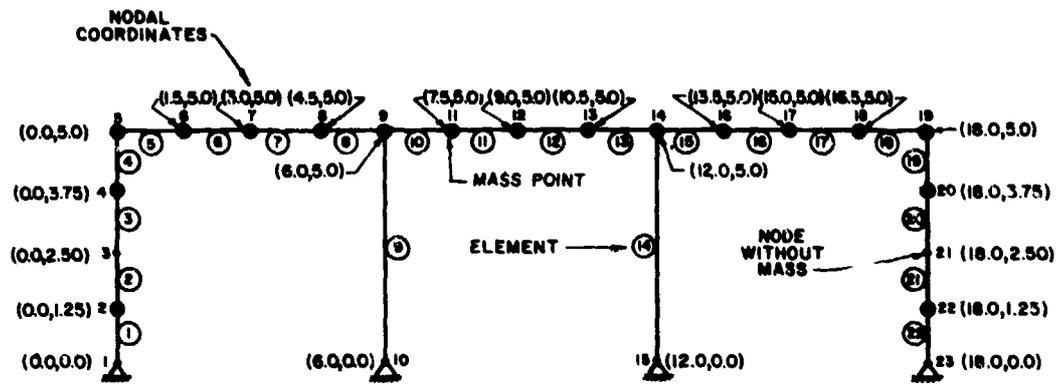


b) BLASTWARD END OF ROOF

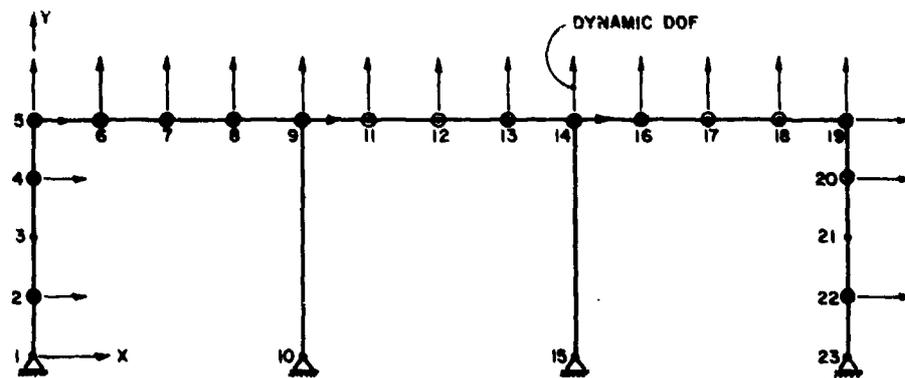


c) LEEWARD END OF ROOF

Fig 6 Blastward wall and roof pressure-time histories, Examples A.3



a) NODAL POINT AND ELEMENT ID NUMBERS AND NODAL COORDINATES FOR MODEL



b) DYNAMIC DOFS FOR MODEL

Fig 7 Analytical model of frame on column line B

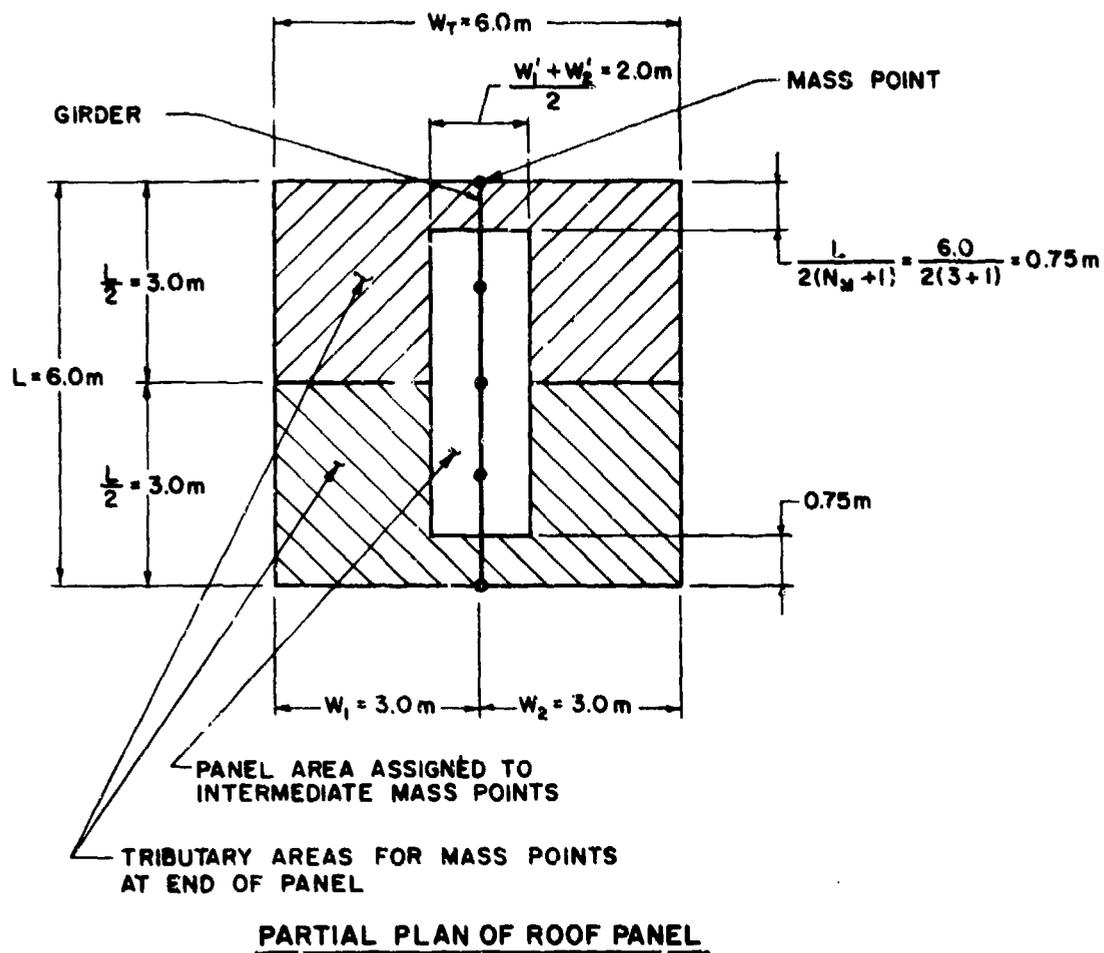
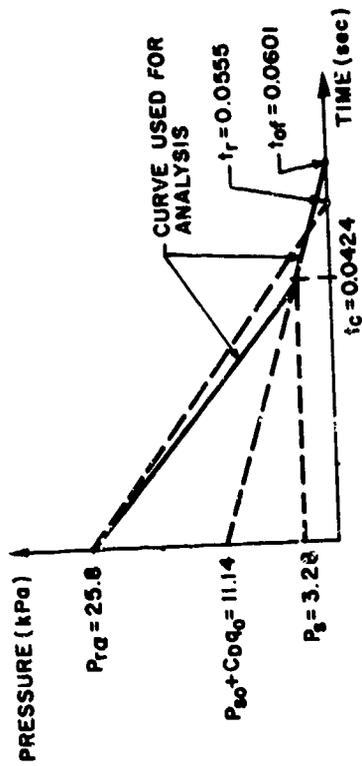
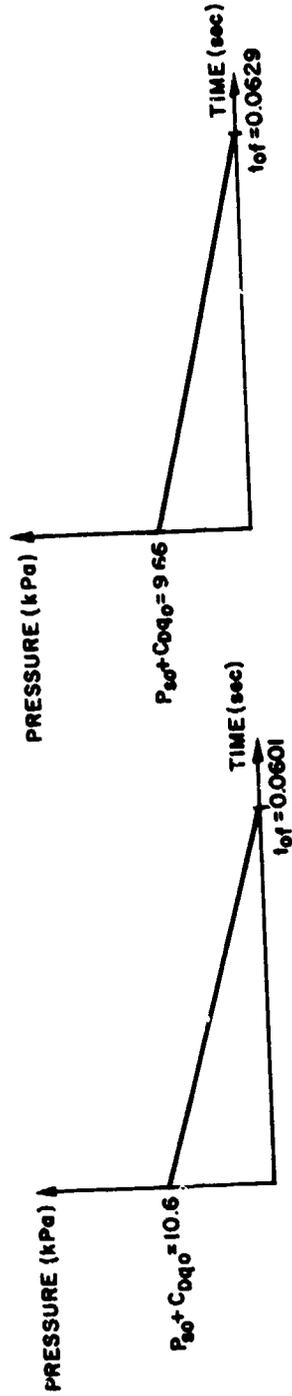


Fig 8 Subdivision of area of roof panel for model of frame on column line 8



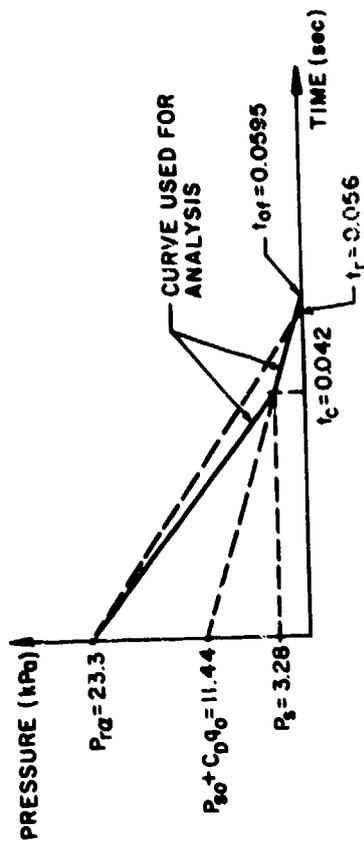
a) BLASTWARD WALL



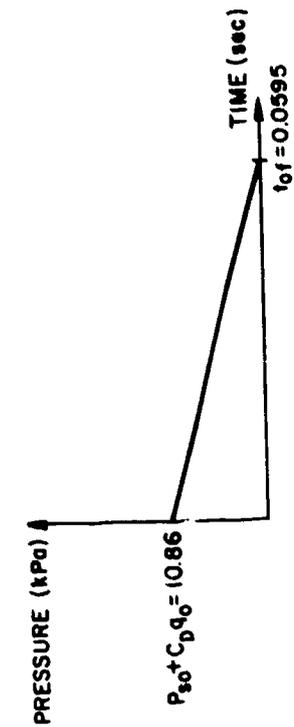
b) BLASTWARD END OF ROOF

c) LEEWARD END OF ROOF

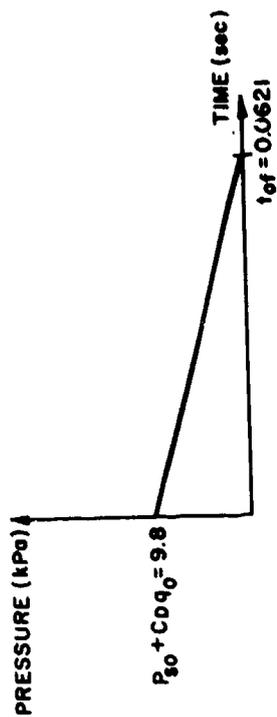
Fig 10 Blastward wall and roof pressure-time histories for model of frame on column line B, Example A.4



a) BLASTWARD WALL



b) BLASTWARD END OF ROOF



c) LEEWARD END OF ROOF

Fig 11 Blastward wall and roof pressure-time histories for model of frame on column line 2, Example A.4

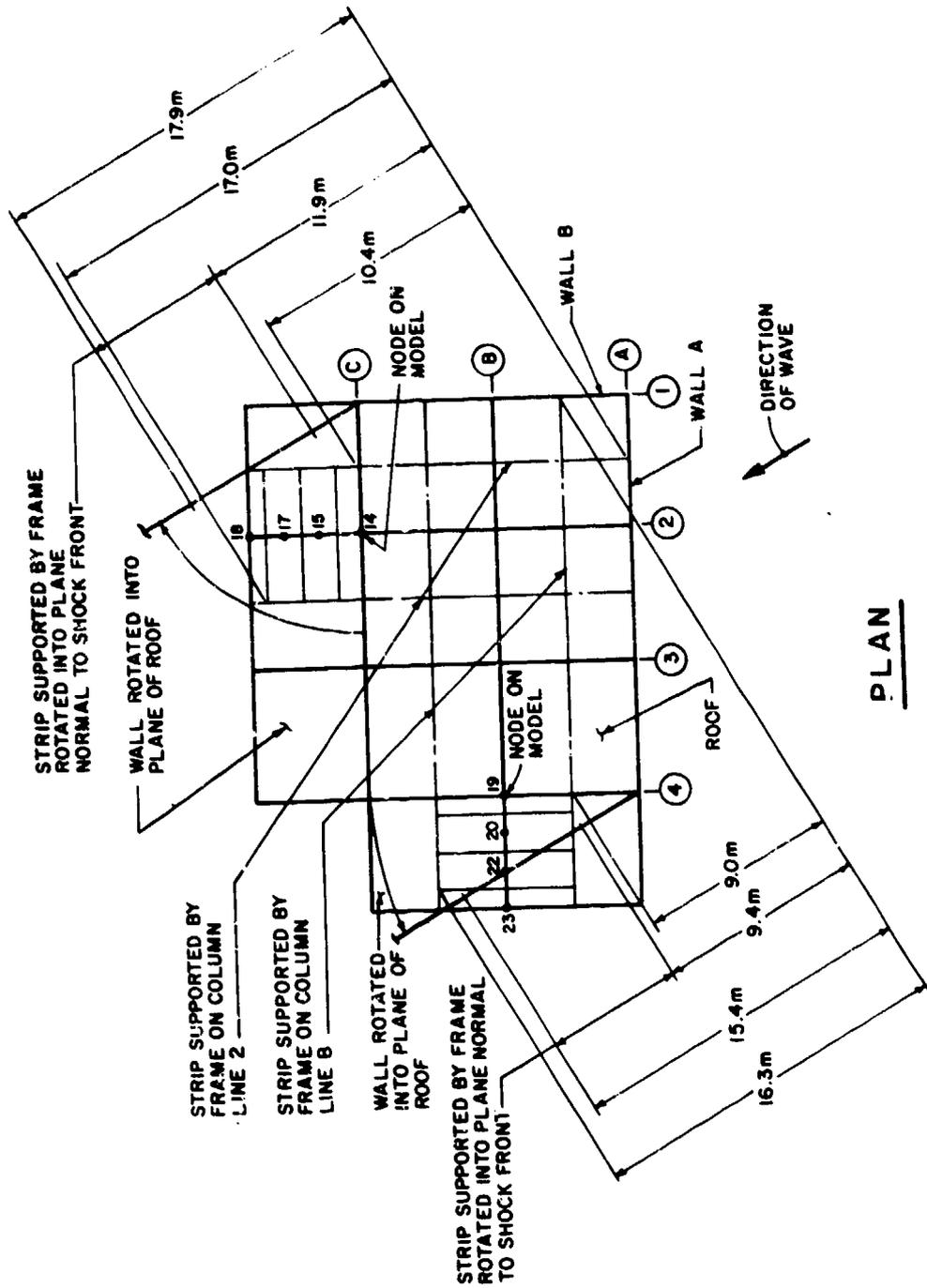


Fig 12 Measurement of distances to leeward walls traveled by quartering blast wave, Example A.4

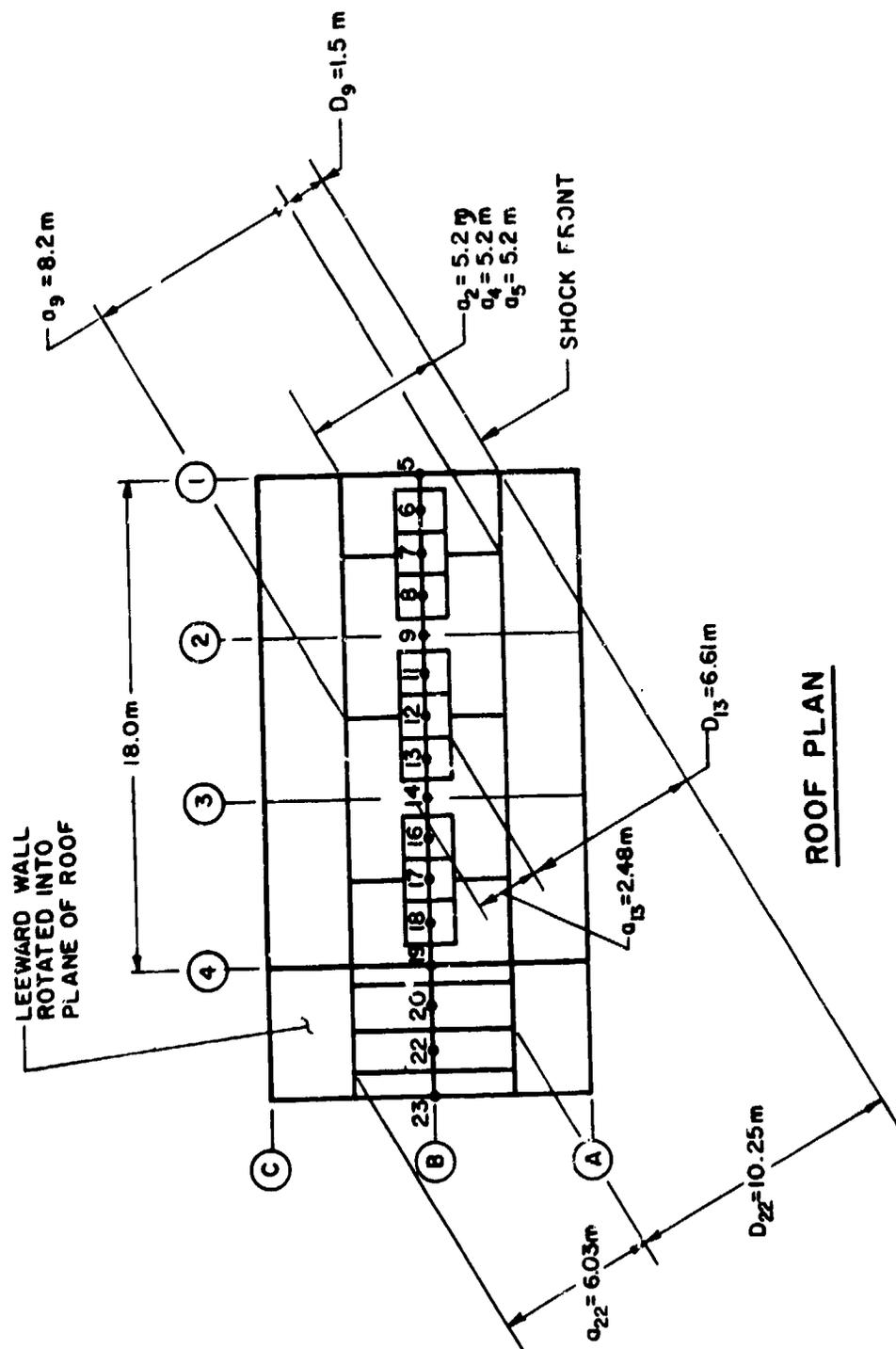


Fig 13 Measurement of parameters α and D for selected mass points on model of frame on column line B, Example A.4

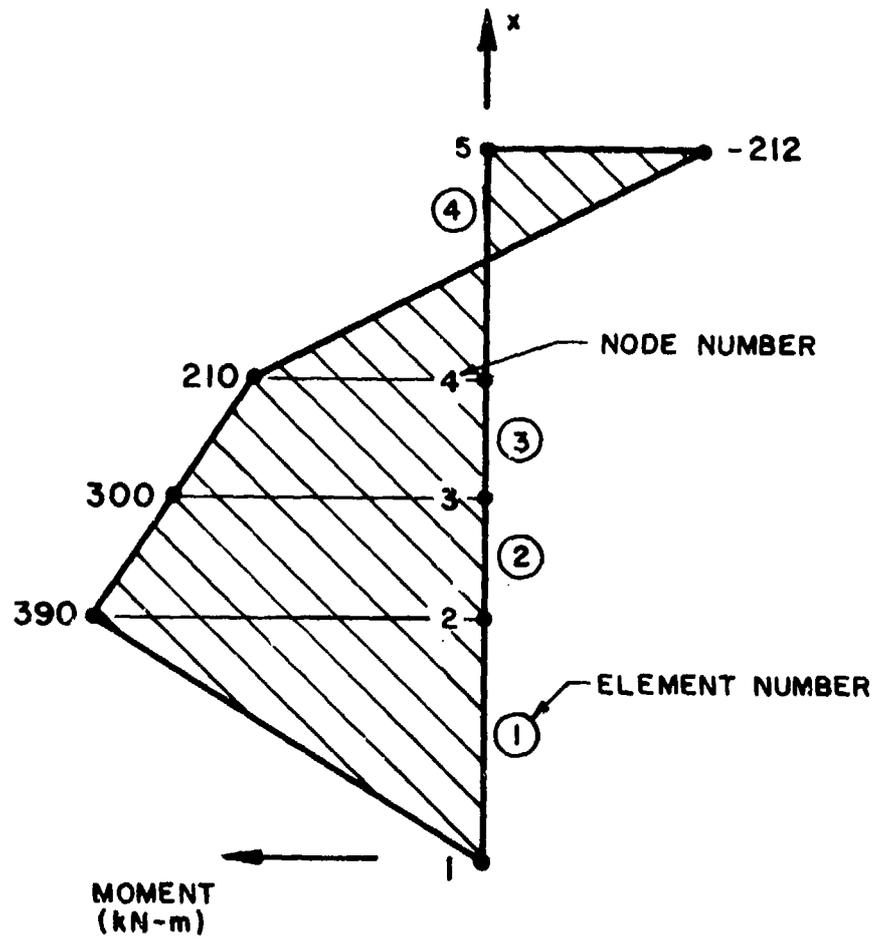


Fig 14 Bending moment diagram for exterior blastward column at $t = 0.0175$ second, Example A.5

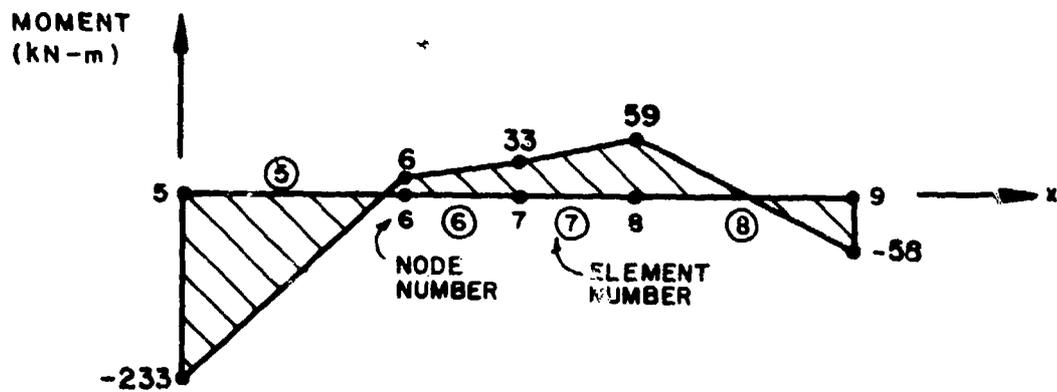
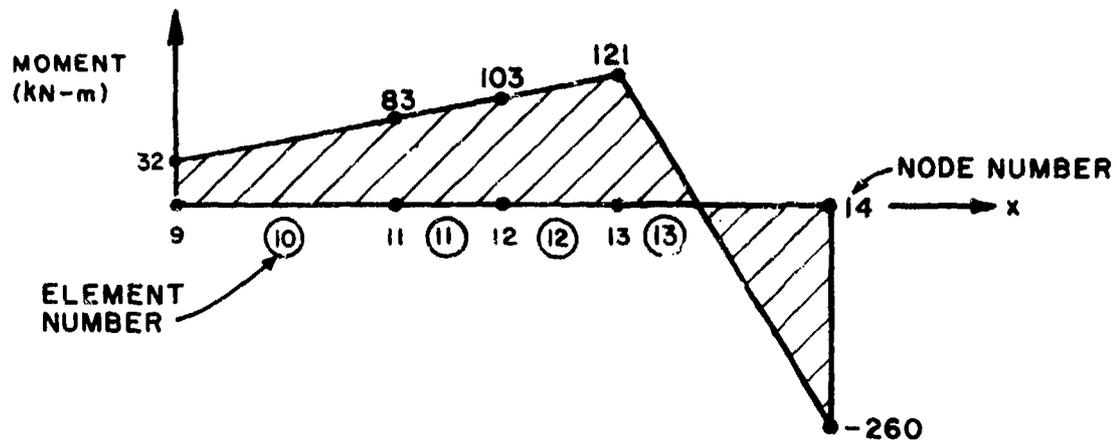
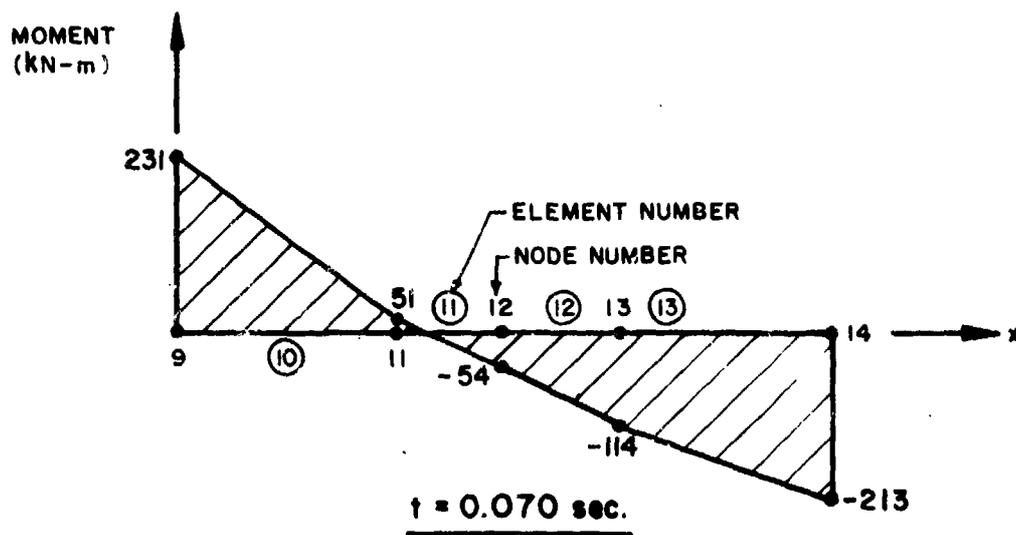


Fig 15 Bending moment diagram for blastward girder at $t = 0.0140$ second, Example A.5

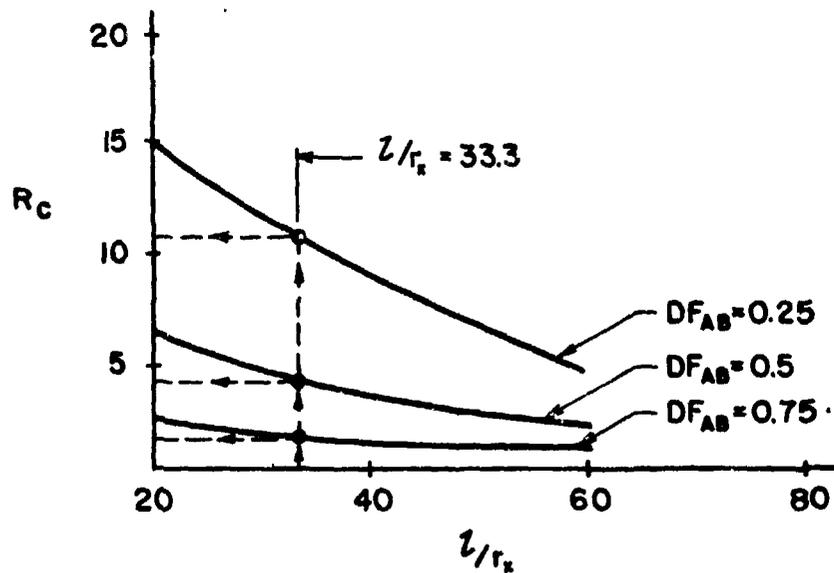


t = 0.055 sec.

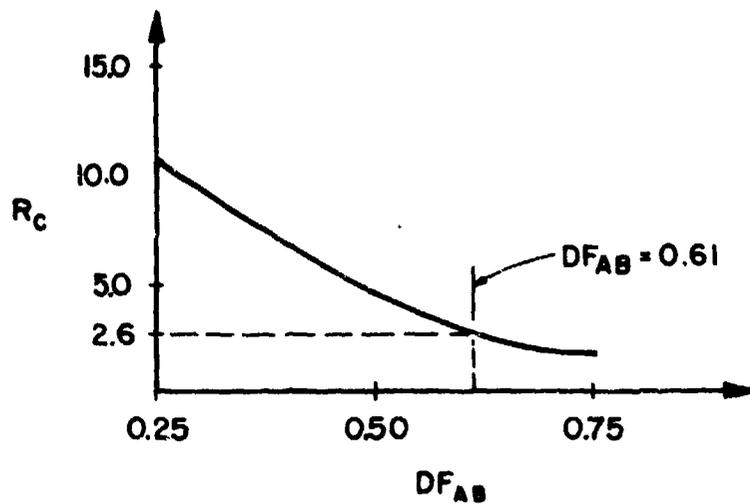


t = 0.070 sec.

Fig 16 Bending moment diagrams for leeward girder, Example A.5



a) R_c vs l/r_x FOR $P/P_p = 0.064$ & $DF_{AB} = 0.25, 0.50, 0.75$



b) R_c vs. DF_{AB} FOR $l/r_x = 33.3$

Fig 17 Interpolation for R_c for exterior blastward column, Example A.5

APPENDIX B

SAMPLES OF INPUT AND OUTPUT OF DYNFA

APPENDIX B
SAMPLES OF INPUT AND OUTPUT OF DYNFA

8.1 Introduction

This appendix contains samples of the input and output of DYNFA. The input is presented in terms of a listing of the input data deck for Example A.5 of Appendix A. The listing includes subtitles designating the various input cards as well as the various input parameters. Following this are extracts from the printed output of DYNFA for the same problem.

COLUMN	1	10	20	30	40	50	60	70	80

CARD TYPE 6 ELEMENT CARD (ONE CARD REQUIRED FOR EACH BEAM)									
ELEM	JA	JB	PIN	A	I	M4A	M4B	PP	PU
17			AS						
1	1	2	00	101.0	22550.0	390.0		2750.0	2370.0
2	2	3	00	101.0	22550.0	390.0		2750.0	2370.0
3	3	4	00	101.0	22550.0	390.0		2750.0	2370.0
4	4	5	00	101.0	22550.0	390.0		2750.0	2370.0
5	5	6	00	76.1	12834.0	254.0		2078.0	1867.0
6	6	7	00	76.1	12834.0	254.0		2078.0	1867.0
7	7	8	00	76.1	12834.0	254.0		2078.0	1867.0
8	8	9	00	76.1	12834.0	254.0		2078.0	1867.0
9	10	9	00	115.0	26670.0	415.0		3155.0	2222.0
10	9	11	00	76.1	12834.0	254.0		2078.0	1867.0
11	11	12	00	76.1	12834.0	254.0		2078.0	1867.0
12	12	13	00	76.1	12834.0	254.0		2078.0	1867.0
13	13	14	00	76.1	12834.0	254.0		2078.0	1867.0
14	15	14	00	101.0	22550.0	390.0		2750.0	2370.0
15	16	15	00	101.0	22550.0	390.0		2750.0	2370.0
16	17	16	00	101.0	22550.0	390.0		2750.0	2370.0
17	18	17	00	101.0	22550.0	390.0		2750.0	2370.0

CARD TYPE 7 DAMPING (REQUIRED)
DAMP
0.1

CARD TYPE 8 INTEGRATION TIME CARD (REQUIRED)
DT DT NSKIP
510 0.00035 10

CARD TYPE 9 LOADING SPECIFICATION CARD
NWF
11

CARD TYPE 10 WAVEFORM SPECIFICATION CARD AND
CARD TYPE 11 P-T CARD
WF NO NPOINT
1 4
PI TI
24.2 0.0
3.5 0.0420
0.0 0.0594
0.0 1.0
2 4
0.0 0.0
10.4 0.0028
0.0 0.0608
0.0 1.0

COLUMN	1	10	20	30	40	50	60	70	80
--------	---	----	----	----	----	----	----	----	----

COLUMN	1	10	20	30	40	50	60	70	80
		WF NO	NPOINT						
		3	5						
		PI	TI						
		0.0	0.0						
		0.0	0.0028						
		10.4	0.0084						
		0.0	0.0655						
		0.0	1.0						
		4	5						
		0.0	0.0						
		0.0	0.0085						
		10.2	0.0141						
		0.0	0.0717						
		0.0	1.0						
		5	5						
		0.0	0.0						
		0.0	0.0141						
		10.0	0.0197						
		0.0	0.0778						
		0.0	1.0						
		6	5						
		0.0	0.0						
		0.0	0.0197						
		9.83	0.0253						
		0.0	0.0839						
		0.0	1.0						
		7	5						
		0.0	0.0						
		0.0	0.0254						
		9.64	0.0310						
		0.0	0.0901						
		0.0	1.0						
		8	5						
		0.0	0.0						
		0.0	0.0311						
		9.67	0.0339						
		0.0	0.0949						
		0.0	1.0						
		9	5						
		0.0	0.0						
		0.0	0.0339						
		9.71	0.0362						
		0.0	0.0975						
		0.0	1.0						
		10	5						
		0.0	0.0						
		0.0	0.0363						
		9.34	0.0410						
		0.0	0.1015						
		0.0	1.0						

COLUMN 1 10 20 30 40 50 60 70 80

.....*

WF 40 NPOINT

11 5

PI T1

0.0 0.0

0.0 0.0410

9.15 0.0457

0.0 0.1066

0.0 1.0

CARD TYPE 12 TRIJUTARY AREA CARD (ENTER CARDS ONLY FOR MASS POINTS
SUBJECTED TO BLAST LOADS)

NODE NO	AFX	INDX	AFY	INDY
2	10.0	1	0.0	0
4	10.0	1	0.0	0
5	5.0	1	-6.0	2
6	0.0	0	-12.0	3
8	0.0	0	-12.0	4
9	0.0	0	-12.0	5
11	0.0	0	-12.0	6
13	0.0	0	-12.0	7
14	-5.0	9	-6.0	8
15	-10.0	10	0.0	0
17	-10.0	11	0.0	0

CARD TYPE 13 ELEMENT UNIFORM LOADS DIRECTION INDICATOR CARD (REQUIRED)

ADIR
-Y

CARD TYPE 14 ELEMENT UNIFORM STATIC LOAD CARD (ONE CARD REQ'D FOR EACH ELEMENT)

ELEM NO	WUNIF
1	78.9
2	78.9
3	78.9
4	78.9
5	59.4
6	59.4
7	59.4
8	59.4
9	90.5
10	59.4
11	59.4
12	59.4
13	59.4
14	78.9
15	78.9
16	78.9
17	78.9

COLUMN 1 10 20 30 40 50 60 70 80

.....*

COLUMN	1	10	20	30	40	50	60	70	80
.....*									
CARD TYPE 15 STATIC NODAL LOADS CARD CENTER CARDS ONLY FOR THOSE NODES									
AT WHICH LOADS ARE ACTING)									
NODE NO		FX		FY		MZ			
5		0.0		-984.3		0.0			
6		0.0		-415.2		0.0			
8		0.0		-415.2		0.0			
9		0.0		-367.6		0.0			
11		0.0		-415.2		0.0			
13		0.0		-415.2		0.0			
14		0.0		-984.3		0.0			

END OF DATA INDICATOR
END

B.3 Portions of Printed Output for Example A.5

EXAMPLE A.5 TWO-BAY RIGID FRAME SUBJECTED TO BLAST LOAD

UNITS-METRIC

BASIC UNITS
LENGTH - METERS
MASS - KILOGRAMS
TIME - SECONDS
UNITS FOR ELEMENT PROPERTIES, CAPACITIES AND LOADS
AREA - SQUARE CENTIMETERS(CM**2)
MOMENT OF INERTIA - CENTIMETERS(CM**4)
AXIAL LOAD AND SHEAR - KILONEWTONS(KN)
BENDING MOMENT - KILONEWTON-METER(KN-M)
APPLIED LOADS
PRESSURE - KILOPASCALS(KPA)
TRIBUTARY AREA - SQUARE METERS(M**2)
UNIFORM LOADS - KILOGRAMS/METER(M)
STATIC LOADS AT NODES - KILOGRAMS(KG)
STATIC MOMENTS AT NODES - KILOGRAM-METER(KG-M)
CONSTANTS
ACCELERATION OF GRAVITY - 9.8 METERS PER SECOND SQUARED
MODULUS OF ELASTICITY - KILOPASCALS(KPA)

NOTE: - APPLIED STATIC LOADS IN MASS UNITS
REFLECTIONS
MODAL DISPLACEMENTS - MILLIMETERS(MM)
MODAL AND ELEMENT END ROTATIONS - DEGREES(DDG)

EXAMPLE A-5 TWO-BAY RIGID FRAME SUBJECTED TO BLAST LOAD

NUMBER OF ELEMENTS = 17
 NUMBER OF NODES = 18
 NUMBER OF NODES WITH RESTRAINTS = 3
 NUMBER OF NODES WITH MASSES = 11
 NUMBER OF NODES WITH FORCES = 11
 DISPLACEMENTS PRINTING INDEX = 1
 NOMINAL L-SHEAR PRINTING INDEX = 1
 MODULUS OF ELASTICITY = 207300000 KPA
 PERCENTAGE OF ELASTIC STIFFNESS AFTER YIELD = .051

*** NODAL COORDINATE TABLE ***

NODE	X (M)	Y (M)
1	0.00	0.00
2	0.00	1.67
3	0.00	2.58
4	0.00	3.13
5	1.00	0.00
6	1.00	1.67
7	1.00	2.58
8	1.00	3.13
9	2.00	0.00
10	2.00	1.67
11	2.00	2.58
12	2.00	3.13
13	10.00	0.00
14	12.00	0.00
15	12.00	1.67
16	12.00	2.58
17	12.00	1.67
18	12.00	0.00

NODAL RESTRAINT TABLE

NODE	X	Y	ROTATION
1	1	1	1
2	1	1	1
3	1	1	1
4	1	1	1
5	1	1	1
6	1	1	1
7	1	1	1
8	1	1	1
9	1	1	1
10	1	1	1
11	1	1	1
12	1	1	1
13	1	1	1
14	1	1	1
15	1	1	1
16	1	1	1
17	1	1	1
18	1	1	1

INPUT NODAL MASS TABLE

NODE	MX (KG)	MY (KG)
1		
2		
3		
4		
5		
6		
7		
8		
9		
10		
11		
12		
13		
14		
15		
16		
17		
18		

2 500.71 6.03
 3 500.71 6.03
 4 1200.30 12.06
 5 500.71 6.03
 6 500.71 6.03
 7 500.71 6.03
 8 500.71 6.03
 9 500.71 6.03
 10 500.71 6.03
 11 500.71 6.03
 12 500.71 6.03
 13 500.71 6.03
 14 500.71 6.03
 15 500.71 6.03
 16 500.71 6.03
 17 500.71 6.03

ELEMENT TABLE

ELEM	JA	JB	JIN CODE	AREA (CMP*2)	I (CM*%)	MMB (KMP*H)	MPB (KMP*H)	PP (KMS)	PU (KMS)
1	1	2	0	111.00	2561.06	390.00	390.00	2750.00	2750.00
2	2	3	0	111.00	2561.06	390.00	390.00	2750.00	2750.00
3	3	4	0	111.00	2561.06	390.00	390.00	2750.00	2750.00
4	4	5	0	111.00	2561.06	390.00	390.00	2750.00	2750.00
5	5	6	0	111.00	2561.06	390.00	390.00	2750.00	2750.00
6	6	7	0	111.00	2561.06	390.00	390.00	2750.00	2750.00
7	7	8	0	111.00	2561.06	390.00	390.00	2750.00	2750.00
8	8	9	0	111.00	2561.06	390.00	390.00	2750.00	2750.00
9	9	10	0	111.00	2561.06	390.00	390.00	2750.00	2750.00
10	10	11	0	111.00	2561.06	390.00	390.00	2750.00	2750.00
11	11	12	0	111.00	2561.06	390.00	390.00	2750.00	2750.00
12	12	13	0	111.00	2561.06	390.00	390.00	2750.00	2750.00
13	13	14	0	111.00	2561.06	390.00	390.00	2750.00	2750.00
14	14	15	0	111.00	2561.06	390.00	390.00	2750.00	2750.00
15	15	16	0	111.00	2561.06	390.00	390.00	2750.00	2750.00
16	16	17	0	111.00	2561.06	390.00	390.00	2750.00	2750.00
17	17	18	0	111.00	2561.06	390.00	390.00	2750.00	2750.00

DAMPING FACTOR= 0.0000

INTEGRATION TIME INTERVAL= .0003500 SEC, PRINT TIME INTERVAL= .0335000 SEC, NUMBER OF INCREMENTS= 500

DYNAMIC LOAD TABLE

PRESSURE WAVEFORM 1 NO. OF POINTS= 4

F	T
KPA	SEC
240.20	0.200000
315.00	0.400000
510.00	0.600000
510.00	1.000000

PRESSURE WAVEFORM 2 NO. OF POINTS= 4

G	T
KPA	SEC
100.00	1.000000
100.00	1.000000
100.00	1.000000
100.00	1.000000

PRESSURE WAVEFORM 3 NO. OF POINTS= 5

H	T
KPA	SEC
100.00	1.000000
100.00	1.000000
100.00	1.000000
100.00	1.000000
100.00	1.000000

0.60 0.80000
 0.60 0.83300
 0.71 0.83620
 0.60 0.69750
 0.60 1.00000

PRESSURE WAVEFORM 10 NO.OF POINTS= 5

P	T
KPA	SEC
0.60	0.80000
0.60	0.83300
0.71	0.83620
0.60	0.69750
0.60	1.00000

0.60 0.80000
 0.60 0.84100
 0.15 0.84970
 0.60 0.85660
 0.60 1.00000

PRESSURE WAVEFORM 11 NO.OF POINTS= 5

P	T
KPA	SEC
0.60	0.80000
0.60	0.84100
0.15	0.84970
0.60	0.85660
0.60	1.00000

** TRIBUTARY AREAS **

NODE	AX (M ²)	ID	AY (M ²)	ID
2	10.00	1	0.00	8
4	10.00	1	0.00	3
5	5.00	1	-6.00	2
6	0.00	0	-12.00	3
8	0.00	0	-12.00	4
9	0.00	0	-12.00	5
11	0.00	0	-12.00	6
13	0.00	0	-12.00	7
14	-5.00	9	-6.00	8
15	-10.00	10	-6.00	8
17	-10.00	11	0.00	8

DEAD LOAD PLUS LIVE LOAD

ELEM	UNIFORM LOAD (KGS/M)
1	78.98
2	78.98
3	78.98
4	78.98
5	59.40
6	59.40
7	59.40
8	59.40
9	38.00
10	59.40
11	59.40
12	59.40

MODE	FX (%)	FY (KG)	RZ (CM-H)
13	0.	-904E+03	0.
14	0.	-615E+03	0.
15	0.	-615E+03	0.
16	0.	-566E+03	0.
17	0.	-615E+03	0.
		-615E+03	0.
		-904E+03	0.

EXAMPLE A.5 TWO-BAY RIGID FRAME SUBJECTED TO BLAST LOAD
DEAD AND LIVE LOAD DISPLACEMENTS AND MEMBER LOADS

NODE	X-DISPL (MM)	Y-DISPL (MM)	ROTATION (DEG)	PA (KNS)	YA (KMS)	MA (KMN-M)	PB (KNS)	YB (KMS)	MB (KMN-M)
1	L.	0.	0.						
2	-1152E+00	-1634E-01	4458E-02	-1735E+02	-0703E+00	-623E-13	-1735E+02	-0703E+00	-1466E+01
3	-1495E+00	-218E-01	2959E-02	-1676E+02	-0731E+00	-1466E+01	-1676E+02	-0731E+00	-2190E+01
4	-1431E+00	-276E-01	1596E-02	-1572E+02	-0731E+00	-2190E+01	-1596E+02	-0731E+00	-6396E+01
5	-3349E+02	-3994E-01	-9032E-02	-1477E+02	-0731E+00	2931E+01	-1477E+02	-0731E+00	-4895E+01
6	-223E+02	-4437E+00	-7572E-02	-0703E+00	-513E+01	-6396E+01	-0703E+00	-513E+01	-4246E+01
7	-1674E-02	-4893E+00	2204E-02	-0925E+00	-0925E+00	-6396E+01	-0925E+00	-0925E+00	-3389E+01
8	-1115E-02	-485E+00	149E-01	-5384E+01	-5384E+01	-3389E+01	-5384E+01	-5384E+01	-0951E+01
9	-912E-14	-4360E-01	-5533E-16	-1244E-13	-1244E-13	-3389E+01	-1244E-13	-1244E-13	-3979E-12
10	L.	0.	0.						
11	-1116E-02	-3754E+00	-1043E-01	-0703E+00	-0598E+01	-6396E+01	-0703E+00	-0598E+01	-4246E+01
12	-1074E-02	-4893E+00	-2204E-02	-0731E+00	-0731E+00	-6396E+01	-0731E+00	-0731E+00	-6396E+01
13	-223E-02	-4437E+00	7572E-02	-0731E+00	-0731E+00	-6396E+01	-0731E+00	-0731E+00	-6396E+01
14	-134E-02	-3994E-01	9032E-02	-0731E+00	-0731E+00	-6396E+01	-0731E+00	-0731E+00	-6396E+01
15	-1431E+00	-276E-01	1596E-02	-0731E+00	-0731E+00	-6396E+01	-0731E+00	-0731E+00	-6396E+01
16	-1495E+00	-218E-01	2959E-02	-0731E+00	-0731E+00	-6396E+01	-0731E+00	-0731E+00	-6396E+01
17	-1152E+00	-1634E-01	4458E-02	-0731E+00	-0731E+00	-6396E+01	-0731E+00	-0731E+00	-6396E+01
18	0.	0.	0.						

EXAMPLE 4.5 TWO-BAY RIGID FRAME SUBJECTED TO BLAST LOAD

TIME	Y-DISPL (MM)	X-DISPL (MM)	ROTATION (DEG)	MA (KN-M)	MB (KN-M)	MC (KN-M)	MA/MB	MB/MS
1	0.	0.	0.	0.	0.	0.	0.	0.
2	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
3	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
4	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
5	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
6	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
7	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
8	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
9	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
10	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
11	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
12	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
13	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
14	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
15	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
16	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
17	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
18	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000

ELEMENT END ROTATIONS IN DEGREES

ELEM	ELASTIC	PLASTIC	ELASTIC	PLASTIC	P/PP
1	0.000000	0.000000	0.000000	0.000000	0.000000
2	0.000000	0.000000	0.000000	0.000000	0.000000
3	0.000000	0.000000	0.000000	0.000000	0.000000
4	0.000000	0.000000	0.000000	0.000000	0.000000
5	0.000000	0.000000	0.000000	0.000000	0.000000
6	0.000000	0.000000	0.000000	0.000000	0.000000
7	0.000000	0.000000	0.000000	0.000000	0.000000
8	0.000000	0.000000	0.000000	0.000000	0.000000
9	0.000000	0.000000	0.000000	0.000000	0.000000
10	0.000000	0.000000	0.000000	0.000000	0.000000
11	0.000000	0.000000	0.000000	0.000000	0.000000
12	0.000000	0.000000	0.000000	0.000000	0.000000
13	0.000000	0.000000	0.000000	0.000000	0.000000
14	0.000000	0.000000	0.000000	0.000000	0.000000
15	0.000000	0.000000	0.000000	0.000000	0.000000
16	0.000000	0.000000	0.000000	0.000000	0.000000
17	0.000000	0.000000	0.000000	0.000000	0.000000

EXAMPLE A.5 TWO-BAY RIGID FRAME SUBJECTED TO BLAST LOAD

TIME = .175E-01 SEC

NODE	R-DISPL (MM)	Y-DISPL (MM)	ROTATION (DEG)	PA	PC	JA (KNS)	MA (K-N)	WA/WMA	VB (KNS)	WB (K-N)	MB/WMB
1	0.	0.	-1.107E+01	0.	0.	2.335E+03	6.04E-11	.1551E-13	-2.235E+03	3.093E+03	.993E+00
2	2.35E+02	1.342E+00	-6.277E+03	0.	0.	-9.97E+02	-3.093E+03	.993E+00	6.947E+02	2.718E+03	8.498E+00
3	2.35E+02	2.016E+00	1.11E+00	0.	0.	-9.97E+02	-3.315E+03	8.498E+00	6.947E+02	2.718E+03	8.498E+00
4	2.35E+02	1.262E+00	4.205E+00	0.	0.	-9.97E+02	-2.718E+03	8.498E+00	6.947E+02	2.718E+03	8.498E+00
5	9.44E+01	-2.285E-01	4.726E+00	0.	0.	-2.976E+03	2.228E+03	8.778E+00	-2.976E+03	2.228E+03	8.778E+00
6	9.44E+01	6.030E+00	-1.750E+00	0.	0.	1.082E+03	2.228E+03	8.778E+00	-2.976E+03	2.228E+03	8.778E+00
7	8.214E+01	-2.771E-01	-1.532E+00	0.	0.	6.658E+02	-3.405E+02	1.709E+01	-6.658E+02	3.405E+02	1.709E+01
8	7.957E+01	5.913E+01	1.176E-01	0.	0.	-1.328E+03	1.095E+03	1.861E-16	1.095E+03	-1.328E+03	1.861E-16
9	7.754E+01	0.	3.914E-01	0.	0.	-1.946E+00	4.43E-12	1.061E-16	-1.946E+00	4.43E-12	1.061E-16
10	7.595E+01	-3.065E-01	8.626E-02	0.	0.	1.705E+02	2.453E+02	9.958E-01	-1.705E+02	2.453E+02	9.958E-01
11	7.255E+01	2.673E+00	2.313E-01	0.	0.	-5.32E+01	-9.958E+01	3.155E-01	5.32E+01	9.958E+01	3.155E-01
12	7.155E+01	6.097E+00	-2.268E-01	0.	0.	-8.13E+01	-3.155E+01	3.155E-01	8.13E+01	3.155E+01	3.155E-01
13	6.955E+01	-8.295E-01	1.138E+00	0.	0.	2.517E+02	3.860E+01	1.323E-01	-2.517E+02	3.860E+01	1.323E-01
14	3.125E+01	5.562E-01	-1.361E+00	0.	0.	-1.356E+02	2.870E+02	7.302E-01	1.356E+02	-2.870E+02	7.302E-01
15	1.655E+01	-6.263E-01	9.562E-01	0.	0.	1.878E+02	4.815E+02	1.878E+01	-1.878E+02	4.815E+02	1.878E+01
16	0.0E+00	-2.268E-01	-4.008E-01	0.	0.	-3.08E+02	3.305E-13	1.365E-04	3.08E+02	-3.305E-13	1.365E-04
17	0.	0.	-3.755E-02	0.	0.	-3.08E+02	3.305E-13	1.365E-04	3.08E+02	-3.305E-13	1.365E-04

ELEMENT END ROTATIONS IN DEGREES

ELEM	ELASTIC	PLASTIC	ELASTIC	PLASTIC	P/PP
1	-1.122E+08	0.	0.	0.	0.
2	-1.743E+08	0.	0.	0.	0.
3	-1.596E+08	0.	0.	0.	0.
4	-1.107E+08	0.	0.	0.	0.
5	2.840E+06	0.	0.	0.	0.
6	1.894E+02	0.	0.	0.	0.
7	-7.662E-01	0.	0.	0.	0.
8	-8.799E-01	0.	0.	0.	0.
9	-6.295E-01	0.	0.	0.	0.
10	2.767E-01	0.	0.	0.	0.
11	-8.445E-02	0.	0.	0.	0.
12	-1.874E-02	0.	0.	0.	0.
13	4.461E-03	0.	0.	0.	0.
14	8.132E-03	0.	0.	0.	0.
15	1.857E-03	0.	0.	0.	0.
16	-2.235E-03	0.	0.	0.	0.
17	-1.754E-03	0.	0.	0.	0.

EXAMPLE A.5 THO-BAY RIGID FLAME SUBJECTED TO BLAST LOAD

TIME	MOLE	2180E-01 SEC	V-DISPL	ROTATION	MA	MA/RMA	V5	M3	M3/RM3
			(MM)	(DEG)	(KN-M)		(KNS)	(KN-M)	
1									
2									
3									
4									
5									
6									
7									
8									
9									
10									
11									
12									
13									
14									
15									
16									
17									
18									

ELEMENT END ROTATIONS IN DEGREES

ELEM	PLA	PA/PC	VA	MA	MA/RMA	V5	M3	M3/RM3
	(KNS)		(KNS)	(KN-M)		(KNS)	(KN-M)	
1								
2								
3								
4								
5								
6								
7								
8								
9								
10								
11								
12								
13								
14								
15								
16								
17								

EXAMPLE A.5 TWO-BAY RIGID FRAME SUBJECTED TO BLAST LOAD

TIME=	HOGE	.7030E-01	SEC	X-DISPL	Y-DISPL	ROTATION	MA	MB	MC	MB/MBB
				(MM)	(MM)	(DEG)	(KMN)	(KMN)	(KMN)	
1	C.	1.0								
2		1.0								
3		1.0								
4		1.0								
5		1.0								
6		1.0								
7		1.0								
8		1.0								
9		1.0								
10		1.0								
11		1.0								
12		1.0								
13		1.0								
14		1.0								
15		1.0								
16		1.0								
17		1.0								
18		1.0								

ELEMENT END ROTATIONS IN DEGREES

ELEM	END A	ELASTIC	PLASTIC	ELASTIC	PLASTIC	P/PP
1						
2						
3						
4						
5						
6						
7						
8						
9						
10						
11						
12						
13						
14						
15						
16						
17						

EXAMPLE A.5 TWO-BAY RIGID FRAME SUBJECTED TO BLAST LOAD

ELEM	NOTE	0-33E-01 SFC	X-DISPL (MM)	Y-DISPL (MM)	ROTATION (DEG)	MA (KN-M)	MB (KN-M)	MA/MB	MB (KNS)	MB (KN-M)	MB/MB
1	C	0.12E+02	5787+00	-1.307E+01	-1.307E+01	6027E-11	1545E-13	0.111E-01	-1.430E+02	2303E+02	0.111E-01
2		5487E+02	486.7E+00	-1.307E+01	-1.307E+01	-2.58E+02	0.111E-01	0.111E-01	-1.307E+02	6344E+02	0.111E-01
3		5271E+02	1889E+00	-1.307E+01	-1.307E+01	0.644E+02	1.93E+01	0.111E-01	-1.307E+02	7774E+02	0.111E-01
4		865E+02	468E+00	-1.307E+01	-1.307E+01	-9.74E+02	2.68E+00	0.111E-01	-1.307E+02	6821E+02	0.111E-01
5		863E+02	1128E+02	-1.307E+01	-1.307E+01	5.233E+02	0.079E+00	0.111E-01	-1.307E+02	1.94E+03	0.111E-01
6		739E+02	1168E+02	-1.307E+01	-1.307E+01	-1.01E+03	0.079E+00	0.111E-01	-1.307E+02	1.01E+03	0.111E-01
7		749E+02	1743E+01	-1.307E+01	-1.307E+01	-1.458E+03	0.079E+00	0.111E-01	-1.307E+02	0.079E+00	0.111E-01
8		742E+02	1.580E+01	-1.307E+01	-1.307E+01	2.160E+02	0.079E+00	0.111E-01	-1.307E+02	0.079E+00	0.111E-01
9	C	0.101E+02	980E+00	-1.307E+01	-1.307E+01	0.678E+02	0.079E+00	0.111E-01	-1.307E+02	0.079E+00	0.111E-01
10		812E+02	234E+01	-1.307E+01	-1.307E+01	-1.458E+03	0.079E+00	0.111E-01	-1.307E+02	0.079E+00	0.111E-01
11		812E+02	234E+01	-1.307E+01	-1.307E+01	0.678E+02	0.079E+00	0.111E-01	-1.307E+02	0.079E+00	0.111E-01
12		737E+02	1.577E+03	-1.307E+01	-1.307E+01	-1.458E+03	0.079E+00	0.111E-01	-1.307E+02	0.079E+00	0.111E-01
13		737E+02	1.577E+03	-1.307E+01	-1.307E+01	0.678E+02	0.079E+00	0.111E-01	-1.307E+02	0.079E+00	0.111E-01
14		593E+02	3.621E-01	-1.307E+01	-1.307E+01	-1.458E+03	0.079E+00	0.111E-01	-1.307E+02	0.079E+00	0.111E-01
15		593E+02	3.621E-01	-1.307E+01	-1.307E+01	0.678E+02	0.079E+00	0.111E-01	-1.307E+02	0.079E+00	0.111E-01
16		322E+02	1.179E+00	-1.307E+01	-1.307E+01	-1.458E+03	0.079E+00	0.111E-01	-1.307E+02	0.079E+00	0.111E-01
17	C	0.7837E+01	0.7837E+01	-1.307E+01	-1.307E+01	0.678E+02	0.079E+00	0.111E-01	-1.307E+02	0.079E+00	0.111E-01

ELEMENT END ROTATIONS IN DEGREES

ELEM	ELASTIC	PLASTIC	ELASTIC	PLASTIC	P/PP
1	0.029E-02	0.	-0.09E-02	0.	0.267E-01
2	-0.595E-02	0.	-2.46E-01	0.	0.754E-01
3	-0.374E-01	0.	0.337E-01	0.	0.97E-01
4	-0.889E-01	0.	0.764E-01	0.	0.291E-01
5	-2.528E+00	0.	-2.88E+00	0.	0.495E-01
6	-1.676E+00	0.	-1.287E+00	0.	0.495E-01
7	-0.508E-01	0.	0.156E-01	0.	0.495E-01
8	-0.535E+00	0.	-0.327E+00	0.	0.495E-01
9	-0.301E+00	0.	-0.682E+00	0.	0.220E-02
10	-2.221E+00	0.	0.682E+00	0.	0.220E-02
11	0.728E-03	0.	-0.682E+00	0.	0.220E-02
12	0.295E-01	0.	-1.63E-01	0.	0.759E-01
13	0.196E+00	0.	-1.63E-01	0.	0.759E-01
14	-1.350E+00	0.	-1.276E+00	0.	0.384E-01
15	-1.746E-01	0.	-0.726E-01	0.	0.384E-01
16	-0.782E-01	0.	-0.726E-01	0.	0.384E-01
17	-0.535E-01	0.	-0.167E+00	0.	0.384E-01

EXAMPLE A-5 TWO-BAY RIGID FRAME SUBJECTED TO BLAST LOAD
 TABULATION OF MAXIMUM AND MINIMUM ELEMENT END LOADS

ELEMENT	MAXP (KNS)	ASSOC.M (KN-M)	ASSOC.V (KNS)	TIME (SEC)	HM (KN-M)	MAX V (KN-M)	ASSOC.P (KNS)	ASSOC.V (KNS)	TIME (SEC)	MIN/MAX
1 1	.3335E+03	.0624E-11	.1327E+03	.2765E-01	.592E+03	.9624E-11	-.2698E+03	.6295E+02	.1064E+00	.00
	-.3665E+03	.7196E-11	-.0183E+02	.6475E-01	.3905E+03	-.2147E-12	.1207E+03	-.1449E+02	.1747E+00	-.00
1 2	.3335E+03	.1711E+03	-.1327E+03	.2765E-01	.3905E+03	.9624E+03	.1491E+03	.2376E+03	.2033E+01	.622
	-.3665E+03	.6972E+02	-.0183E+02	.6475E-01	.3905E+03	-.1167E+03	.1703E+03	.1803E+03	.1346E+00	-.43
2 2	.3317E+03	-.1711E+03	-.2367E+02	.2765E-01	.3905E+03	.1672E+03	.1691E+03	.5925E+02	.1341E+00	.43
	-.3670E+03	.6972E+02	.0810E+02	.6475E-01	.3905E+03	-.1961E+03	.1478E+03	-.1108E+03	.2030E-01	-1.02
2 3	.3317E+03	.1515E+03	-.2367E+02	.2765E-01	.3905E+03	.1672E+02	.2221E+03	.2492E+02	.1435E-01	.91
	-.3670E+03	.1432E+03	.0810E+02	.6475E-01	.3905E+03	-.1179E+03	.1590E+03	-.5925E+02	.1341E+00	-.30
3 3	.3311E+03	-.1515E+03	-.2367E+02	.2765E-01	.3905E+03	.1179E+03	.1604E+03	.5925E+02	.1341E+00	.30
	-.3685E+03	.1317E+03	.0810E+02	.6475E-01	.3905E+03	-.1359E+03	.2015E+03	-.2492E+02	.1435E-01	-.51
3 4	.3334E+03	-.1317E+03	-.1545E+03	.2765E-01	.3905E+03	.3348E+03	.2824E+03	.2396E+02	.1408E-01	.06
	-.3631E+03	.2166E+03	.6796E+01	.6475E-01	.3905E+03	-.3348E+03	.1693E+02	.1693E+02	.1302E+00	-.29
4 4	.3334E+03	.1545E+03	-.1545E+03	.2765E-01	.3905E+03	.1192E+03	.6779E+01	.1346E+03	.1302E+00	.29
	-.3631E+03	.2166E+03	.6796E+01	.6475E-01	.3905E+03	-.3348E+03	.2317E+03	-.3403E+03	.1408E-01	-.06
4 5	.3334E+03	.1192E+03	.1505E+03	.2765E-01	.3905E+03	.2652E+03	.2649E+02	.2248E+02	.1833E-01	.63
	-.3631E+03	.2203E+03	.0796E+01	.6475E-01	.3905E+03	-.2325E+03	.2008E+03	.3403E+03	.1435E-01	-.00
5 5	.3846E+03	.2033E+03	.0905E+02	.1492E-01	.2540E+03	.2325E+03	.2971E+03	.1188E+03	.1435E-01	.92
	-.2014E+03	-.1875E+02	.1629E+02	.1492E-01	.2540E+03	-.2652E+03	.5179E+02	-.1379E+03	.6099E-01	-.97
5 6	.3846E+03	.2203E+02	-.3249E+02	.1492E-01	.2540E+03	.1876E+03	.6327E+02	-.0772E+02	.6346E-01	.74
	-.2014E+03	.5016E+02	.1247E+02	.1492E-01	.2540E+03	-.1176E+03	-.0827E+02	-.8723E+02	.4346E-01	-.74
6 6	.3846E+03	.2203E+02	.7012E+02	.1492E-01	.2540E+03	.1316E+03	.6788E+02	.5566E+02	.1572E+00	.92
	-.2014E+03	.5016E+02	.1247E+02	.1492E-01	.2540E+03	-.1176E+03	-.0827E+02	-.8723E+02	.4346E-01	-.74
6 7	.3846E+03	.0762E+02	-.6994E+02	.1492E-01	.2540E+03	.1003E+03	.1033E+02	.6666E+02	.0577E-01	.63
	-.2014E+03	.6234E+02	.1189E+02	.1492E-01	.2540E+03	-.1108E+03	.7730E+02	-.0777E+02	.1592E+00	-.30
7 7	.3846E+03	.0624E+02	.6994E+02	.1492E-01	.2540E+03	.7730E+02	.7807E+02	.6747E+02	.1502E+00	.30
	-.2014E+03	.6234E+02	.1189E+02	.1492E-01	.2540E+03	-.1108E+03	-.1083E+02	-.6666E+02	.0577E-01	-.63
7 8	.3846E+03	.1189E+03	-.6894E+02	.1492E-01	.2540E+03	.1367E+03	.1367E+03	-.5232E+02	.1750E+00	.51
	-.2014E+03	.7303E+02	.1136E+02	.1492E-01	.2540E+03	-.1091E+03	-.1636E+02	.4860E+02	.1635E+00	-.43
8 8	.3846E+03	.1189E+03	.1099E+03	.1492E-01	.2540E+03	.1891E+03	.1636E+02	.1248E+03	.1635E+00	.93
	-.2014E+03	.7303E+02	.1045E+03	.1492E-01	.2540E+03	-.1347E+03	-.1523E+02	-.1895E+03	.1750E+00	-.91
8 9	.3846E+03	.1045E+03	.1111E+03	.1492E-01	.2540E+03	.1305E+03	.2935E+02	-.1227E+03	.1635E+00	.94
	-.2014E+03	.1439E+03	.1095E+03	.1492E-01	.2540E+03	-.1271E+03	.3727E+02	.1606E+03	.6830E-01	-1.07
9 10	.4717E+03	.1278E-11	.2242E+02	.6725E-01	.4150E+03	.2278E-11	.3468E+02	.6033E+02	.7942E-01	.06
	-.2339E+03	.3429E-12	.6877E+01	.6725E-01	.4150E+03	-.1408E-12	.1203E+03	-.5579E+06	.1750E+00	-.00
9 3	.6217E+03	.1121E+03	-.2642E+02	.7235E-01	.4150E+03	.3518E+03	.1447E+03	.7830E+03	.0268E-01	.05
	-.2339E+03	.3833E+02	.6877E+01	.7235E-01	.4150E+03	-.8671E+02	-.7815E+02	.1694E+02	.1624E+00	-.20

A PLUS SIGN (+) FOR MAXP INDICATES COMPRESSION AND A MINUS SIGN (-) FOR MAXP INDICATES TENSION

EXAMPLE A.5 TWO-BAY RIGID FRAME SUBJECTED TO BLAST LOAD

ELEM NO	MAXP (KNS)	ASSOC. V (KM-M)	TIME (SEC)	MM (CM-1)	MAX. V (KM-1)	ASSOC. P (KNS)	ASSOC. V (KNS)	TIME (SEC)	MM/M/4M
10 9	.2642E+03	-.2230E+03	.6645E-01	.2540E+03	.1623E+03	.3173E+02	.1102E+03	.6595E-01	.64
	-.1803E+03	-.5976E+02	.0535E-01	.2540E+03	-.2572E+03	-.8649E+02	-.1398E+03	.7200E-01	-1.61
10 11	.2642E+03	.0626E+02	.6937E+02	.2540E+03	.2198E+03	-.1102E+02	.1902E+03	.6659E-01	.87
	-.1803E+03	-.4401E+01	.0535E-01	.2540E+03	-.6242E+02	.0202E+02	.1294E+03	.1244E+00	-.25
11 11	.2642E+03	-.0626E+02	.1111E+03	.2540E+03	.6242E+02	.0202E+02	.1315E+02	.1244E+00	.25
	-.1803E+03	.4431E+01	.0505E-01	.2540E+03	-.2198E+03	-.1102E+02	-.6200E+02	.6659E-01	-.87
11 12	.2642E+03	-.2516E+02	.1117E+03	.2540E+03	.1772E+03	-.3267E+02	.4824E+02	.6480E-01	.62
	-.1803E+03	.6185E+01	.0535E-01	.2540E+03	-.7221E+02	.3267E+02	.7517E+02	.7770E-01	-.28
12 12	.2642E+03	.2516E+02	.1117E+03	.2540E+03	.7221E+02	.3267E+02	.7517E+02	.7770E-01	.28
	-.1803E+03	-.6185E+01	.0535E-01	.2540E+03	-.1575E+03	-.1267E+02	-.5324E+02	.6810E-01	-.62
12 17	.2642E+03	-.1372E+03	.1124E+03	.2540E+03	.1832E+03	.0279E+01	-.1102E+02	.5324E-01	.64
	-.1803E+03	.0470E+01	.2577E-01	.2540E+03	-.1653E+03	.1678E+03	.0746E+02	.7945E-01	.65
13 13	.2642E+03	.1372E+03	.1124E+03	.2540E+03	.1653E+03	.1678E+03	.0746E+02	.7945E-01	.65
	-.1803E+03	-.0470E+01	.2577E-01	.2540E+03	-.1632E+03	.6279E+01	-.2132E+03	.5320E-01	-.64
13 14	.2642E+03	-.2520E+03	.1529E+02	.2540E+03	.1317E+03	-.0116E+02	-.6077E+02	.1753E-01	.52
	-.1803E+03	.1517E+03	.0721E+02	.2540E+03	-.2664E+03	.6665E+02	.2070E+03	.5495E-01	-.65
14 15	.2642E+03	-.1522E+03	.1958E+02	.3980E+03	.1668E+03	-.1524E+03	.1590E+03	.1620E-01	.63
	-.1803E+03	.6962E+01	.1244E+02	.3980E+03	-.2821E+03	.3148E+03	-.1775E+02	.6909E-01	-.72
14 16	.2642E+03	.2101E+03	.1958E+02	.3980E+03	.2664E+03	.2773E+03	.1117E+03	.5495E-01	.68
	-.1803E+03	-.6790E+02	.2461E+02	.3980E+03	-.1317E+03	-.2602E+02	-.1119E+03	.1750E+00	-.34
15 16	.2642E+03	-.0658E+02	.0601E+02	.3980E+03	.1349E+03	.3700E+03	.3087E+01	.6759E-01	.34
	-.1803E+03	.4216E+02	.5097E+02	.3980E+03	-.1051E+03	-.2522E+03	.2675E+02	.6620E-01	-.67
15 15	.2642E+03	.1522E+03	.1958E+02	.3980E+03	.0601E+02	.4821E+02	.1102E+03	.6909E-01	.72
	-.1803E+03	-.6962E+01	.1244E+02	.3980E+03	-.1664E+03	.6665E+02	.5085E+02	.1620E+00	-.63
16 17	.2642E+03	-.0936E+01	.0631E+02	.3980E+03	.2204E+03	.1702E+03	.1192E+03	.7070E-01	.57
	-.1803E+03	.9120E+02	.5097E+02	.3980E+03	-.2362E+03	-.0221E+02	-.0440E+02	.1632E+00	-.99
16 15	.2642E+03	.0936E+01	.0631E+02	.3980E+03	.0601E+02	.4821E+02	.1102E+03	.6909E-01	.67
	-.1803E+03	-.6962E+01	.1244E+02	.3980E+03	-.1664E+03	.6665E+02	.5085E+02	.1620E+00	-.63
17 18	.2642E+03	.5466E-11	.5361E+01	.3980E+03	.7222E-11	.2334E+03	.5267E+02	.6480E-01	.68
	-.1803E+03	.3224E-11	.5475E+02	.3980E+03	-.2362E-12	.6120E+02	-.2942E+02	.1579E-01	-.68
17 17	.2642E+03	.0926E+01	.5361E+01	.3980E+03	.2302E+03	-.0492E+02	-.1301E+03	.1432E+00	.59
	-.1803E+03	-.9120E+02	.5475E+02	.3980E+03	-.2204E+03	.1794E+03	.1322E+03	.7070E-01	-.37

A PLUS SIGN (+) FOR MAXP INDICATES COMPRESSION AND A MINUS SIGN (-) FOR MAXP INDICATES TENSION

EXAMPLE A.5 TWO-BAY RIGID FRAME SUBJECTED TO BLAST LOAD

		MAXIMUM ELEMENT END ROTATIONS				MINIMUM ELEMENT END ROTATIONS					
		ELAS	PLAS	P/PPP	DISPL/ITY RATIO	ELAS	TIME	PLAS	TIME	P/PPP	DISPL/ITY RATIO
ELEM 1	MODE 1	-0.696	-0.134	0.0000	0.000	3.000	0.000	0.0000	0.000	0.000	0.000
	MODE 2	-2.444	-0.020	-0.3030	-0.044	2.957	-0.1392	-0.134	0.0000	0.000	0.000
ELEM 2	MODE 2	-1.895	-0.134	0.0000	0.000	1.000	-0.1743	-0.020	-0.044	0.000	0.000
	MODE 3	-1.051	-0.015	0.0000	0.000	1.000	-0.0007	-0.134	0.0000	0.000	0.000
ELEM 3	MODE 3	-0.395	-0.131	0.0000	0.000	1.000	-0.1773	-0.134	0.0000	0.000	0.000
	MODE 4	-1.745	-0.014	0.0000	0.000	1.000	-0.0561	-0.134	0.0000	0.000	0.000
ELEM 4	MODE 4	-0.640	-0.175	0.0000	0.000	1.000	-0.2005	-0.113	0.0000	0.000	0.000
	MODE 5	-2.324	-0.065	0.0000	0.000	1.000	-0.0752	-0.175	0.0000	0.000	0.000
ELEM 5	MODE 5	-0.932	-0.014	0.1579	0.014	1.559	-0.3700	-0.068	-0.0752	-0.166	-0.0752
	MODE 6	-3.134	-0.009	0.0000	0.000	1.000	-0.1936	-0.117	0.0000	0.000	0.000
ELEM 6	MODE 6	-1.222	-0.157	0.0000	0.000	1.000	-0.1709	-0.043	0.0000	0.000	0.000
	MODE 7	-1.408	-0.066	0.0000	0.000	1.000	-0.1023	-0.158	0.0000	0.000	0.000
ELEM 7	MODE 7	-1.846	-0.163	0.0000	0.000	1.000	-0.0956	-0.175	0.0000	0.000	0.000
	MODE 8	-1.103	-0.175	0.0000	0.000	1.000	-0.1032	-0.163	0.0000	0.000	0.000
ELEM 8	MODE 8	-0.275	-0.097	0.0000	0.000	1.000	-0.1343	-0.174	0.0000	0.000	0.000
	MODE 9	-0.354	-0.051	0.0000	0.000	1.000	-0.3477	-0.066	-0.1399	-0.066	-0.1317
ELEM 9	MODE 10	-0.733	-0.162	0.0000	0.000	1.000	-0.3643	-0.043	0.0000	0.000	0.000
	MODE 11	-0.606	-0.083	0.0000	0.000	1.000	-0.1465	-0.162	0.0000	0.000	0.000
ELEM 10	MODE 10	-0.395	-0.075	0.0000	0.000	1.000	-0.3006	-0.070	-0.1911	-0.070	-0.144
	MODE 11	-0.320	-0.064	0.0000	0.000	1.000	-0.1706	-0.172	0.0000	0.000	0.000
ELEM 11	MODE 11	-0.000	-0.074	0.0000	0.000	1.000	-0.2144	-0.047	0.0000	0.000	0.000
	MODE 12	-1.917	-0.047	0.0000	0.000	1.000	-0.0716	-0.074	0.0000	0.000	0.000

*NOTE - THE LETTER C INDICATES COMPRESSION AND THE LETTER T INDICATES TENSION

EXAMPLE A.5 TWO-BAY RIGID FRAME SUBJECTED TO BLAST LOAD

	MAXIMUM ELEMENT END ROTATIONS				MINIMUM ELEMENT END ROTATIONS			
	ELAS	TIME	PLAS	TIME	PLAS	TIME	PLAS	TIME
ELEM12 NODE 12	-1.92	.079	5.0000	0.000	0.000	1.000	0.000	0.000
ELEM13 NODE 13	-1.659	.153	5.0270	0.000	0.000	1.000	0.000	0.000
ELEM15 NODE 15	-1.033	.066	0.0000	0.000	0.000	1.000	0.000	0.000
ELEM16 NODE 16	-1.263	.104	0.0010	0.104	0.0287	1.000	0.000	0.000
ELEM15 NODE 15	-0.867	.152	5.0000	0.000	0.000	1.000	0.000	0.000
ELEM16 NODE 16	-0.800	.089	0.0000	0.000	0.000	1.000	0.000	0.000
ELEM15 NODE 15	-0.715	.102	0.0000	0.000	0.000	1.000	0.000	0.000
ELEM16 NODE 16	-1.271	.055	0.0000	0.000	0.000	1.000	0.000	0.000
ELEM15 NODE 15	-0.900	.071	0.0000	0.000	0.000	1.000	0.000	0.000
ELEM16 NODE 16	-0.960	.065	0.0000	0.000	0.000	1.000	0.000	0.000
ELEM17 NODE 17	-0.682	.071	0.0000	0.000	0.000	1.000	0.000	0.000
ELEM17 NODE 17	-0.570	.143	0.0000	0.000	0.000	1.000	0.000	0.000

*NOTE - THE LETTER C INDICATES COMPRESSION AND
THE LETTER T INDICATES TENSION

EXAMPLE A-5 TWO-BAY RIGID FRAME SUBJECTED TO BLAST LOAD

MAXIMUM LOCAL DISPLACEMENT TABLE

MODE	X(1) (MM)	TIME (SECS)	X(2) (MM)	TIME (SECS)	Y(1) (MM)	TIME (SECS)	Y(2) (MM)	TIME (SECS)	R(1) (DEG)	TIME (SECS)	R(2) (DEG)	TIME (SECS)
1	6.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
2	45.42446	0.1116	-0.0919	0.0835	0.1141	0.0675	-0.1762	0.0942	0.0982	0.1750	-0.07969	0.0615
3	56.31314	0.2260	-0.1197	0.0835	1.21725	0.0475	-0.26395	0.0844	0.1945	0.1750	-0.06085	0.0498
4	66.07019	0.3315	-0.1174	0.0835	1.30415	0.0475	-0.35139	0.0844	0.4261	0.1800	-0.78861	0.0415
5	81.84794	0.4420	0.0000	0.0806	1.65900	0.0475	-0.52559	0.0844	0.5122	0.1470	-0.61970	0.0525
6	4.33451	0.6005	0.0000	0.10443	9.27036	0.15785	-1.5.56503	0.0715	0.9092	0.1650	-0.28029	0.17503
7	82.30235	0.6250	0.0000	0.0000	5.56985	0.15923	-12.67131	0.08645	0.2181	0.13265	-0.15793	0.1855
8	89.20754	0.8015	0.0000	0.2084	7.7402	0.16170	-9.77332	0.17504	0.3654	0.16545	-0.20518	0.18750
9	78.64103	0.8090	0.0000	0.3000	5.8849	0.15120	-9.98611	0.0723	0.4546	0.11960	-0.57378	0.0835
10	6.0000	1.0000	0.0000	0.0000	6.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
11	80.25929	0.8705	-0.0111	0.0000	0.16416	0.07455	-17.43517	0.0723	0.27517	0.17505	-0.28435	0.05943
12	8.0000	1.0000	0.0000	0.0000	9.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
13	81.42577	0.8575	-0.0223	0.0000	10.89886	0.07710	-16.11417	0.05005	0.48578	0.16655	-0.66000	0.17500
14	73.50485	0.8940	-0.04584	0.1750	5.9423	0.17220	-0.02034	0.1740	0.0945	0.17500	-0.02567	0.0755
15	56.50825	0.8820	0.2044	0.0000	3.9516	0.17220	-0.55323	0.1740	0.16017	0.17500	-0.90702	0.07140
16	46.59247	0.8750	-0.23575	0.1000	2.9596	0.17220	-0.41535	0.1740	0.09225	0.17500	-0.92536	0.08990
17	32.52081	0.8540	-0.01053	0.1070	1.9709	0.17220	-0.27721	0.1740	0.08976	0.16380	-1.04781	0.08855
18	0.0000	1.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.15995	-1.16725	0.08473

APPENDIX C

METHOD OF INELASTIC DYNAMIC ANALYSIS

APPENDIX C

METHOD OF INELASTIC DYNAMIC ANALYSIS

C.1 Introduction

The theory forming the basis for the inelastic dynamic analysis of frame structures was presented in Section 3 of this report. This appendix presents the mathematical techniques utilized to implement this theory in DYNFA.

C.2 Solution of General Equations of Motion

The general equations of motion for the system are presented in Section 3.4 [Eq (11)]. Separating the independent (dynamic) from the dependent degrees-of-freedom and taking the damping matrix, [C], as being proportional to the mass matrix, [M], Equation (11) can be rewritten in partitioned form as follows:

$$\begin{bmatrix} M_i & 0 \\ 0 & 0 \end{bmatrix} \begin{Bmatrix} \ddot{u}_i \\ \ddot{u}_d \end{Bmatrix} + a_0 \begin{bmatrix} M_i & 0 \\ 0 & 0 \end{bmatrix} \begin{Bmatrix} \dot{u}_i \\ \dot{u}_d \end{Bmatrix} + \begin{bmatrix} K_{\alpha\alpha} & K_{\alpha\beta} \\ K_{\beta\alpha} & K_{\beta\beta} \end{bmatrix} \begin{Bmatrix} u_k \\ u_d \end{Bmatrix} = \begin{Bmatrix} F_i(t) \\ F_d(t) \end{Bmatrix} \quad (1)$$

In the above equation, the subscript "i" refers to the independent (or dynamic) degrees-of-freedom; whereas the subscript "d" refers to the dependent degrees-of-freedom. The parameter "a₀" is the constant of proportionality between the damping matrix and the mass matrix. The stiffness matrix, [K], is written in a partitioned format.

In a non-linear system, the elastic properties of the structure will vary over the duration of the response as the individual members undergo yielding. However, it can be assumed that the structure will respond linearly in a relatively short time interval. Therefore, the non-linear response of a system may be obtained by sequencing a series of short duration linear responses in which the elastic properties of the structure are varied from one-time interval to the next.

Based on this assumption, Equation (1) can be rewritten as follows:

$$\begin{bmatrix} M_i & 0 \\ 0 & 0 \end{bmatrix} \begin{Bmatrix} \ddot{u}_i \\ \ddot{u}_d \end{Bmatrix}_\tau + a_0 \begin{bmatrix} M_i & 0 \\ 0 & 0 \end{bmatrix} \begin{Bmatrix} \dot{u}_i \\ \dot{u}_d \end{Bmatrix}_\tau + \int_{t=0}^{\tau} \begin{bmatrix} K_{\alpha\alpha} & K_{\alpha\beta} \\ K_{\beta\alpha} & K_{\beta\beta} \end{bmatrix} \begin{Bmatrix} \Delta u_i \\ \Delta u_d \end{Bmatrix}_t = \begin{Bmatrix} F_i \\ F_d \end{Bmatrix}_\tau \quad (2)$$

In this equation, the resistance term, $[K]\{u\}$, at time τ , is expressed as the summation of the incremental resistance forces computed in successive time intervals, Δt , from time 0 to time τ . Hence:

$$\begin{bmatrix} K_{\alpha\alpha} & K_{\alpha\beta} \\ K_{\beta\alpha} & K_{\beta\beta} \end{bmatrix}_t = \text{the partitioned system stiffness matrix over a given time interval, } \Delta t$$

$$\begin{Bmatrix} \Delta u_i \\ \Delta u_d \end{Bmatrix}_t = \text{the corresponding incremental displacement vector over the given time interval } \Delta t.$$

The resistance term of Equation (2) can be rewritten as follows:

$$\sum_{t=0}^{\tau} \begin{bmatrix} K_{\alpha\alpha} & K_{\alpha\beta} \\ K_{\beta\alpha} & K_{\beta\beta} \end{bmatrix}_t \begin{Bmatrix} u_i \\ u_d \end{Bmatrix}_t = \begin{Bmatrix} R_i \\ R_d \end{Bmatrix}_\tau + \begin{bmatrix} K_{\alpha\alpha} & K_{\alpha\beta} \\ K_{\beta\alpha} & K_{\beta\beta} \end{bmatrix}_\tau \begin{Bmatrix} \Delta u_i \\ \Delta u_d \end{Bmatrix}_\tau \quad (3)$$

where $\{R_i\}_\tau \equiv \sum_{t=0}^{\tau-\Delta t} [K_{\alpha\alpha}]\{\Delta u_i\} + [K_{\alpha\beta}]\{\Delta u_d\}$

and $\{R_d\}_\tau \equiv \sum_{t=0}^{\tau-\Delta t} [K_{\beta\alpha}]\{\Delta u_i\} + [K_{\beta\beta}]\{\Delta u_d\}$

Substituting Expression (3) into Equation (2) and rearranging terms, we obtain the following relationship at time τ :

$$[M_i]\{\ddot{u}_i\}_\tau + a_0[M_i]\{\dot{u}_i\}_\tau + [K_{eq}]\{\Delta u_i\}_\tau = \{F_i\}_\tau - \{R_i\}_\tau - [K_C]\{F_d - R_d\}_\tau \quad (4)$$

where $[K_{eq}] = [K_{\alpha\alpha}] - [K_{\alpha\beta}][K_{\beta\beta}]^{-1}[K_{\beta\alpha}]$

and $[K_C] = [K_{\alpha\beta}][K_{\beta\beta}]^{-1}$

Equation (4) can then be integrated over time interval Δt to obtain the acceleration vector $\{\ddot{u}\}$ at time τ . The integration is accomplished using the linear acceleration method of numerical integration which is based on the assumption that the acceleration of a given degree-of-freedom varies linearly over the time interval Δt . Applying this principle yields the following expressions for the velocities and incremental displacements at time τ :

$$\{\dot{u}_i\}_\tau = (\Delta t/2)[\{\ddot{u}_i\}_\tau - \{A\}_\tau] + \{B\}_\tau \quad (5a)$$

$$\{\Delta u_i\}_\tau = (\Delta t^2/6)[\{\ddot{u}_i\}_\tau - \{A\}_\tau] \quad (5b)$$

where $\{A\}_\tau = (-6/\Delta t)\{\dot{u}_i\}_{\tau-\Delta t} - 2\{\ddot{u}_i\}_{\tau-\Delta t}$

and $\{B\}_\tau = -\{\dot{u}_i\}_{\tau-\Delta t} - (\Delta t/2)\{\ddot{u}_i\}_{\tau-\Delta t}$

Substituting Equations (5a) and (5b) into Equation (4) yields the following simplified equation:

$$[M^*]\{\ddot{u}_i\}_\tau = \{F^*\}_\tau \quad (6)$$

where $[M^*]_\tau = [M_i] + (a_0\Delta t/2)[M_i] + (\Delta t^2/6)[K_{eq}]$

and $\{F^*\}_\tau = [F_i(\tau)] - \{R_i\} - [K_c]\{F_d - R_d\} - a_0[M_i]\{B\} + (a_0\Delta t/2)[M_i]\{A\} + (\Delta t^2/6)[K_{eq}]\{A\}$

Equation (6) is solved for the unknown accelerations as shown below:

$$\{\ddot{u}_i\}_\tau = [M^*]_\tau^{-1}\{F^*\}_\tau \quad (7)$$

where $[M^*]_\tau^{-1}$ = the inverse of matrix $[M^*]_\tau$.

With the accelerations thus obtained, the velocities, $\{\dot{u}_i\}_\tau$, and the incremental displacements, $\{\Delta u_i\}_\tau$, can be obtained using Equations (5a) and (5b).

The solutions of Equation (7) is obtained with a step-by-step procedure, starting at zero time when the displacement and velocity are presumably known. Hence, the initial acceleration $\{\ddot{u}_i\}_0$ can be obtained from the following expression:

$$\{\ddot{u}_i\}_0 = [M_i]^{-1}\left\{F_i(0) - [K_{eq}]\{u_i\}_0 - [K_c]\{F_d(0)\}\right\} - a_0\{\dot{u}_i\}_0 \quad (8)$$

The time scale is divided into discrete intervals, Δt , and one progresses by successively extrapolating the accelerations from one time station to the next.

C.3 Composition of Applied Loads Matrix

The applied loads matrix, $\{F(t)\}$, is composed of the following quantities:

1. The time dependent blast loads, $P(t)$, applied to the independent degrees-of-freedom.
2. The unbalanced shears produced by the second order effects ($P-\Delta$, beam column) which occurs as the structure responds to the blast.

In a partitioned format, the matrix of the applied loads is written as follows:

$$\begin{Bmatrix} F_i(t) \\ F_d(t) \end{Bmatrix} = \begin{Bmatrix} P(t) \\ 0 \end{Bmatrix} + \begin{Bmatrix} V_i \\ V_d \end{Bmatrix}_t \quad (9)$$

The blast loads, $P(t)$, are computed by the analyst; whereas the unbalanced shears, V , are computed on the basis of the deflected position of the elements at the end of each time interval.

Consider the element shown in Figure 1. By the end of the $(i - 1)$ th time increment, the element has deflected to the position shown in Figure 1a, and the unbalanced moment at this time is:

$$(M_u)_{i-1} = P(y_B - y_A) \quad (10)$$

Over the next increment, the element deflects to the position shown in Figure 1b and the axial load increases by an amount ΔP . Therefore, at the end of the (i) th time increment, the unbalanced moment is:

$$(M_u)_i = (P + \Delta P)[y_B + \Delta y_B - (y_A + \Delta y_A)] \quad (11)$$

The change in the unbalanced moment over the (i) th increment is determined by subtracting Equation (11) from Equation (10), thereby obtaining:

$$\Delta M_u = P(\Delta y_B - \Delta y_A) + \Delta P[(y_B + \Delta y_B) - (y_A + \Delta y_A)] \quad (12)$$

The incremental unbalanced moment, ΔM_u , is applied to the element in terms of an unbalanced shear, ΔV_u , which is obtained as follows:

$$\Delta V = (\Delta M_u/L) \quad (13)$$

where L is the length of the element. These unbalanced shears are applied at the ends of the element as shown in Figure 1c.

The total unbalanced shear at a given time increment is then the sum of the previously computed incremental shears.

$$\begin{Bmatrix} V_i \\ V_d \end{Bmatrix} = \sum_{t=0}^{t=\tau} \begin{Bmatrix} V_i \\ V_d \end{Bmatrix} \quad (14)$$

The computation of these unbalanced shears, and their inclusion in the applied loads vector are performed internally by DYNFA.

C.4 Element Stiffness Matrix

In the analysis, the element stiffness matrices may vary from one time increment to the next due to the occurrence of non-linear behavior. During a given time increment, however, the elements are assumed to behave linearly. Thus, the force-displacement relationship for an element is expressed on an incremental basis, as follows:

$$\{\Delta f\} = \{k\}\{\Delta u\} \quad (15)$$

where

$\{\Delta f\}$ = the change in the element axial loads, shears and bending moments over a given time increment

$[k]$ = the element stiffness matrix for the time increment

As discussed in Section 3, the total stiffness of an element is assumed to consist of an elasto-plastic component acting in parallel with an infinitely elastic component. Hence, the total stiffness of an element can be expressed as follows:

$$[k] = (p + q)[k] \quad (16)$$

where

$p[k]$ = the stiffness of the infinitely elastic component (k_{ie} in Fig 9b of text)

$q[k]$ = the stiffness of the elasto-plastic component (k_{ep} in Fig 9b of text)

$$p + q = 1$$

In most cases, the following quantities are utilized for the parameters p and q:

$$p = 0.05$$

$$q = 0.95$$

In expanded form, Equation (15) is rewritten as follows:

$$\begin{Bmatrix} \Delta P_A \\ \Delta V_A \\ \Delta M_A \\ \Delta P_B \\ \Delta V_B \\ \Delta M_B \end{Bmatrix} = \begin{bmatrix} k_1 & 0 & 0 & -k_1 & 0 & 0 \\ 0 & k_3 & k_2 & 0 & -k_3 & k_5 \\ 0 & k_2 & k_4 & 0 & -k_2 & k_6 \\ -k_1 & 0 & 0 & k_1 & 0 & 0 \\ 0 & -k_3 & -k_2 & 0 & k_3 & -k_5 \\ 0 & k_5 & k_6 & 0 & -k_5 & k_7 \end{bmatrix} \begin{Bmatrix} \Delta x_A \\ \Delta y_A \\ \Delta \theta_A \\ \Delta x_B \\ \Delta y_B \\ \Delta \theta_B \end{Bmatrix} \quad (17)$$

In the above equation, the incremental element loads vector, $\{\Delta f\}$, is expressed in terms of the incremental axial load, ΔP , incremental shear, ΔV , and incremental bending moment, ΔM ; likewise, the incremental displacement vector, $\{\Delta u\}$, is expressed in terms of the components of the incremental deformations in the local coordinate system for the element. The subscripts A and B refer to the ends of the element.

The stiffness coefficients, k_i , vary depending upon the yield and restraint conditions at the ends of the element. Expressions for computing these coefficients are provided in Table 1 for the various combinations of end conditions.

C.5 Computation of Plastic Component of Element End Rotations

The incremental plastic components of the relative element end rotations are computed as follows for the ten cases of end conditions specified in Table 1:

Cases 1, 2 and 3 - Elastic restraints at ends A and/or B and vice-versa:

$$\Delta \beta_A = \Delta \beta_B = 0$$

Case 4 - Yield at (A); elastic restraint at (B):

$$\Delta\beta_A = (\Delta\gamma_A - \Delta\alpha) + (1/2)(\Delta\gamma_B - \Delta\alpha)$$

$$\Delta\beta_B = 0$$

Case 5 - Yield at (A); pin at (B):

$$\Delta\beta_A = \Delta\gamma_A - \Delta\alpha$$

$$\Delta\beta_B = 0$$

Case 6 - Elastic restraint at (A); yield at (B):

$$\Delta\beta_A = 0$$

$$\Delta\beta_B = (\Delta\gamma_B - \Delta\alpha) + (1/2)(\Delta\gamma_A - \Delta\alpha)$$

Case 7 - Pin at (A); yield at (B):

$$\Delta\beta_A = 0$$

$$\Delta\beta_B = \Delta\gamma_B - \Delta\alpha$$

Cases 8 and 9 - Pin at (A) and (B), elastic and plastic conditions:

$$\Delta\beta_A = \Delta\beta_B = 0$$

Case 10 - Yield at (A) and (B):

$$\Delta\beta_A = \Delta\gamma_B - \Delta\alpha$$

$$\Delta\beta_B = \Delta\gamma_B - \Delta\alpha$$

Each of the ten conditions listed above is illustrated in Figures 2, 3 and 4.

C.6 Summary of Analysis Procedure

A summary of the overall analysis procedure is presented in terms of a logic flow diagram in Figure 5. The sequence of operations depicted is followed by program DYNFA.

C.7 Evaluation of Analysis Method

The validity of the analysis method and of program DYNFA was verified on the basis of a comparison with the known solutions for the response of the portal frame shown in Figure 6.

The comparison was made on the basis of the following:

1. An elastic analysis with MRI STARDYNE computer program (22).
2. An elasto-plastic analysis with INELAS 2D computer program (10).
3. An inelastic analysis with the DYNFA computer program.

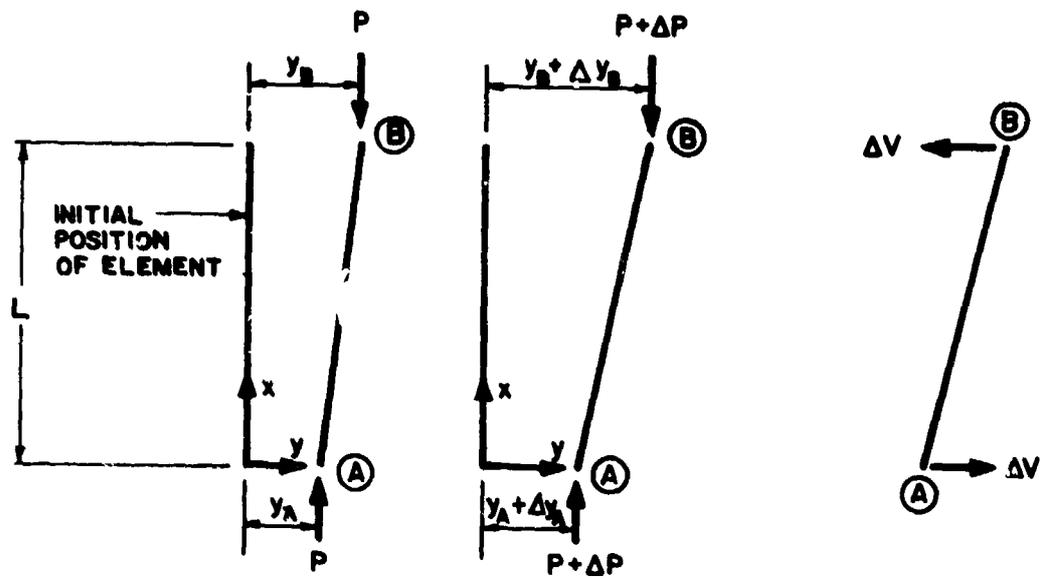
The results of each of these analyses are presented in terms of a plot of the horizontal displacement history of the frame (Fig 7). Inspection of the plotted displacement histories reveals the occurrence of inelastic action in the structure commencing at time 0.20 second and continuing throughout the remainder of the response. Note that there is good correlation between the DYNFA response and the INELAS 2D response, which is purported to be an exact solution for the problem. By virtue of this comparison, it can be concluded that the method of inelastic dynamic analysis implemented in DYNFA will produce a reasonably good prediction of the inelastic dynamic response of a frame structure.

Table 1
Element stiffness coefficients

Case No.	Member end conditions	k ₁	k ₂	k ₃	k ₄	k ₅	k ₆	k ₇
1	Elastic restraint, (A) and (B)	e	c	d	a	c	b	a
2	Elastic restraint at (A), pin at (B)	e	g	h	f	0	0	0
3	Pin at (A), elastic restraint at (B)	e	0	h	0	g	0	f
4	Yield at (A), elastic restraint at (B)	pe	pc	pd + qh	pa	pc + qg	pb	pa + qf
5	Yield at (A), pin at (B)	pe	pg	ph	pf	0	0	0
6	Elastic restraint at (A), yield at (B)	pe	pc + qg	pd + qh	pa + qf	pc	pb	pa
7	Pin at (A), yield at (B)	pe	0	ph	0	pg	0	pf
8	Pin at (A) and (B), elastic condition	e	0	0	0	0	0	0
9	Pin at (A) and (B), plastic condition	pe	0	0	0	0	0	0
10	Yield at (A) and (B)	pe	pc	pd	pa	pc	pd	pa

where $a = 4EI/L$; $b = 2EI/L$; $c = 6EI/L^2$; $d = 12EI/L^3$;

$e = AE/L$; $f = 3EI/L$; $g = 3EI/L^2$; $h = 3EI/L$

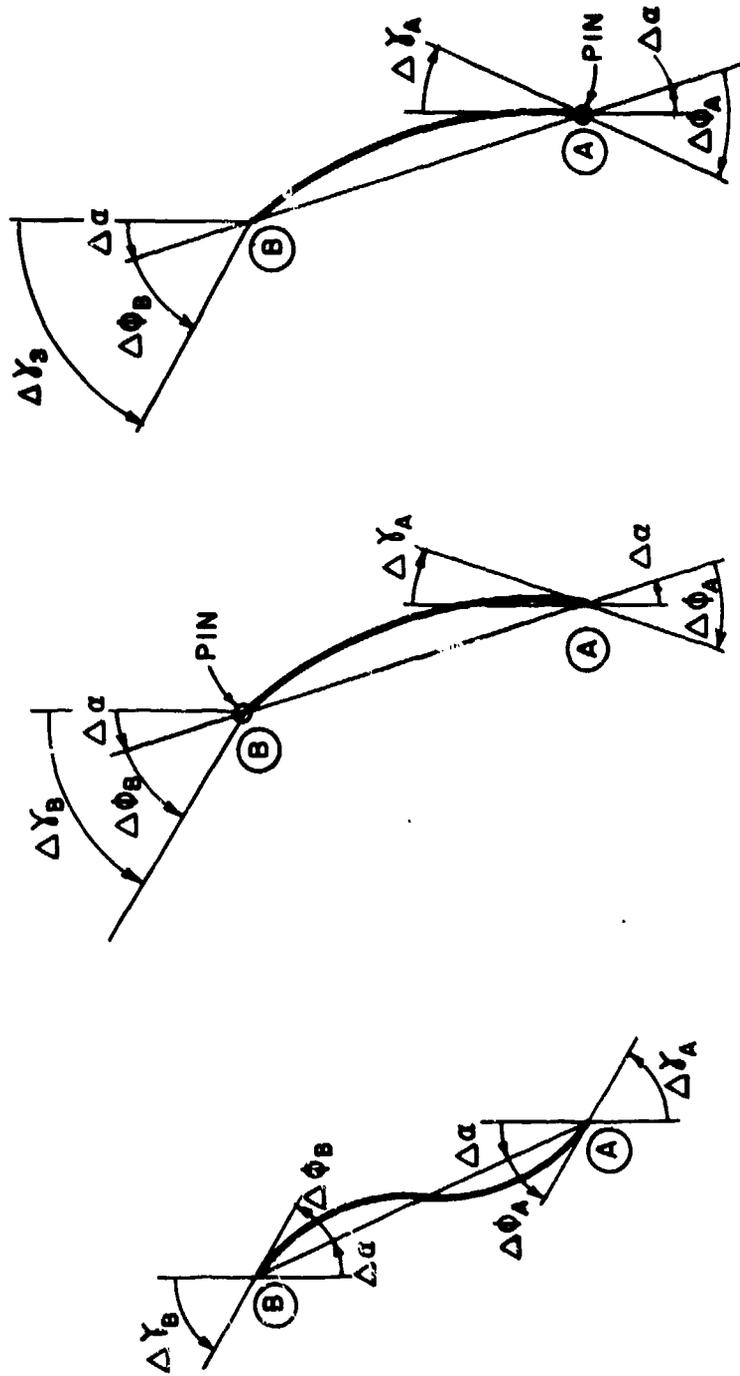


a) Position at
End of $(i-1)$ th
Increment

b) Position at
End of (i) th
Increment

c) Unbalanced
Incremental
Shear at
End of (i) th
Increment

Fig 1 Calculation of second order effects

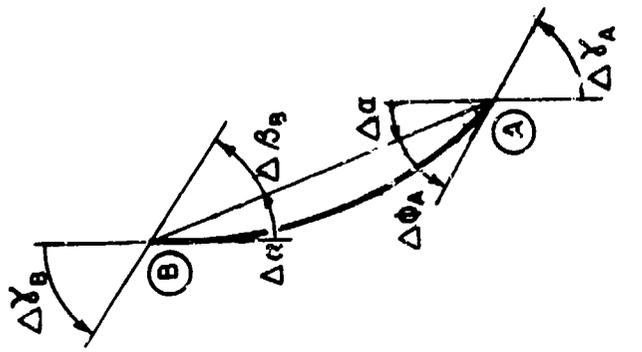


CASE 1

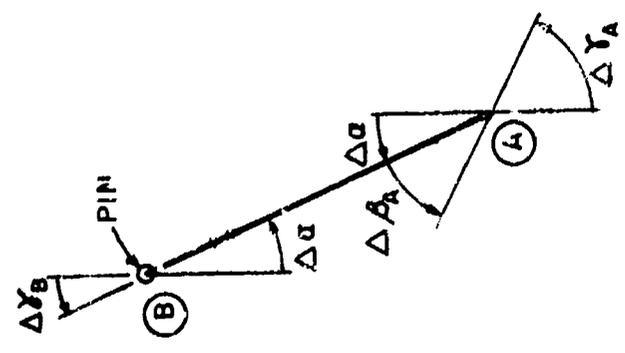
CASE 2

CASE 3

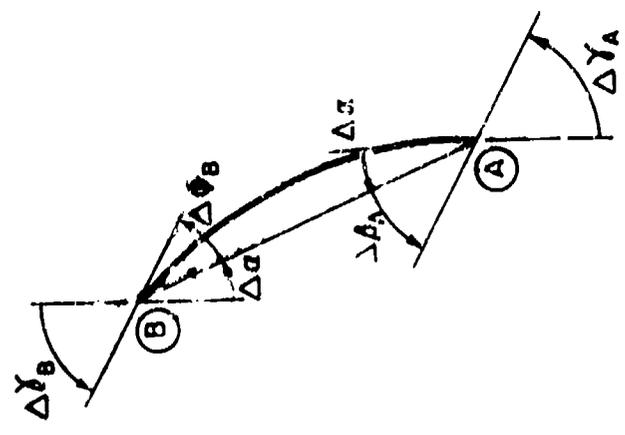
Fig 2 Components of differential element end rotation: yield cases 1, 2 and 3



CASE 6

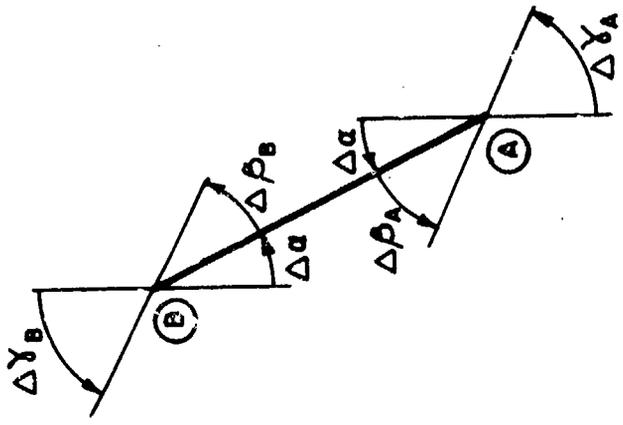


CASE 5

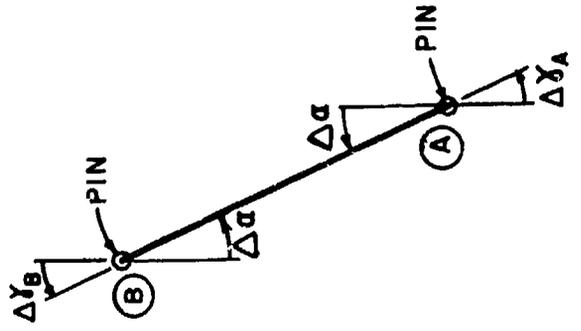


CASE 4

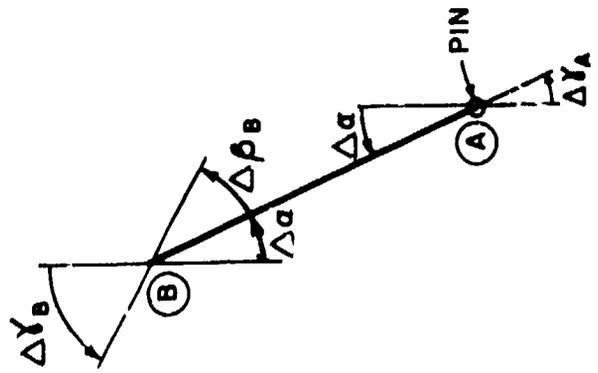
Fig 3 Components of differential element end rotation: yield cases 4, 5, 6



CASE 10



CASES 8 8 9



CASE 7

Fig 4 Components of differential element end rotation: yield cases 7, 8, 8, 9 and 10

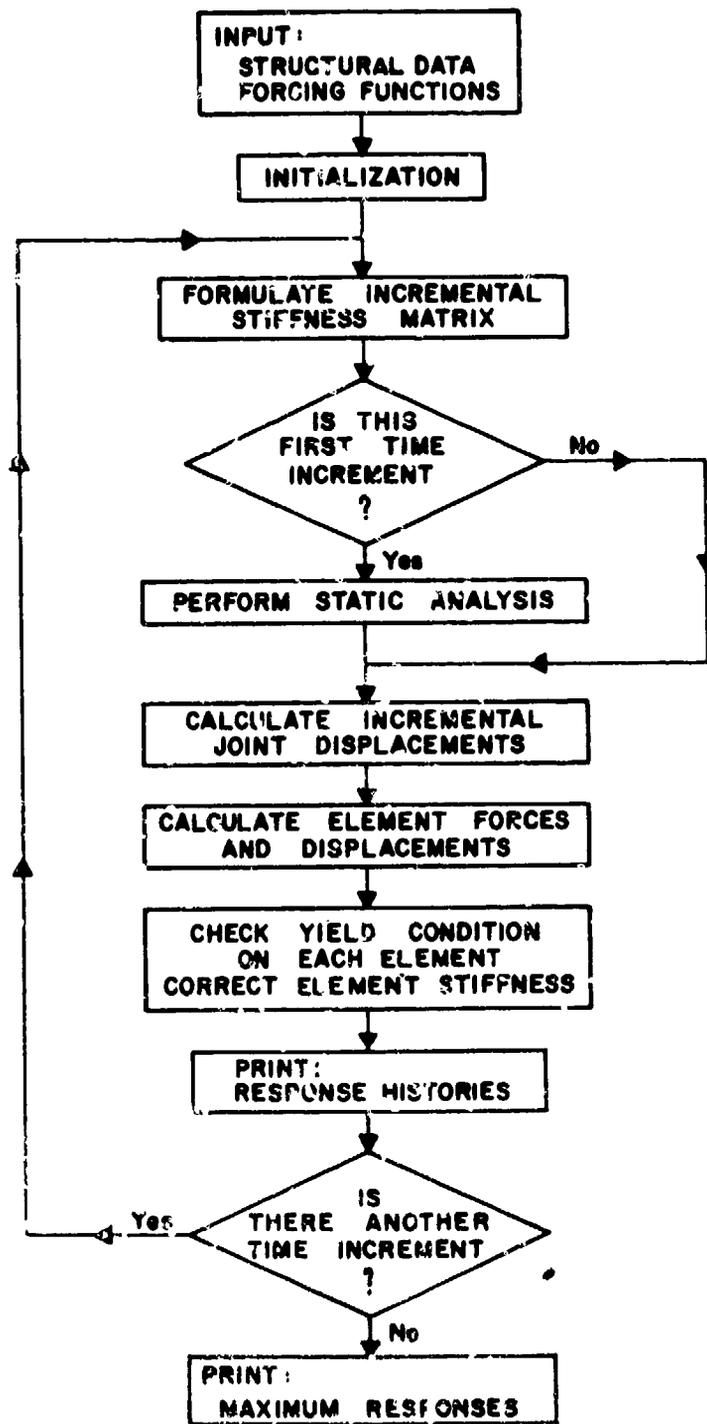


Fig 5 Summary of analysis procedure

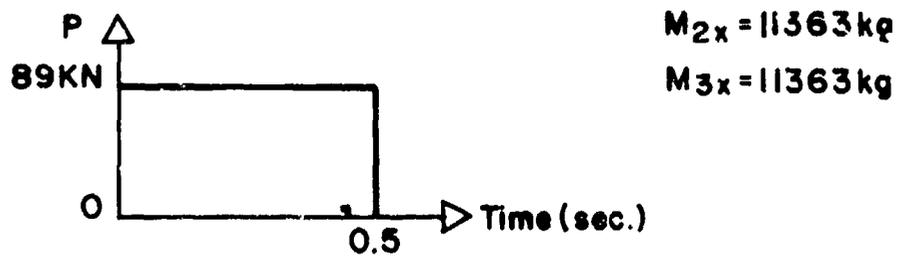
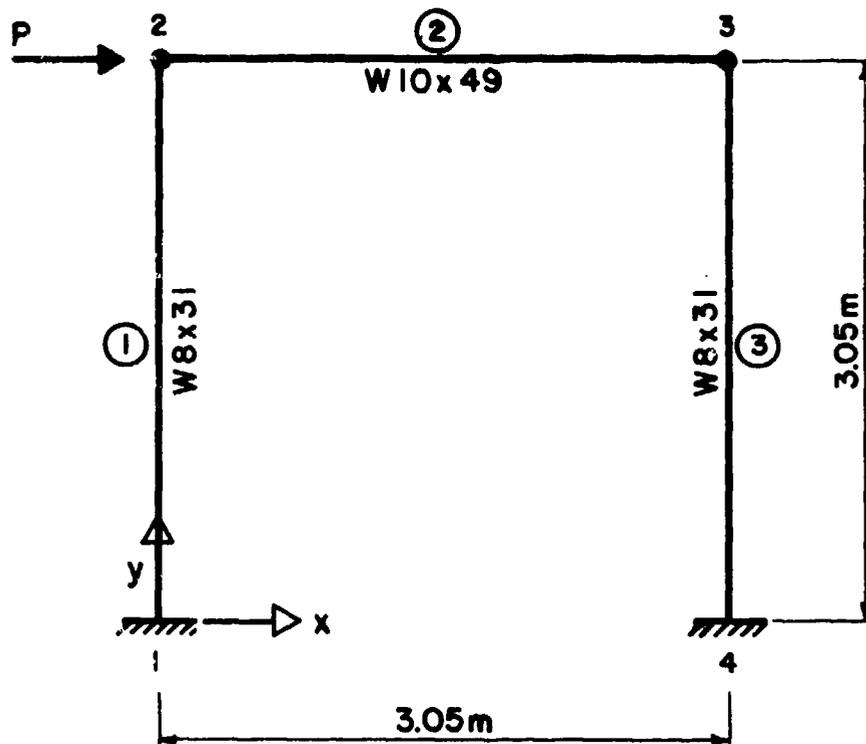


Fig 6 Portal frame analyzed with DYNFA, INELAS 2D and STARDYNE

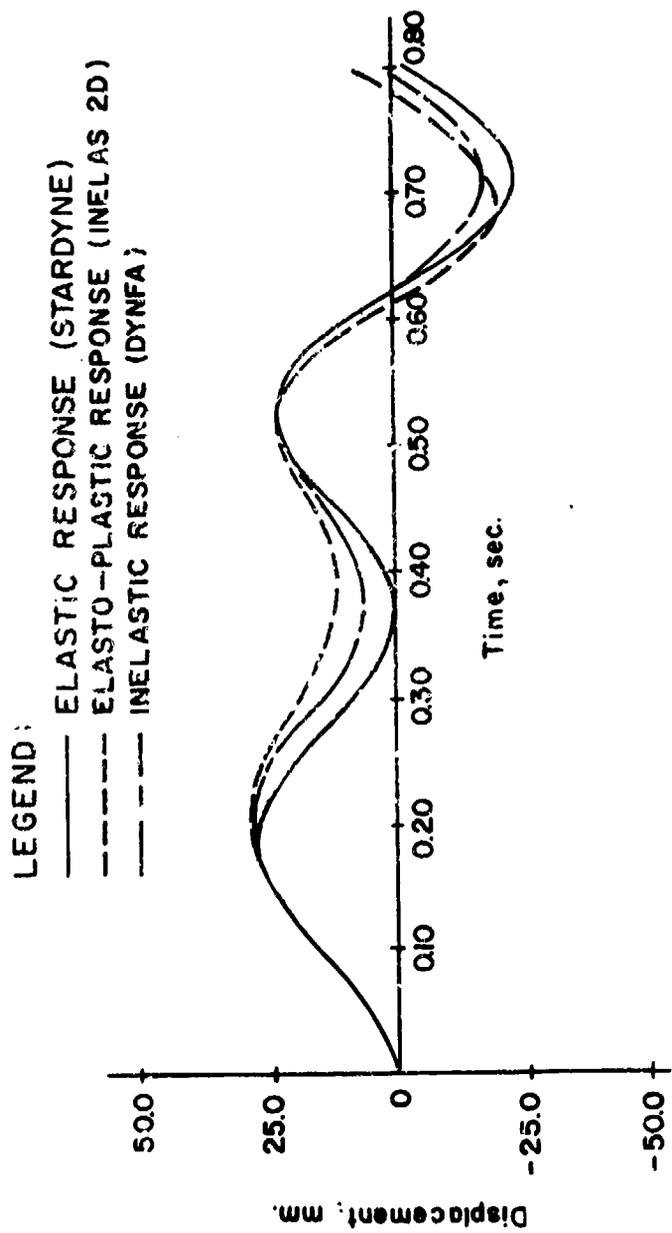


Fig 7 Comparison of DYNFA with INELAS 2D and STARDYNE

APPENDIX D

FORTRAN LISTING OF COMPUTER PROGRAM DYNFA

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FORTRAN LISTING OF COMPUTER PROGRAM DYNFA

D.1 General

This appendix contains the FORTRAN listing of computer program DYNFA. Preceding the listing are summaries of the functions performed by the mainline and all of the subroutines of the program.

D.2 Structure of Program

The computer program is written in FORTRAN IV for execution on the CDC 6600 computer using the Extended (FTN) FORTRAN compiler. A central memory field length of 170,000 words (octal) is required for execution of the program on this computer.

The program consists of a main routine and eleven subroutines. The operations performed by each are summarized below:

Main Routine DYNFA - The main routine initiates the execution and directs the overall sequence of operations performed in the frame analysis. To perform the required operations, the main routine calls the following subroutines:

1. Subroutine READ
2. Subroutine ZERO
3. Subroutine STIF
4. Subroutine PLOAD
5. Subroutine DYDIS
6. Subroutine ELEFD
7. Subroutine PRINT

Subroutine READ - This subroutine reads in and prints out the input data. In addition, the subroutine counts the total number of degrees-of-freedom as well as the number of dynamic degrees-of-freedom in the model. This subroutine also determines the line count parameter which establishes the amount of data printed on every page of the output.

Subroutine ZERO - This subroutine assigns a value of zero to all of the variables in the dimensioned storage arrays.

Subroutine STIF - Subroutine STIF generates the element and system stiffness matrices. The subroutine partitions the system stiffness matrix into the various submatrices utilized in the numerical integration routine. In addition, this subroutine forms the condensed stiffness matrix (see Ref 11) of the system. In performing these operations, the subroutine calls the two utility subroutines listed below:

1. Subroutine TRANS - Subroutine TRANS generates a transformation matrix for each element which orientates the local element coordinate system with respect to the global coordinate system.
2. Subroutine MINV - This subroutine performs a matrix inversion on a symmetric, non-singular matrix.

Subroutine PLOAD - This subroutine performs the dead and live load analysis of the frame. The subroutine utilizes subroutines MINV and TRANS in the computation. The results of the analysis are printed in the format shown in Appendix B.

Subroutine DYDIS - Subroutine DYDIS performs the numerical integration of the equations of motion of the system using the integration method outlined in Appendix C. The results of this integration consist of the accelerations of the dynamic degrees-of-freedom of the model. The displacements are computed as described in Appendix C. Subroutine DYDIS also records the maxima and minima of the nodal displacements. The time histories of the nodal displacements are also printed by this subroutine.

Subroutine ELEFD - This subroutine computes the element end loads using subroutine TRANS, and determines, using subroutine YIELD, whether the elements are in either the elastic or plastic condition. The subroutine also computes the element end rotations. The time histories of the element end loads and rotations are printed out by this subroutine. The peaks (maxima and minima) of these response quantities are extracted from the time histories.

Subroutine ELEFD also performs the $P-\Delta$ and beam column approximations by computing equivalent element shears using subroutine PDEL.

Summaries of the tasks performed by subroutines YIELD and PDEL are provided below:

1. Subroutine YIELD - This subroutine utilizes the interaction equations given in Section 3.3 to determine whether the element is in the elastic or plastic condition.
2. Subroutine PDEL - Subroutine PDEL computes the equivalent element shears using the method outlined in Appendix C.

Subroutine PRINT - This subroutine computes the ductility ratios for the elements of the model and prints out the tabulations of the significant response parameters described in Section 9.4.

The following pages contain the FORTRAN IV listing of program DYNFA.

```

PROGRAM DYNFA(INPUT,OUTPUT,TAPE2=INPUT,TAPES=OUTPUT)
C   NON-LINEAR DYNAMIC FRAME ANALYSIS
    DIMENSION UMAX(30,3),UMIN(30,3),TMAX(30,3),TMIN(30,3)
    DIMENSION R(60),RD(60),RDR(60),IIOLD(30,2)
    DIMENSION SAB(60,60),SAA(60,60),SBB(60,60)
    DIMENSION CMNP(30,2),CMNV(30,2),CMPP(30,2),CMPV(30,2),CLPM(30,2),
1CLPV(30,2),CLNH(30,2),CLNV(30,2)
    DIMENSION CMOP(30,2),CMOMN(30,2),CLOAD(30,2),THOMP(30,2),
1THOMN(30,2),TIMEL(30,2)
    DIMENSION PY(30),PCRY(30)
    DIMENSION YMY(30,2),
                                JA(30),
1JB(30),VL(30),CN(30),SN(30),A(30),AI(30),MPA(30),MPB(30),NINDF(60)
2,NOEDF(60),IOF( 90),
                                FAREA( 90),WMA(60),IMYAT(30)
3,ACEL(60),VELS(60),SEQ(60,60),DISP(60),U( 90),
4DU( 90),DDISP(60),SBB(60,60),SBA(60,60),DLTD(60,1),DDLTD(60,1),
5SEL(30,6,6),
                                ELMF(30,6),IMYA(30),IMYB(30),TBA(30),
6TBB(30),IMYBT(30),WUNIF(30),NSFOF( 90),NRX1(30),NRX2(30),NRX3(30)
    DIMENSION AK( 8100),MATX1( 90),MATX2( 90),PPP(30,20),TTT(30,20)
    DIMENSION SSY( 90, 90)
    DIMENSION ILASTA(30),ILASTB(30)
    DIMENSION TEBB(30),TEBA(30)
    DIMENSION RMAX(30,4),RMIN(30,4),TIM(30,4),TMM(30,4)
    DIMENSION TITLE(20)
    DIMENSION PYMAX(30,2),PYMIN(30,2)
    COMMON NUNIT,PYMAX,PYMIN
    COMMON ICOUN,NCOUN,TITLE
    COMMON RMAX,RMIN,TIM,TMM,TEBA,TEBB
    COMMON ILASTA,ILASTB,NELAS
    COMMON UMAX,UMIN,TMAX,TMIN
    COMMON NSKIP,INDEXM
    COMMON SAB,SAA,SBB
    COMMON R,RD,RDR,IIOLD
    COMMON PY,IPRD,IPRMP
    COMMON NDT,LL,NMEM,MYM,
                                CLOAD,CMOP,CMOMN,THOMP,THOMN,
1TIMEL,CMNP,CMNV,CMPP,CMPV,CLPM,CLPV,CLNH,CLNV
    COMMON NNODE,
                                JA,JB,VL,CN,SN,A,AI,E,MPA,MPB,NINDF,NOEDF,
1NINTL,NDETL,IOF,
                                FAREA,WMA,IMYAT,IMYBT,PPP,TTT,NWF,
2DT,ACEL,VELS,DAMP,SEQ,DISP,U,
                                DU,DDISP,SBB,SBA,
3DLTD,DDLTD,
                                SEL,
                                ELMF,
                                IMYA,IMYB,TBA,TBB
    COMMON YFACT,PCRY,SSY,NSFOF,AK,MATX1,MATX2
    COMMON WUNIF,NFS-NDIR,NRX1,NRX2,NRX3,NLNODE,NOEAD
    DO 500 NI=1,100
    NCOJN=0
C   *****
C   CALL SUBROUTINE READ TO READ INPUT FROM CARDS
C   *****
1 CALL READ
  NSIG=1
C   *****
C   CALL SUBROUTINE ZERO TO INITIALIZE VARIABLES FOR STATIC ANALYSIS
C   *****
  CALL ZERO(NSIG)
  IF(NOEAD)2,2,3
2 LL=0
C   *****
C   CALL SUBROUTINE STIF TO FORMULATE STIFFNESS MATRIX FOR STATIC
C   ANALYSIS

```

DYNFA

```

C *****
C CALL STIF
C *****
C CALL SUBROUTINE PLOAD TO PERFORM PLASTIC ANALYSIS
C *****
C CALL PLOAD(SSY, NSDF, NFS, SN, CV, VL, NDIR, MUNIF, NMEM, JA, JB, ELMF,
1 NNODE, CU, U, SEL, AK, MATX1, MATX2, MPA, MPB, NLNODE, TITLE, NCON, TEBB,
1 TEBA, ICOUN, NUNIT)
C NSIG=2
C *****
C CALL SUBROUTINE ZERO TO INITIALIZE VARIABLES AFTER STATIC ANALYSIS
C *****
C CALL ZERO(NSIG)
3 DO 4 J=1, NMEM
4 NSDF(J)=0
4 DO 45 J=1, NNODE
5 IF(NRX1(J) +NRX2(J))45,45,5
5 IF(NRX3(J))10,10,45
10 NCON=0
4 DO 20 K=1, NMEM
5 IF(J-JA(K))15,12,15
12 NEND=1
13 NCON=NCON+1
13 MATX1(NCON)=K
13 MATX2(NCON)=NEND
13 GO TO 20
15 IF(J-JB(K))20,14,20
14 NEND=2
14 GO TO 13
20 CONTINUE
20 IF(NCON-1)25,25,30
25 K=MATX1(NCON)
25 N=MATX2(NCON)
25 NSDF(K)=4
25 GO TO 45
30 NJ=0
30 DO 35 L=1, NCON
30 K=MATX1(L)
30 NPIN=MATX2(L)
30 GO TO(31,32), NPIN
31 IF(MPA(K)-1)35,33,35
32 IF(MPB(K)-1)35,33,35
33 NJ=NJ+1
35 CONTINUE
35 NDIF=NCON-NJ
35 IF(NDIF-1)45,39,45
39 DO 41 L=1, NCON
39 K=MATX1(L)
39 NPIN=MATX2(L)
39 GO TO(42,43), NPIN
42 IF(MPA(K)-1)44,41,41
43 IF(MPB(K)-1)44,41,41
44 NSDF(K)=NPIN
44 GO TO 45
41 CONTINUE
45 CONTINUE

```

DYNFA

```

INDEXW=0
NWRITE=NSKIP
DO 200 LL=1,NDT
IF((L-NWRITE)40,50,50
50 INDEXW=1
NWRITE=NWRITE+ NSKIP
GO TO 300
40 INDEXW=0
300 IF((LL-1)350,350,325
325 DO 100 K=1,NMEM
NLASA=IMYAT(K)-ILASTA(K)
NLASB=IMYBT(K)-ILASTB(K)
IF(NLASA)350,80,350
80 IF(NLASB)350,100,350
100 CONTINUE
GO TO 400
C *****
C CALL SUBROUTINE STIF TO FORMULATE STIFFNESS MATRIX FOR DYNAMIC
C ANALYSIS
C *****
350 CALL STIF
C *****
C CALL SUBROUTINE DYDIS TO SOLVE EQUATIONS OF MOTION
C *****
400 CALL DYDIS
C *****
C CALL SUBROUTINE ELEFD TO COMPUTE ELEMENT LOADS AND END ROTATIONS
C (ELASTIC AND PLASTIC)
C *****
CALL ELEFD
200 CONTINUE
C *****
C CALL SUBROUTINE PRINT TO PRINT TABULATIONS OF MAXIMA AND MINIMA OF
C ELEMENT LOADS, ELEMENT END ROTATIONS AND NODAL DISPLACEMENTS
C *****
CALL PRINT
500 CONTINUE
CALL EXIT
END

```

```

SUBROUTINE READ
READ AND PRINT SUBROUTINE
DIMENSION UMAX(30,3),UMIN(30,3),THAX(30,3),THIN(30,3)
DIMENSION R(60),RD(60),RDR(60),IIOLD(30,2)
DIMENSION SAB(60,60),SAA(60,60),SBB(60,60)
DIMENSION CMNP(30,2),CMNV(30,2),CMPP(30,2),CMPV(30,2),CLPH(30,2),
1CLPV(30,2),CLNH(30,2),CLNV(30,2)
DIMENSION CMHP(30,2),CMCHN(30,2),CLOAD(30,2),THOMP(30,2),
1THOMN(30,2),TIMEL(30,2)
DIMENSION PY(30),PCRY(30),YMY(30,2)
DIMENSION X(30),Y(30),NRX1(30),NRX2(30),NRX3(30),WX(30),WY(30),
1JA(30),JB(30),MPA(30),MPB(30),A(30),AI(30),          CN(30),SN(30),
2VL(30),PPP(30,20),TTT(30,20)          ,FAREA( 90),IDF( 90),TITLE(20),
3NOR( 90),NFSD( 90),NBDF(30),W( 90),NINDF(60),WNAS(60),NOEDF(60)
DIMENSION IMYAT(30),IMYBT(30),ACEL(60),VELS(60),SEQ(60,60),
1DISP(60),U( 90),          DU( 90),DDISP(60),SBB(60,60),
2SBA(60,60),DLTD(60,1),DDLTD(60,1),SEL(30,6,6),
3ELMF(30,6),IMYA(30),IMYB(30),TBA(30),TBB(30),
4          SSY( 90, 90),MUNIF(30)
DIMENSION AK( 8100),MATX1( 90),MATX2( 90)
DIMENSION ILASTA(30),ILASTB(30)
DIMENSION TEBB(30),TEBA(30)
DIMENSION RMAX(30,4),RMIN(30,4),TIM(30,4),TMM(30,4)
DIMENSION NSIGN(2),IXORY(2)
DIMENSION EMOD(2),UNIT(2),UNWG(2),BUNIT(2,4),AUNIT(2),UNFUN(2),
1 G(2)
DIMENSION PYMAX(30,2),PYMIN(30,2)
COMMON NUNIT,PYMAX,PYMIN
COMMON ICOUN,NCOUN,TITLE
COMMON RMAX,RMIN,TIM,TMM,TEBA,TEBB
COMMON ILASTA,ILASTB,NELAS
COMMON UMAX,UMIN,THAX,THIN
COMMON NSKIP,INDEXW
COMMON SAB,SAA,SBB
COMMON R,RD,RDR,IIOLD
COMMON PY,IPRD,IPRMP
COMMON NDT,LL,NMEM,YMY,          CLOAD,CMHP,CMCHN,THOMP,TOMN,
1TIMEL,CMNP,CMNV,CMPP,CMPV,CLPH,CLPV,CLNH,CLNV
COMMON NNODE,          JA,JB,VL,CN,SN,A,AI,E,MPA,MPB,NINDF,NOEDF,
1NINTL,NOETL,IDF,          FAREA,WNAS,IMYAT,IMYBT,PPP,TTT,NWF,
2DT,ACEL,VELS,DAMP,SEQ,DISP,U,          DU,DDISP,SBB,SBA,
3DLTD,DDLTD,          SEL,          ELMF,          IMYA,IMYB,TBA,TBB
COMMON YFACT,PCRY,SSY,NFSD,AK,MATX1,MATX2
COMMON MUNIF,NFS,NDIR,NRX1,NRX2,NRX3,NLNODE,NDEAD
DATA SIGNAL/4HEND /
DATA NSIGN/1H ,1H-/
DATA IXORY/1HX,1HY/
DATA G/1000.0,386.0/
DATA LUNIT/2HUS/
DATA EMOD/4H KPA,4H PSI/
DATA UNIT/4H (M),4H(IN)/
DATA UNWG/4H(KG),4H(LB)/
DATA BUNIT/7H(CM**2),7H(IN**2),7H(CM**4),7H(IN**4),
1 8H (KN-M),8H (LB-IN),5H(KNS),5H(LBS)/
DATA AUNIT/7H(M**2),7H(IN**2)/
DATA UNFUN/8H (KGS/M),8H(LBS/IN)/
NUNIT=1

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READ

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WRITE(5,495)
495 FORMAT(1H1)
C *****
C READ STRUCTURE DESCRIPTION CARD - CARD TYPE 1 - CHECK FOR END OF
C DATA INDICATOR
C *****
READ(2,500) TITLE
IF(TITLE(1).EQ.SIGNAL)CALL EXIT
WRITE(5,500) TITLE
500 FORMAT(20A4)
C *****
C READ PROBLEM SPECIFICATION CARD - CARD TYPE 2
C *****
READ(2,505)NMEN,NNODE,NNOR,NNOW,NNOF,IPRJ,IPRMP,E ,YFACT,NELAS.
1 NDEAD,NLNODE,NU
505 FORMAT(7I5,2F10.0,3I5,3X,A2)
IF(NU.EQ.LUNIT)NUNIT=2
IF(NUNIT.EQ.2)GO TO 1001
WRITE(5,1000)
WRITE(5,1006)
WRITE(5,1004)
GO TO 1002
1001 WRITE(5,1003)
WRITE(5,1005)
WRITE(5,1007)
1000 FORMAT(/,
1 //, 37X,12HUNITS-METRIC,/,37X,12H-----,
2 37X,11HBASIC UNITS,/,
3 35X,17HLENGTH - METRE(M),/,
4 37X,19HMASS - KILOGRAM(KG),/,
5 37X,18HTIME - SECOND(SEC),/,
6 18X,50HUNITS FOR ELEMENT PROPERTIES, CAPACITIES AND LOADS,/,
7 37X,32HAREA - SQUARE CENTIMETERS(CM**2),/,
8 24X,58HMOMENT OF INERTIA - CENTIMETERS TO THE FOURTH PO
9 WER(CM**4),/,21X,39HAXIAL LOAD AND SHEAR - KILONENTONS(KNS),/,
10 27X,41HBENDING MOMENT - KILONENTON-METER(KN-M) )
1006 FORMAT(/,
1 36X,13HAPPLIED LOADS,/,
2 33X,26HPRESSURE - KILOPASCAL(KPA),/,
3 27X,36HTRIBUTARY AREA - SQUARE METRES(M**2),/,
4 21X,44HMEMBER UNIFORM LOADS - KILOGRAMS/METRE(KG/M),/,
5 20X,37HSTATIC LOADS AT NODES - KILOGRAMS(KG),/,
6 18X,47HSTATIC MOMENTS AT NODES - KILOGRAM-METER(KG-M) ,/,
7 39X,9HCONSTANTS,/,
8 18X,55HACCELERATION OF GRAVITY - 9.8 METRES PER SECOND SQUARE
9 80./, 20X,40HMODULUS OF ELASTICITY - KILOPASCALS(KPA),/,22X,42H*
10 9NOTE - APPLIED STATIC LOADS IN MASS UNITS)
1004 FORMAT(/,37X,
1 11HDEFLECTIONS,/,
2 22X,36HNODAL DISPLACEMENTS - MILLIMETRES(MM),/,10X,
3 46HNODAL AND ELEMENT END ROTATIONS - DEGREES(DEG))
1003 FORMAT(/,
1 37X, 8HUNITS-US,/,37X,8H-----,
2 //, 37X,11HBASIC UNITS,/,
3 35X,19HLENGTH - INCHES(IN),/,
4 35X,20HWEIGHT - POUNDS(LBS),/,
5 37X,18HTIME - SECOND(SEC),/,
6 18X,50HUNITS FOR ELEMENT PROPERTIES, CAPACITIES AND LOADS,/,
7 37X,27HAREA - SQUARE INCHES(IN**2),/,
8 24X,53HMOMENT OF INERTIA - INCHES TO THE FOURTH POWER(I

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                                READ
      8N**4),/,21X,34HAXIAL LOAD AND SHEAR - POUNDS(LBS),/,
      9      2X,36HBENDING MOMENT - POUND-INCH(LB-IN) )
1007 FORMAT(/,
      1      36X,13HAPPLIED LOADS,/,
      2      33X,38HPRESSURE - POUNDS PER SQUARE INCH(PSI),
      3      27X,37HTRIBUTARY AREA - SQUARE INCHES(IN**2),/,
      4      21X,46HMEMBER UNIFORM LOADS - POUNDS PER INCH(LBS/IN),/,
      5      20X,35HSTATIC LOADS AT NODES - POUNDS(LBS),/,
      6      18X,45HSTATIC MOMENTS AT NODES - POUND-INCH(LB-IN) ),/,
      7      39X,9HCONSTANTS,/,
      8      18X,57HACCELERATION OF GRAVITY - 386.4 INCHES PER SECOND SQUA
      9RED,/, 20X,51HMODULUS OF ELASTICITY - POUNDS PER SQUARE INCH(PSI))
1005 FORMAT(/,37X,
      1      11HDEFLECTIONS,/,
      2      22X, 32HNODAL DISPLACEMENTS - INCHES(IN),/,10X,
      3      2 46HNODAL AND ELEMENT END ROTATIONS - DEGREES(DEG))
1002 WRITE(5,495)
      WRITE(5,500)TITLE
C *****
C IPRD=1 PRINT DISPL. IPRMP=1 PRINT M-V-P
C *****
      1 WRITE(5,510)NKEM
      510 FORMAT(/,5X,34HNUMBER OF ELEMENTS = ,I2)
      WRITE(5,512)NNODE
      512 FORMAT(5X,34HNUMBER OF NODES = ,I2)
      WRITE(5,514)NNOR
      514 FORMAT(5X,34HNUMBER OF NODES WITH RESTRAINTS = ,I2)
      WRITE(5,516)NNOW
      516 FORMAT(5X,34HNUMBER OF NODES WITH MASSES = ,I2)
      WRITE(5,518)NNOF
      518 FORMAT(5X,34HNUMBER OF NODES WITH FORCES = ,I2)
      WRITE(5,519)IPRD
      519 FORMAT(5X,34HDISPLACEMENTS PRINTING INDEX = ,I2)
      WRITE(5,517)IPRMP
      517 FORMAT(5X,34HMOH,AXIAL L,SHEAR PRINTING INDEX = ,I2)
      WRITE(5,520)E,EMOD(NUNIT)
      520 FORMAT(5X,34HMODULUS OF ELASTICITY = ,F12.0,1X,A4)
      IF(YFACT)5,5,6
      5 YFACT=.05
      6 WRITE(5,521)YFACT
      521 FORMAT(5X,46HPERCENTAGE OF ELASTIC STIFFNESS AFTER YIELD = ,F6.3)
      DO 10 I=1,NNODE
C *****
C READ NODAL COORDINATES CARDS - CARD TYPE 3
C *****
      READ(2,525)NT,XT,YT
      X(NT)=XT
      Y(NT)=YT
      10 CONTINUE
      WRITE(5,530)
      530 FORMAT(1H0,8X,31H *** NODAL COORDINATE TABLE ***)
      WRITE(5,532)UNIT(NUNIT),UNIT(NUNIT)
      532 FORMAT(1H0,3X,4HNODE,13X,1HX,14X,1HY,/, 19X,A4,1X,A4/)
      600 FORMAT(1H )
      WRITE(5,534)(I,X(I),Y(I),I=1,NNODE)
      534 FORMAT(1H ,4X,I3,2F15.2)
      DO 15 I=1,NNODE
      NRX1(I)=0

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READ

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NRX2(I)=0
NRX3(I)=0
15 CONTINUE
DO 20 K=1,NNOR
C *****
C READ NODAL RESTRAINTS CARDS - CARD TYPE 4
C *****
READ(2,535)NT,NT1,NT2,NT3
NRX1(NT)=NT1
NRX2(NT)=NT2
NRX3(NT)=NT3
20 CONTINUE
525 FORMAT(I10,2F10.0)
535 FORMAT(4I10)
WRITE(5,540)
540 FORMAT(1H0,10X,27H***NODAL RESTRAINT TABLE***)
WRITE(5,545)
545 FORMAT(1H0,3X,4HNODE,7X,1HX,9X,1HY,4X,8HROTATION)
WRITE(5,550)(I,NRX1(I),NRX2(I),NRX3(I),I=1,NNODE)
WRITE(5,600)
550 FORMAT(3X,I4,5X,I4,6X,I4,6X,I4)
IF(NJNIT.EQ.2)GO TO 551
WRITE(5,556)
GO TO 557
556 FORMAT(1H0,10X,30H***INPUT NODAL MASS TABLE***)
551 WRITE(5,555)
557 WRITE(5,560)UNWG(NUNIT),UNWG(NUNIT)
560 FORMAT(1H0,3X,4HNODE,13X,2HMX,13X,2HMY,/, 20X,A4,11X,A4/)
555 FORMAT(1H0,10X,30H***INPUT NODAL WEIGHT TABLE***)
DO 25 K=1,NNODE
WX(K)=0.0
WY(K)=0.0
25 CONTINUE
DO 30 K=1,NNOW
C *****
C READ NODAL MASS CARDS - CARD TYPE 5
C *****
READ(2,525)NT,T1,T2
WX(NT)=T1
WY(NT)=T2
WRITE(5,565)NT,T1,T2
30 CONTINUE
565 FORMAT(4X,I3,3X,2F15.2)
WRITE(5,570)
570 FORMAT(1H0,9X,19H***ELEMENT TABLE***)
WRITE(5,575)(BUNIT(NUNIT,N),N=1,3),BUNIT(NUNIT,3),BUNIT(NUNIT,4),
1 BUNIT(NUNIT,4)
575 FORMAT(1H0,3X,4HELEM,4X,2HJA,4X,2HJB,2X,8HPIN CODE,9X,4HAREA,11X,
12H I,15X,3HMA,12X,3HMB,13X,2HPP,12X,2HPU,/,
2 38X,A7,7X,A7,9X,A8,8X,A8,
38X,A5,10X,A5)
WRITE(5,600)
DO 35 M=1,NMEM
C *****
C READ ELEMENT CARDS - CARD TYPE 6
C *****
```

```

                                READ
READ(2,580)IBT,JAT,JBT,MPAT,MPBT,AT,TI,THYA,THYB,TPY,TPCR
JA(IBT)=JAT
JB(IBT)=JBT
MPA(IBT)=MPAT
MPB(IBT)=MPBT
34 A(IBT)=AT
   AI(IBT)=TI
   YMY(IBT,1)=THYA
   YMY(IBT,2)=THYB
   IF(THYB.EQ.0.0)YMY(IBT,2)=THYA
   PY(IBT)=TPY
   PCRY(IBT)=TPCR
35 CONTINUE
580 FORMAT(3I5,3X,2I1,6F10.0)
      WRITE(5,585)(M,JA(M),JB(M),MPA(M),MPB(M),A(M),AI(M),
1  (YMY(M,J),J=1,2),PY(M),PCRY(M),M=1,NMEM)
585 FORMAT(5X,I3,3X,I3,3X,I3,4X,I1,4X,I1,6F15.2)
C *****
C FORMATION OF STIFFNESS FACTORS
C *****
DO 40 M=1,NMEM
IF(NUNIT.EQ.2)GO TO 39
A(M)=0.0001*A(M)
AI(M)=1.0E-08*AI(M)
39 MA=JA(M)
   MB=JB(M)
   XX=X(MB)-X(MA)
   YY=Y(MB)-Y(MA)
   SPAN=SQRT(XX**2+YY**2)
   SN(M)=YY/SPAN
   CN(M)=XX/SPAN
   VL(M)=SPAN
40 CONTINUE
C *****
C READ DAMPING CARD - CARD TYPE 7
C *****
READ(2,590)DAMP
590 FORMAT(F10.0)
WRITE(5,595) DAMP
595 FORMAT(1H0,4X,15HDAMPING FACTOR=,F10.5)
C *****
C READ INTEGRATION TIME CARD - CARD TYPE 8
C *****
READ(2,605)NDT,DT,NSKIP
605 FORMAT(5X,I5,F10.0,I10)
DT1=NSKIP*DT
WRITE(5,606)DT,DT1,NDT
606 FORMAT(1H0,4X,26HINTEGRATION TIME INTERVAL=,F15.8,26H SEC, PRINT T
1IME INTERVAL=,F15.8,27H SEC, NJMBER OF INCREMENTS=,I5)
C *****
C READ LOADING SPECIFICATION CARD - CARD TYPE 9
C *****
READ(2,610)NMF
610 FORMAT(2I10)
WRITE(5,612)
612 FORMAT(1H0,10X,24H***DYNAMIC LOAD TABLE***)

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```

                                READ
DO 60 I=1,NWF
C *****
C READ WAVEFORM SPECIFICATION CARD - CARD TYPE 10
C *****
READ(2,610)NWFT,NPTT
WRITE(5,615)NWFT,NPTT
615 FORMAT(1HG,4X,1.9HPRESSURE WAVEFORM ,11,5X,13HNO.OF POINTS=,I3)
WRITE(5,620)EMOD(NUNIT)
620 FORMAT(1HG,21X,1HP,13X,1HT,/, 20X,A4,11X,3HSEC/)
DO 50 J=1,NPT
C *****
C READ P-T CARDS - CARD TYPE 11
C *****
READ(2,625)PTM,TM
PPP(I,J)=PTM
TTT(I,J)=TM
50 CONTINUE
WRITE(5,630)(PPP(I,M),TTT(I,M),M=1,NPTT)
50 CONTINUE
625 FORMAT(2F10.0)
630 FORMAT(10X,F15.2,F15.6)
WRITE(5,635)
635 FORMAT(1HG,10X,21H** TRIBUTARY AREAS **)
WRITE(5,640)AUNIT(NUNIT),AUNIT(NUNIT)
640 FORMAT(1HG,3X,4HNODE,12X,2HAX,6X,2HID, 12X,2HAY,6X,2HID,/,
1 18X,A7,15X,A7)
WRITE(5,640)
N3=3*NNOE
DO 70 K=1,N3
FAREA(K)=0.0
70 IDF(K)=0
DO 75 K=1,NNOF
C *****
C READ TRIBUTARY AREAS CARDS - CARD TYPE 12
C *****
READ(2,650)NTEM,FXT,IDX,FYT,IDYT
WRITE(5,655)NTEM,FXT,IDX,FYT,IDYT
NXT=3*NTEM-2
NYT=3*NTEM-1
FAREA(NXT)=FXT
FAREA(NYT)=FYT
IDF(NXT)=IDX
IDF(NYT)=IDYT
75 CONTINUE
650 FORMAT(I10,F10.0,I10,F10.0,I10)
655 FORMAT(5X,I3,F15.2,I7,F15.2,I7)
IF(NDEAD)76,76,77
76 WRITE(5,701)UNFUN(NUNIT)
C *****
C READ ELEMENT UNIFORM LOADS DIRECTION INDICATOR CARD - CARD TYPE 13
C *****
READ(2,581)ISIGN,DIR
581 FORMAT(2A1)
IF(ISIGN,NE,NSIGN(1))GO TO 71
NMULT=1
GO TO 72

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READ

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71 IF(ISIGN.NE.NSIGN(2))CALL EXIT
    NMULT=-1
72 IF(IDIR.NE.IXORY(1))GO TO 73
    NDIR=NMULT
    GO TO 74
73 IF(IDIR.NE.IXORY(2))CALL EXIT
    NDIR=2*NMULT
74 ADIR=NDIR
    ADIR=ADIR/ABS(ADIR)
    DO 700 I=1,NMEM
C *****
C READ ELEMENT UNIFORM STATIC LOADS CARDS - CARD TYPE 14
C *****
    READ(2,65C)N,WGT
    WUNIF(N)=ADIR*WGT
    IF(NJNIT.EQ.1)WUNIF(N)=J.(098*WUNIF(N)
700 WRITE(5,655)N,WGT
701 FORMAT(///1X,24HDEAD LOAD PLUS LIVE LOAD//,5X,4HELEM,5X,12HUNIFORM
    1 LOAD,/,16X,A8//)
C *****
C FORMULATION OF DOF
C *****
77 DO 80 I=1,NNODE
    N=3*I
    NOR(N-2)=NRX1(I)
    NOR(N-1)=NRX2(I)
    NOR(N)=NRX3(I)
80 CONTINUE
    NFS=0
    NBD=0
    NDTOT=3*NNODE
    DO 85 L=1,NDTOT
    IND=NOR(L)
    IF(IND-1) 90,83,83
90 NFS=NFS+1
    NFSDF(NFS)=L
    GO TO 85
83 NBD=NBD+1
    NBDF(NBD)=L
85 CONTINUE
C *****
C TOTAL NUMBERS OF FINAL STATIC DOF (NFSTL), BOUNDARY DOF(NBDTL)
C *****
    NBDTL=NBD
    NFSTL=NFS
C *****
C FORMULATE DYNAMIC DOF
C *****
    DO 95 I=1,NNODE
    N=3*I
    W(N-2)=WX(I)
    W(N-1)=WY(I)
    W(N)=0.0
95 CONTINUE
    NIN=0
    NDE=0
```

READ

```
DO 100 M=1,NFSTL
MTEMP=NFSDF(M)
WMT=W(MTEMP)/G(NUNIT)
IF(WMT-3.0) 105,105,113
105 NDE=NDE+1
NDEJF(NDE)=NFSDF(M)
GO TO 100
110 NIN=NIN+1
NINJF(NIN)=NFSDF(M)
WMAS(NIN)=WMT
100 CONTINUE
NDETL=NDE
NINTL=NIN
*****
C TOTAL NUM ERS OF IND.DYNAMIC DOF(NINTL),DEPT.DYNAMIC DOF(NJETL)
C *****
C *****
C DETERMINE PAGE SKIP PARAMETER
C *****
NN=IPRD + IPRNP
IF(NN)220,220,205
205 GO TO(206,212),NN
206 IF(IPRD-1)208,207,207
207 ICOJN=60/(NNOE + 3)
GO TO 215
208 IF(NELAS)209,209,210
209 ICOJN=60/(2*NMEM + 5)
GO TO 215
210 ICOJN=60/(NMEM + 3)
GO TO 215
212 IF(NELAS)213,213,214
213 ICOJN=60/(NNOE + 2*NMEM + 7)
GO TO 215
214 ICOJN=60/(NNOE + NMEM + 4)
215 IF(ICOJN)217,217,218
217 ICOJN=1
218 IF(NJEAD)220,220,216
216 WRITE(5,230)TITLE
230 FORMAT(1H1,29X,18A4,/,1X,16HRESPONSE HISTORY/)
220 RETURN
END
```

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SUBROUTINE ZERO(NSIG)
*****
C INITIAL CONDITIONS
C *****
C
DIMENSION UMAX(30,3),UMIN(30,3),TMAX(30,3),TMIN(30,3)
DIMENSION R(60),RO(60),RF(60),IIOLD(30,2)
DIMENSION SAB(0,60),SAA(0,60),SBB(60,60)
DIMENSION CMNP(30,2),CMNV(30,2),CMPP(30,2),CMPV(30,2),CLPM(30,2),
1CLPV(30,2),CLNM(30,2),CLNV(30,2)
DIMENSION CMOM(30,2),CMOMN(30,2),CLOAD(30,2),T4OMP(30,2),
1TMOMN(30,2),TIMEL(30,2)
DIMENSION PY(30),PCRY(30)
DIMENSION DU( 90),VELS(60),DISP(60),IDF( 90),NINDF(60),
1 ACEL(60),FAREA( 90),WMAS(60),PPP(30,20),TTT(30,20),
2 IMYA(30),IMYR(30),IMYAT(30),IMYBT(30),TBA(30),TBB(30),
3 ELMF(30,6),DDISP(60)
DIMENSION JA(30),JB(30),VL(30),CN(30),SN(30),A(30),AI(30),
1MPA(30),MPB(30),NOEDF(60),SEQ(60,60),SBB(60,60),
2SJA(60,60),DLTD(60,1),DDLTD(60,1),SEL(30,6,6),
3YMY(30,2), SSY( 90, 90),NSFLF( 90),WUNIF(30)
DIMENSION AK( 8100),MATX1( 90),MATX2( 90)
DIMENSION ILASTA(30),ILASTR(30)
DIMENSION TEBB(30),TEBA(30)
DIMENSION TITLE(20),NRX1(30),NRX2(30),NRX3(30)
DIMENSION RMAX(30,4),RMIN(30,4),TIM(30,4),TMM(30,4)
DIMENSION PYMAX(30,2),PYMIN(30,2)
COMMON NUNIT,PYMAX,PYMIN
COMMON ICOUN,NCOUN,TITLE
COMMON RMAX,RMIN,TIM,TMM,TEBA,TEBB
COMMON ILASTA,ILASTR,NELAS
COMMON UMAX,UMIN,TMAX,TMIN
COMMON NSKIP,INDEXW
COMMON SAB,SAA,SBB
COMMON R,RO,RJR,IIOLD
COMMON PY,IPRO,IPRMP
COMMON NDT,LL,NMEM,YMY, CLOAD,CMOMP,CMOMN,T4OMP,TMOMN,
1TIMEL,CMNP,CLNV,CMPP,CMPV,CLPM,CLPV,CLNM,CLNV
COMMON NNODE, JA,JB,VL,CN,SN,A,AI,E,MPA,MPB,NINDF,NOEDF,
1NINTL,NOETL,IDF, FAREA,WMAS,IMYAT,IMYBT,PPP,TTT,NWF,
2DT,ACEL,VELS,CAMP,SEQ,DISP,U, DU,DDISP,SBB,SBA-
3DLTD,DDLTD, SEL, ELMF, IMYA,IMYR,TBA,TBB
COMMON YFACT,PCRY,SSY,NSFLF,AK,MATX1,MATX2
COMMON WUNIF,NFS,NDIR,NRX1,NRX2,NRX3
N3=3*NNOGE
DO 100 N=1,N3
GO TO(99,100),NSIG
99 U(N)=0.0
100 DU(N)=J.C
DO 200 N=1,NINTL
VELS(N)=0.0
KK=NINDF(N)
GO TO(101,111),NSIG
101 DISP(N)=0.0
GO TO 120
111 DISP(N)=U(KK)
120 K=IDF(KK)
IF(K) 210,210,220

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ZERO

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210 PDIF=0.0
    GO TO 225
220 PDIF=PPP(K,1)
225 ACEL(N)=FAREA(KK)*PDIF/WMAS(N)
230 CONTINUE
    DO 300 N=1,NDET
    GO TO(250,275),NSIG
250 DDISP(N)=0.0
    GO TO 300
275 KK=NDET(N)
    DDISP(N)=U(KK)
300 CONTINUE
    DO +30 N=1,NMEM
C *****
C INITIALIZE PLASTIC ROTATION
C *****
    IMVA(N)=0
    IMVB(N)=0
    TBA(N)=0.0
    TBB(N)=0.0
    IMVAT(N)=0
    IMVBT(N)=0
    ILASTA(N)=0
    ILASTB(N)=0
C *****
C INITIALIZE MAX MOM. AXIAL LOAD, PLASTIC ROTATION
C *****
    DO 15 I=1,2
    CMOMP(N,I)=0.0
    CMOMN(N,I)=0.0
    CLOAD(N,I)=0.0
    TMOMP(N,I)=0.0
    TMOMN(N,I)=0.0
    TIMEL(N,I)=0.0
    CMNP(N,I)=0.0
    CMNV(N,I)=0.0
    CMPP(N,I)=0.0
    CMPV(N,I)=0.0
    CLPM(N,I)=0.0
    CLPV(N,I)=0.0
    CLNM(N,I)=0.0
    CLNV(N,I)=0.0
    PYMAX(N,I)=0.0
    PYMIN(N,I)=0.0
15 CONTINUE
    DO 15 I=1,6
    DO 16 IK=1,6
16 SEL(N,I,IK)=0.0
C *****
C TOTAL ELEMENT FORCES IN LOCAL COORD. ELMF
C *****
    GO TO(399,400),NSIG
399 DO 305 NK=1,6
305 ELMF(N,NK)=0.0
    DO 306 NK=1,4
    RMAX(N,NK)=0.0
```

ZERO

```
RMIN(N,NK)=0.0  
TIM(N,NK)=0.0  
306 TMM(N,NK)=0.0  
TEBA(N)=0.0  
TEBB(N)=0.0  
400 CONTINUE  
DO 523 N=1,NINTL  
RDF(N)=0.0  
523 R(N)=0.0  
DO 527 N=1,NDETL  
527 RD(N)=0.0  
DO 800 N=1,NMEM  
DO 800 I=1,2  
800 IIOLD(N,I)=0  
DO 810 N=1,NNODE  
DO 810 NN=1,3  
UMAX(N,NN)=0.0  
UMIN(N,NN)=0.0  
TMAX(N,NN)=0.0  
TMIN(N,NN)=0.0  
810 CONTINUE  
RETURN  
END
```

```

SUBROUTINE STIF
C FORMULATION OF MEMBER STIFFNESS
  DIMENSION UMAX(30,3),UMIN(30,3),TMAX(30,3),TMIN(30,3)
  DIMENSION R(50),RD(60),RDR(60),IIOLO(30,2)
  DIMENSION SB80(60,60)
  DIMENSION CMNP(30,2),CMNV(30,2),CMPP(30,2),CMPV(30,2),CLPM(30,2),
1CLPV(30,2),CLNM(30,2),CLNV(30,2)
  DIMENSION CCOMP(30,2),CMOMN(30,2),CLOAD(30,2),THOMP(30,2),
1TMOMN(30,2),TIMEL(30,2)
  DIMENSION PY(30),PCRY(30)
  DIMENSION SEL(30,6,6),SSY( 90, 90),NDEL(6) ,VL(30),CN(30),SN(30),
1A(30),AI(30),MPA(30),MPB(30),T(6,6),ELM(6,6),C(6,6),TT(6,6),
2FES(6,6),
3JA(30),JB(30),SAA(60,60),NDEDF(60),NINDF(60),SBA(60,60),
4SBB(60,60), SEQ(60,60),SAB(60,60),MCA(30),MCB(30)
5,INYA(30),IMYB(30),IMYAT(30),IMYBT(30)
  DIMENSION IDF( 90), FAREA( 90),WNAS(60),
1ACEL(60),VELS(60),DISP(60),U( 90),PPP(30,20),TTT(33,20),
2DU( 90),DDISP(60),DLDLTD(60,1),DDLTD(60,1),ELMF(30,6),YMY(30,2),
3TBB(30),TBA(30)
  DIMENSION AK( 8100),MATX1( 90),MATX2( 90)
  DIMENSION MUNIF(30),NSDF( 90),NRX1(30),NRX2(30),NRX3(30)
  DIMENSION ILASTA(30),ILASTB(30)
  DIMENSION TEBB(30),TEBA(30)
  DIMENSION RMAX(30,4),RMIN(30,4),TIM(30,4),TMN(30,4)
  DIMENSION TITLE(20)
  DIMENSION PYMAX(30,2),PYMIN(30,2)
  COMMON NUNIT,PYMAX,PYMIN
  COMMON ICOUN,NCOUN,TITLE
  COMMON RMAX,RMIN,TIM,TMN,TEBA,TEBB
  COMMON ILASTA,ILASTB,NELAS
  COMMON UMAX,UMIN,TMAX,TMIN
  COMMON NSKIP,INDEXM
  COMMON SAB,SAA,SB80
  COMMON R,RO,RDR,IIOLO
  COMMON PY,IPRD,IPRMP
  COMMON NDT,LL,NMEM,YMY, CLOAD,CCOMP,CMOMN,THOMP,THOMN,
1TIMEL,CMNP,CMNV,CMPP,CMPV,CLPM,CLPV,CLNM,CLNV
  COMMON NNODE, JA,JB,VL,CN,SN,A,AI,E,MPA,MPB,NINDF,NDEDF,
1NINTL,NDETL,IDF, FAREA,WNAS,IMYAT,IMYBT,PPP,TTT,NWF,
2DT,ACEL,VELS,DAMP,SEQ,DISP,U, DU,DDISP,SBB,SBA,
3DLTD,DDLTD, SEL, ELMF, INYA,IMYB,TBA,TBB
  COMMON YFACT,PCRY,SSY,NSDF,AK,MATX1,MATX2
  COMMON MUNIF,NFS,NOIR,NRX1,NRX2,NRX3,NLNODE
  NDTOT=3*NNODE
  IF(LL-1) 5,5,27
5 DO 25 I=1,NMEM
  IMYA(I)=0
  IMYB(I)=0
  MCA(I)=MPA(I)
25 MCB(I)=MPB(I)
C .....
C INITIALIZATION
C .....
27 DO 710 J=1,NDTOT
  DO 710 K=1,NDTOT
710 SSY(J,K)=0.0

```

STIF

```

DO 30 I=1,NMEM
  SNA=SN(I)
  CNA=CN(I)
  IF(ILL-1)400,400,410
410 IF(ILASTA(I)-INMAT(I))9,6,9
  6 IF(ILASTB(I)-INMAT(I))9,615,9
  9 DO 10 J=1,6
    DO 10 K=1,6
  10 SEL(I,J,K)=0.0
*****
C   FORMULATE ELEMENT STIFFNESS MATRIX
C   *****
  INYA(I)=INMAT(I)
  INYB(I)=INMAT(I)
  IF(MPA(I)-1)430,435,435
435 MCA(I)=1
  GO TO 440
440 MCA(I)=INYA(I)
440 IF(MPB(I)-1)445,450,450
450 MCB(I)=1
  GO TO 400
445 MCB(I)=INYB(I)
400 SPAN=VL(I)
  AREA=A(I)
  VI3=AI(I)
  EE= AREA*E/SPAN
  EIL=E*VI3/SPAN
  AA=4.0*EIL
  BB=2.0*EIL
  CC=6.0*EIL/SPAN
  DD=2.0*CC/SPAN
  FF=3.0*EIL
  GG=CC/2.0
  HH=DD/4.0
  IF(MCA(I)-1) 35,40,40
35 IF(MCB(I)-1) 45,50,50
45 S1=EE
  S2=CC
  S3=DD
  S4=AA
  S5=CC
  S6=BB
  S7=AA
  GO TO 70
50 IF(MPB(I)-1)51,52,51
52 CP=0.0
  CQ=1.0
  CAL=1.0
  GO TO 54
51 CP=YFACT
  CQ=1.0-YFACT
  CAL=YFACT
54 S1=CAL*EE
  S2=CP*CC+CQ*GG
  S3=CP*DD+CQ*HH
  S4=CP*AA+CQ*FF

```

STIF

```
S5=CP*CC
S6=CP*BB
S7=CP*AA
GO TO 70
40 IF(MCB(I)-1) 55,60,60
55 IF(MPA(I)-1)56,57,56
57 CP=0.0
   CQ=1.0
   CAL=1.0
   GO TO 58
58 CP=YFACT
   CQ=1.0-YFACT
   CAL=YFACT
58 S1=CAL*EE
   S2=CP*CC
   S3=CP*DD+CQ*HH
   S4=CP*AA
   S5=CP*CC+CQ*GG
   S6=CP*BB
   S7=CP*AA+CQ*FF
   GO TO 70
60 IF(MPA(I)-1)61,62,61
62 IF(MPB(I)-1) 65,63,65
63 CP=0.0
   CQ=0.0
   CAL=1.0
   GO TO 64
61 IF(MPB(I)-1)69,68,68
68 CP=0.0
   CQ=YFACT
   CAL=YFACT
   GO TO 54
69 CP=YFACT
   CQ=1.0-YFACT
   CAL=YFACT
   GO TO 64
65 CP=0.0
   CQ=YFACT
   CAL=YFACT
   GO TO 58

64 S1=CAL*EE
   S2=CP*CC
   S3=CP*DD
   S4=CP*AA
   S5=CP*CC
   S6=CP*BB
   S7=CP*AA
70 SEL(I,1,1)=S1
   SEL(I,1,4)=-S1
   SEL(I,2,2)=S3
   SEL(I,2,3)=S2
   SEL(I,2,5)=-S3
   SEL(I,2,6)=S5
   SEL(I,3,2)=S2
   SEL(I,3,3)=S4
```

STIF

```

SEL(I,3,5)=-S2
SEL(I,3,6)=S6
SEL(I,4,1)=-S1
SEL(I,4,4)=S1
SEL(I,5,2)=-S3
SEL(I,5,3)=-S2
SEL(I,5,5)=S3
SEL(I,5,6)=-S5
SEL(I,6,2)=S5
SFL(I,6,3)=S6
SEL(I,6,5)=-S5
SEL(I,6,6)=S7
C *****
C FORMULATE TRANSFORMATION MATRIX TO ORIENTATE ELEMENT STIFFNESS
C WITH RESPECT TO GLOBAL COORDINATE SYSTEM
C *****
615 CALL TRANS(CNA,SNA,T)
DO 80 K=1,6
DO 80 L=1,6
80 TT(K,L)=T(L,K)
DO 90 K=1,6
DO 90 L=1,6
90 ELM(K,L)=SEL(I,K,L)
DO 550 K=1,6
DO 550 L=1,6
C(K,L)=0.0
DO 550 M=1,6
550 C(K,L)=C(K,L)+TT(K,M)*ELM(M,L)
DO 560 K=1,6
DO 560 L=1,6
FES(K,L)=0.0
DO 560 M=1,6
560 FES(K,L)=FES(K,L)+C(K,M)*T(M,L)
NA=JA(I)
NB=JB(I)
NDEL(1) =3*NA-2
NDEL(2) =3*NA-1
NDEL(3) =3*NA
NDEL(4) =3*NB-2
NDEL(5) =3*NB-1
NDEL(6) =3*NB
C *****
C FES ARE ELEMENT STIFFNESS IN GLOBAL SYSTEM
C BUILD SYSTEM STIFFNESS MATRIX
C *****
DO 100 L=1,6
NN=NDEL(L)
DO 100 K=1,6
MM=NDEL(K)
100 SSY(NN,MM)=SSY(NN,MM)+FES(L,K)
30 CONTINUE
C *****
C PARTITION SYSTEM STIFFNESS MATRIX AND FORMULATE INDEPENDENT
C STIFFNESS MATRIX
C *****
IF(LI,555,555,115

```

STIF

```

115 DO 110 K=1,NINTL
    NN=NINOF(K)
    DO 120 L=1,NINTL
        MM=NINOF(L)
120 SAA(K,L)=SSY(NN,MM)
    DO 125 L=1,NDETL
        MM=NJEOF(L)
125 SAB(K,L)=SSY(NN,MM)
110 CONTINUE
    DO 130 K=1,NDETL
        NN=NJEOF(K)
    DO 135 L=1,NINTL
        MM=NINOF(L)
135 SBA(K,L)=SSY(NN,MM)
    DO 140 L=1,NDETL
        MM=NJEOF(L)
        SBB(K,L)=SSY(NN,MM)
140 SBB0(K,L)=SBB(K,L)
130 CONTINUE
    CONS=SBB(1,1)
    DO 145 I=1, NDETL
        K=NDETL*(I-1)+1
        L=NDETL*I
        M=0
        DO 145 J=K,L
            M=M+1
145 AK(J)=SBB(I,M)/CONS
C *****
C CALL SUBROUTINE MINV TO INVERT MATRIX SBB
C *****
    CALL MINV(AK,NDETL,DETR,MATX1,MATX2)
    DO 146 I=1,NDETL
        K=NDETL*(I-1)+1
        L=I*NDETL
        M=0
        DO 146 J=K,L
            M=M+1
146 SBB(I,M)=AK(J)/CONS
C *****
C HERE SBB IS INVERSE OF ORIGINAL SBB
C *****
    DO 500 K=1,NINTL
        DO 500 L=1,NDETL
            SEQ(K,L)=0.0
            DO 500 LK=1,NDETL
200 SEQ(K,L) =SEQ(K,L) +SAB(K,LK)*SBB(LK,L)
            DO 505 K=1,NINTL
                DO 505 L=1,NINTL
                    SSY(K,L)=0.0
                    DO 505 LK=1,NDETL
305 SSY(K,L) =SSY(K,L) + SEQ(K,LK)*SBA(LK,L)
            DO 510 K=1,NINTL
                DO 510 L=1,NINTL
510 SEQ(K,L)=SAA(K,L)-SSY(K,L)
C *****
C HERE SEQ=(SAA)-(SAB)*(INV SBB)*(SBA)

```

STIF

C

555 RETURN
END

```

SUBROUTINE PLOAD(SSY, NSDF, NFS, SN, CN, VL, NDIR, WUNIF, NMEM, JA, JB,
1 ELMF, NNODE, DEL, U, SEL, AK, MATX1, MATX2, MPA, MPB, NLNODE, TITLE, NCDUN,
1 TEBB, TEBA, ICOUN, NUNIT)
C *****
C STATIC ANALYSIS ROUTINE
C *****
DIMENSION TITLE(20)
DIMENSION SSY( 90, 90), NSDF( 90), SN(30), CN(30), VL(30),
1 JA(30), JB(30), WUNIF(30), T(6,6), DEL( 90), P(6), ELMF(30,6), TT(6,6),
2 SEL(30,6,6), U( 90)
DIMENSION AK( 8100), MATX1( 90), MATX2( 90)
DIMENSION MPA(30), MPB(30), TEBB(60), TEBA(30)
DIMENSION FUNIT(2,2), XLUNIT(2), FMUNIT(2,2)
DATA XLUNIT/4H(MM), 4H(IN)/
DATA FMUNIT/5H(KNS), 5H(LBS), 8H (KN-M) , 8H (LB-IN)/
DATA FUNIT/5H(KG) , 5H(LBS), 8H (KN-M) , 8H (LB-IN)/
ADIR=NDIR
AD=ABS(ADIR)
LDIR=AD
NDIF=3*NNODE-NFS
C *****
C COMPUTE FIXED ENDED REACTIONS FOR UNIFORM LOADS ON ELEMENTS
C *****
DO 500 I=1, NMEM
IF(WJNIF(I))6,4,6
4 DO 5 K=1,6
5 ELMF(I,K)=0.0
GO TO 32
6 GO TO(10,20), LDIR
10 PX= CN(I)*WUNIF(I)
RXW=-WUNIF(I)*SN(I)
GO TO 30
20 PX= SN(I)*WUNIF(I)
RXW=WUNIF(I)*CN(I)
30 PY=SQRT((WUNIF(I)**2)-(PX**2))
XX=-SN(I)
YY=CN(I)
RXY=CN(I)*YY-SN(I)*XX
RXYDRXW=RXY/RXW
ELMF(I,1)=-.5*PX*VL(I)
ELMF(I,4)=-.5*PX*VL(I)
IF(RXYDRXW)15,13,15
13 DO 14 K=2,3
ELMF(I,K)=0.0
14 ELMF(I,K+3)=0.0
GO TO 32
15 AV=ABS(RXYDRXW)
DIR=RXYDRXW/AV
KK=MPA(I) +MPB(I)
IF(KK)31,31,26
26 GO TO(21,31), KK
21 IF(MPA(I))25,25,24
24 ELMF(I,2)=-3.0*PY*VL(I)*DIR/8.0
ELMF(I,3)=0.0
ELMF(I,5)=-5.0*PY*VL(I)*DIR/8.0
ELMF(I,6)=1.0*DIR*PY*(VL(I)**2.0)/8.0
GO TO 32

```

PLOAD

```

25 ELMF(I,2)=-5.0*PY*VL(I)*DIR/8.0
   ELMF(I,3)=-1.0*DIR*PY*(VL(I)**2.0)/8.0
   ELMF(I,5)=-3.0*PY*VL(I)*DIR/8.0
   ELMF(I,6)=0.0
   GO TO 32
31 ELMF(I,2)=-.5*PY*VL(I)*DIR
   ELMF(I,5)=-.5*PY*VL(I)*DIR
   IF(KK)33,33,34
33 ELMF(I,3)=-DIR*PY*(VL(I)**2)/12.0
   ELMF(I,6)= DIR*PY*(VL(I)**2)/12.0
   GO TO 32
34 ELMF(I,3)=0.0
   ELMF(I,6)=0.0
32 CNA=CN(I)
   SNA=SN(I)
C *****
C CALL SUBROUTINE TRANS TO TRANSFORM FIXED END ELEMENT REACTIONS
C TO GLOBAL SYSTEM
C *****
   CALL TRANS(CNA,SNA,T)
   DO 29 J=1,6
   DO 28 K=1,6
28 TT(J,K)=T(K,J)
   DO 29 J=1,6
   P(J)=0.0
   DO 29 K=1,6
29 P(J)=P(J)+TT(J,K)*ELMF(I,K)
   DO 130 K=1,3
   NDOF=3*JA(I)-3+K
   DO 50 J=1,NFS
   IF(NDOF-NSFOF(J))50,35,50
35 U(J)=J(J)-P(K)
   GO TO 100
50 CONTINUE
100 CONTINUE
   DO 200 K=4,6
   NDOF=3*JB(I)-6+K
   DO 150 J=1,NFS
   IF(NDOF-NSFOF(J))150,135,150
135 U(J)=U(J)-P(K)
   GO TO 200
150 CONTINUE
200 CONTINUE
500 CONTINUE
   IF(NLNODE)760,760,710
710 WRITE(5,9999)FUNIT(NUNIT,1),(FUNIT(NUNIT,J),J=1,2)
   DO 750 I=1,NLNODE
C *****
C READ NODAL POINTS STATIC LOADS CARDS - CARD TYPE 15
C *****
   READ(2,10000)JT,(DEL(J),J=1,3)
   WRITE(5,10001)JT,(DEL(J),J=1,3)
   IF(NUNIT.EQ.2)GO TO 714
   DO 713 J=1,3
713 DEL(J)=0.0098*DEL(J)
C *****

```

```

                                PLOAD
C   COMPLETE ASSEMBLY OF STATIC FORCE MATRIX
C   *****
714 DO 725 J=1,3
    NDOF=3*(JT-1)+J
    DO 720 K=1,NFS
    IF(NDOF-NSFDF(K))720,715,720
715 U(K)=U(K)+DEL(J)
    GO TO 725
720 CONTINUE
725 CONTINUE
750 CONTINUE
9999 FORMAT(1H ,23X,24HSTATIC LOADS-JOINT LOADS,/,3X,4HNODE,17X,2HFX,18
1X,24FY,18X,24HZ,/,23X,A5,15X,A5,13X,A8/)
10000 FORMAT(10,3F10.0)
10001 FORMAT(+X,I2,14X,E10.3,10X,E10.3,10X,E10.3)
760 DO 1001 I=1,NFS
    KK=NSFDF(I)
    K=NFS*(I-1)+1
    L=I*NFS
    M=0
    DO 1001 J=K,L
    M=M+1
    LL=NSFDF(M)
1001 AK(J)=SSY(KK,LL)
    CONS=AK(1)
    MM=NFS**2
    DO 1002 I=1,MM
1002 AK(I)=AK(I)/CONS
C   *****
C   CALL SUBROUTINE MINV TO INVERT SYSTEM STIFFNESS MATRIX
C   *****
    CALL MINV(AK,NFS,DETR,MATX1,MATX2)
    DO 1003 I=1,NFS
    K=NFS*(I-1)+1
    L=I*NFS
    M=0
    DO 1003 J=K,L
    M=M+1
1003 SSY(I,M)=AK(J)/CONS
C   *****
C   COMPUTE NODAL DISPLACEMENTS
C   *****
    DO 2000 I=1,NFS
    DEL(I)=0.0
    DO 2000 J=1,NFS
2000 DEL(I)=DEL(I)+SSY(I,J)*U(J)
    NDDOF=3*NNODE
    DO 2100 I=1,NDDOF
2100 U(I)=0.0
    DO 2200 I=1,NFS
    K=NSFDF(I)
2200 U(K)=DEL(I)
C   *****
C   COMPUTE ELEMENT END LOADS AND ROTATIONS
C   *****
    DO 3000 I=1,NMEM

```

PLOAD

```

CNA=CN(I)
SNA=SN(I)
C *****
C CALL SUBROUTINE TRANS TO TRANSFORM ELEMENT LOADS TO LOCAL SYSTEM
C *****
CALL TRANS(CNA,SNA,T)
DO 2300 K=1,3
2300 NSFJF(K)=3*JA(I)-3 +K
DO 2400 K=4,6
2400 NSFJF(K)=3*JB(I)-6+K
DO 2500 K=1,6
N=NSFJF(K)
2500 P(K)=U(N)
DO 2600 J=1,6
AK(K)=0.0
GO 2600 J=1,6
2600 AK(K)=AK(K) + T(K,J)*P(J)
DO 2700 K=1,6
DO 2700 J=1,6
2700 ELMF(I,K)=ELMF(I,K) +SEL(I,K,J)*AK(J)
IF(AK(3))2701,2702,2701
2701 TE3A(I)=( AK(3)-(( AK(5)- AK(2))/VL(I)))*57.29577951
2702 IF(AK(6))2703,3000,2703
2703 TE3B(I)=( AK(6)-(( AK(5) - AK(2))/VL(I)))*57.29577951
3000 CONTINUE
C *****
C PRINT NODAL DISPLACEMENTS AND ELEMENT END LOADS
C *****
WRITE(5,4999)TITLE
4999 FORMAT(1H1,20X,20A4)
NCOJN=1
WRITE(5,5000)
5000 FORMAT( /,10X,+9HDEAD AND LIVE LOAD DISPLACEMENTS AND MEMBER LOAD
1S//)
WRITE(5,615)XLUNIT(NUNIT),XLUNIT(NUNIT)
DO 4000 I=1,NNODE
K=3*I-2
L=3*I
ROT=57.29577951*U(L)
L=L-1
U01=J(K)
U02=U(L)
IF(NJNIT.EQ.2)GO TO 4000
U01=1000.J*U01
U02=1000.0*U02
4000 WRITE(5,5500)I,U01,U02,ROT
5500 FORMAT(8X,I2,3(3X,E12.4))
615 FORMAT(1H ,5X,4HNODE,8X,7HX-DISPL,8X,7HY-DISPL,7X,4HROTATION,
1/,19X,A+,11X,A4,10X,5H(DEG))
WRITE(5,616)FMUNIT(NUNIT,1),(FMUNIT(NUNIT,J),J=1,2),
1 FMUNIT(NUNIT,1),(FMUNIT(NUNIT,J),J=1,2)
616 FORMAT(1HC ,5X,4HELEM,10X,2HPA,13X,2HVA,13X,2HMA,13X,2HPB,13X,2HVR,
113X,2HMB,/,18X,A5,10X,A5,9X,A8,8X,A5,10X,A5,9X,A8)
DO 630 N=1,NMEM
630 WRITE(5,640)N,(ELMF(N,M),M=1,6)
640 FORMAT(8X,I2,6(3X,E12.4))

```

PLOAD

```
8000 FORMAT(1X,10E12.4)
9000 FORMAT(10I8)
      IF(NCOUN-ICOUN)9001,8999,8999
8999 WRITE(5,4999),TITLE
      NCOUN=0
9001 WRITE(5,9050)
9050 FORMAT(/1X,16HRESPONSE HISTORY/)
      RETURN
      END
```

```

SUBROUTINE JYDIS
DIMENSION UMAX(30,3),UMIN(30,3),TMAX(30,3),TMIN(30,3)
DIMENSION R(60),RD(60),RDR(60),IIOLD(30,2)
DIMENSION SAB(60,60),SAA(60,60),SBB(60,60)
DIMENSION CMNP(30,3),CMNV(30,2),CMPP(30,2),CMPV(30,2),CLPM(30,2),
CLPV(30,2),CLNM(30,2),CLNV(30,2)
DIMENSION CMOM(30,2),CMOMN(30,2),CLOAD(30,2),TMOMP(30,2),
TMOMN(30,2),TIME(30,2)
DIMENSION PY(30),PCRY(30)
DIMENSION AA(60),BB(60),PEF(60,1),NINDF(60),IDF( 90),
IFAREA( 90),WMAS(60), SEQ(60,60), ACEL(60),
2VELS(60),C( 90, 90),PPP(30,20),TTT(30,20),
3DLTD(60,1),SBB(60,60),SBA(60,60), DDISP(60),
+ DDLTD(60,1),U( 90),DU( 90),DISP(60),NOEDF(60)
DIMENSION JA(30),JB(30),VL(30),CN(30),SN(30),A(30),AI(30),
1MPA(30),MPB(30),IMYAT(30),IMYBT(30),SEL(30,6,6),
2ELMF(30,6),YMY(30,2),IMYA(30),IMYB(30),TBA(30),TBB(30),
3NSDF( 90)
DIMENSION AK( 8100),MATX1( 90),MATX2( 90)
DIMENSION ILASTA(30),ILASTB(30)
DIMENSION TEBB(30),TEBA(30)
DIMENSION TITLE(20),PP(30),NSTA(30)
DIMENSION RMAX(30,4),RMIN(30,4),TIM(30,4),TMM(30,4),UU(3)
DIMENSION DUNIT(2)
DIMENSION PYMAX(30,2),PYMIN(30,2)
COMMON DUNIT,PYMAX,PYMIN
COMMON ICOUN,NCOUN,TITLE
COMMON RMAX,RMIN,TIM,TMM,TEBA,TEBB
COMMON ILASTA,ILASTB,NELAS
COMMON UMAX,UMIN,TMAX,TMIN
COMMON NSKIP,INDEXW
COMMON SAB,SAA,SBB
COMMON R,RD,RDR,IIOLD
COMMON PY,IPR3,IPRNP
COMMON NOT,LL,NMEM,YMY, CLOAD,CMOMP,CMOMN,TMOMP,TMOMN,
TIME,CMNP,CMNV,CMPP,CMPV,CLPM,CLPV,CLNM,CLNV
COMMON NNODE, JA,JB,VL,CN,SN,A,AI,E,MPA,MPB,NINDF,NOEDF,
1NINTL,NOETL,IDF, IFAREA,WMAS,IMYAT,IMYBT,PPP,TTT,NWF,
2DT,ACEL,VELS,DAMP,SEQ,DISP,U, DU,DDISP,SBB,SBA,
3DLTD,DOLTD, SEL, ELMF, IMYA,IMYB,TBA,TBB
COMMON YFACT,PCRY,C ,NSDF,AK,MATX1,MATX2
DATA DUNIT/4H(MY),4H(IN)/
*****
C
C
C
INITIALIZATION
*****
IF(LL-1)90,90,99
90 DO 95 N=1,NWF
95 NSTA(N)=1
99 TIME=LL*DT
C2=DT/2.0
C3=6./DT
C4=3./DT
C5=C3/DT
C6=4.0/DT
DO 200 N=1,NINTL
AA(N)=DT*VELS(N) +(DT**2.C)*ACEL(N)/3.0
BB(N)= VELS(N)+C2*ACEL(N)

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200 PEF(N,1)=0.0
    DO 205 N=1,NINTL
    DO 205 J=1,NINTL
    PEF(N,1)=PEF(N,1)-SEQ(N,J)*AA(J)
205 CONTINUE
C *****
C CALCULATION OF EFFECTIVE INCREMENTAL LOAD
C *****
    DO 100 N=1,NWF
    L=NSTA(N)
110 L=L+1
    DDT=TTT(N,L)-TTT(N,L-1)
    DDP=PPP(N,L)-PPP(N,L-1)
    IF(DDT)115,110,120
120 SLOPE=DDP/DDT
    IF(TTT(N,L)-TIME)110,125,125
125 PP(N)=PPP(N,L-1) + (TIME-TTT(N,L-1))*SLOPE
    NSTA(N)=L-1
100 CONTINUE
    DO 250 N=1,NINTL
    KK=NINDF(N)
    K=IDF(KK)
    IF(K) 210,210,220
210 PDIF=C.0
    GO TO 225
220 PDIF=PP(K)
225 PEF(N,1)=PEF(N,1) +FAREA(KK)*PDIF -R(N)-DAMP*WMAS(N)+BB(N)+ROR(N)
250 CONTINUE
C *****
C CALCULATION OF EFFECTIVE INCREMENTAL STIFFNESS
C *****
    DO 350 N=1,NINTL
    DO 350 NN=1,NINTL
    C(N,NN)=0.0
    IF(N=NN)360,370,350
370 WT=WMAS(NN)
    GO TO 380
350 WT=0.0
380 C(N,NN)=(DT**2.0)*SEQ(N,NN)/5.0 +WT*(1.0 +(DAMP*DT/2.0))
350 CONTINUE
C *****
C CALCULATION OF INCREMENTAL DISPLACEMENT
C *****
    CONS= C(1,1)
    DO 376 I=1,NINTL
    K=NINTL*(I-1) +1
    L=I*NINTL
    M=0
    DO 376 J=K,L
    M=M+1
376 AK(J)= C(I,M)/CONS
C *****
C INVERT EFFECTIVE INCREMENTAL STIFFNESS MATRIX
C *****
CALL MINV(AK,NINTL,DETR,MATX1,MATX2)
DO 377 I=1,NINTL

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K=NINTL*(I-1) +1
L=I*NINTL
M=J
DO 377 J=K,L
M=M+1
377 C(I,M)=AK(J)/CONS
DO 550 I=1,NINTL
ACEL(I)=0.0
DO 550 J=1,NINTL
550 ACEL(I)=ACEL(I) + C(I,J)*PEF(J,1)
C
C
C
C
CALCULATION OF ACCELERATION,VELOCITY AND DISPLACEMENT FOR IND.
DYNAMIC DOF.
*****
DO 550 I=1,NDETL
DO 560 J=1,NINTL
C(I,J)=0.0
DO 560 N=1,NDETL
560 C(I,J)=C(I,J)+SBB(I,N)*SBA(N,J)
DO 560 N=1,NINTL
VELS(N)=BB(N)+DT*ACEL(N)/2.0
DLTD(N,1)=AA(N) + (DT**2.0)*ACEL(N)/6.0
400 DISP(N)=DISP(N) + DLTD(N,1)
C
C
C
C
CALCULATION OF ACC VEL DISP FOR DEPENDENT DYNAMIC DOF
*****
DO 570 I=1,NDETL
DDLTD(I,1)=0.0
DO 569 N=1,NDETL
569 DDLTD(I,1)=DDLTD(I,1)-SBB(I,N)*RD(N)
DO 570 J=1,NINTL
570 DDLTD(I,1)=DDLTD(I,1)-C(I,J)*DLTD(J,1)
DO 575 N=1,NDETL
575 DDISP(N)=DDISP(N) + DDLTD(N,1)
C
C
C
C
FORMULATION OF Q,RD,RDR
*****
DO 750 K=1,NINTL
DO 752 L=1,NINTL
752 R(K)=R(K)+SAA(K,L)*DLTD(L,1)
DO 753 L=1,NDETL
753 R(K)=R(K)+SAB(K,L)*DDLTD(L,1)
750 CONTINUE
DO 758 K=1,NDETL
DO 765 L=1,NINTL
765 RD(K)=RD(K)+SBA(K,L)*DLTD(L,1)
DO 768 L=1,NDETL
768 RD(K)=RD(K)+SBB(K,L)*DDLTD(L,1)
758 CONTINUE
DO 785 K=1,NINTL
RDR(K)=0.0
DO 785 L=1,NDETL
C(K,L)=0.0
DO 785 J=1,NDETL
785 C(K,L)=C(K,L)+SAB(K,J)*SBB(J,L)
DO 780 KK=1,NDETL

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RDR(K)=RDR(K)+C(K,KK)*RD(KK)
780 CONTINUE
C *****
C DAGEL,DVELS,DDTSP,DOLTA,DDLTV,DDLTD REFER TO DEPENDENT DYNAMIC DOF
C AGEI,VELS,DISP,DLTA,DLTV,DLTD REFER TO INDEPENDENT DYNAMIC DOF
C *****
C FORMULATION OF DISP.MATRIX U(N),DU(N).WHERE N=TOTAL STATIC DOF
C *****
DO 460 N=1,NINTL
MI=NINDF(N)
U(MI)=DISP(N)
460 DU(MI)=DLTD(N,1)
DO 470 N=1,NOETL
MI=NEDF(N)
U(MI)=DDISP(N)
470 DU(MI)=DDLTD(N,1)
C *****
C RECORD MAXIMA AND MINIMA OF NODAL DISPLACEMENTS
C *****
DO 800 N=1,NNODE
N1=3*N-2
N2=3*N-1
N3=3*N
UU(1)=U(N1)
UU(2)=U(N2)
UU(3)=U(N3)
DO 850 NN=1,3
UMA=UMAX(N,NN)
UMI=UMIN(N,NN)
UUU=UU(NN)
IF(UUU)810,850,820
810 IF(UUU-UMI)830,850,850
830 UMIN(N,NN)=UUU
TIMEIN(N,NN)=TIME
GO TO 850
820 IF(UUU-UMA)850,850,825
825 UMAX(N,NN)=UUU
TIMEAX(N,NN)=TIME
850 CONTINUE
800 CONTINUE
IF(IPRD-1)600,610,610
610 IF(INDEXN-1)600,612,612
612 IF(NCOUN-ICOUN)614,613,613
613 WRITE(5,900)TITLE
900 FORMAT(1H1,20X,20A4)
NCOUN=0
614 WRITE(5,620) TIME
NCOUN=NCOUN + 1
620 ORMAT(1H3,5HTIME=,E12.4,4H SEC)
WRITE(5,615)DUNIT(NUNIT),DUNIT(NUNIT)
615 FORMAT(1H ,5X,4HNODE,8X,7HX-DISPL,8X,7HY-DISPL,7X,6HROTATION,
1 /,19X,A4,11X,A4,10X,5H(DEG)),
C *****
C PRINT NODAL DISPLACEMENTS
C *****
DO 630 N=1,NNODE

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N1=3*N-2
N2=3*N-1
N3=3*N
ROT=57.29577951*U(N3)
UO1=U(N1)
UO2=U(N2)
IF(NJUNIT.EQ.2)GO TO 630
UO1=1000.0*UO1
UO2=1000.0*UO2
630 WRITE(5,6+0)N,UO1,UO2,ROT
640 FORMAT(8X,I2,3(3X,E12.4))
600 RETURN
END
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SUBROUTINE ELEFD
C   CALCULATION OF ELEMENT FORCES AND DEFORMATIONS
DIMENSION UMAX(30,3),UMIN(30,3),TMAX(30,3),TMIN(30,3)
DIMENSION R(6),RD(60),RDR(60),IIOLD(3,2)
DIMENSION SAB(60,60),SAA(60,60),SBBO(60,60)
DIMENSION CMNP(30,2),CMNV(30,2),CMPP(30,2),CMPV(30,2),CLPM(30,2),
1CLPV(30,2),CLNM(30,2),CLNV(30,2)
DIMENSION CMOMP(30,2),CMOMN(30,2),CLOAD(30,2),TMOMP(30,2),
1TMOMN(30,2),TIMEL(30,2)
DIMENSION PY(30),PCRY(30)
DIMENSION JA(30),JB(30),NOEL(6),U( 90),DU( 90),ELMU(6,1),ELMDU(6,1)
1),CN(30),SN(30),T(6,6),          SEL(30,6,6),
2ELMT(6,5),TELMD(6,1),C(6,1),ELMDF(6,1),ELMF(30,5),YMY(30,2),
3IMYAT(30),IMYBT(30),VL(30),          MPA(30),MPB(30),
4IMYA(30),IMYB(30),TBA(30),TBB(30)
DIMENSION A(30),AI(30),NINDF(60),NOEDF(60),IDF( 90),
1FAREA( 90),WMAS(60),PPP(30,20),TTT(30,20)
DIMENSION ACEL(60),VELS(60),SEQ(50,60),DISP(60),
1          DOIIP(60),SBB(60,60),SRA(60,60),DLTD(60,1),DDLTD(60,1)
DIMENSION FAT( 90, 90),NSFDF( 90)
DIMENSION ILASTA(30),ILASTB(30)
DIMENSION TEBB(30),TEBA(30),SUMP(30),SUMA(30),SUMB(30)
DIMENSION TITLE(20)
DIMENSION RMAX(30,4),RMIN(30,4),TIM(30,4),TMM(30,4)
DIMENSION PYMAX(30,2),PYMIN(30,2)
DIMENSION FUNIT(2,2)
COMMON NUNIT,PYMAX,PYMIN
COMMON ICOUN,NCOUN,TITLE
COMMON RMAX,RMIN,TIM,TMM,TEBA,TEBB
COMMON ILASTA,ILASTB,ELAS
COMMON UMAX,UMIN,TMAX,TMIN
COMMON NSKIP,INDEXM
COMMON SAB,SAA,SBBO
COMMON R,RD,RDR,IIOLD
COMMON PY,IPRD,IPRMP
COMMON NDT,LL,NMEM,YMY,          CLOAD,CMOMP,CMOMN,TMOMP,TMOMN,
1TIMEL,CMNP,CMNV,CMPP,CMPV,CLPM,CLPV,CLNM,CLNV
COMMON NNODE,          JA,JB,VL,CN,SN,A,AI,E,MPA,MPB,NINDF,NOEDF,
1NINTL,NOETL,IDF,          FAREA,WMAS,IMYAT,IMYBT,PPP,TTT,NWF,
2DT,ACEL,VELS,DAMP,SEQ,DISP,U,          DU,DOIIP,SBB,SBA,
3DLTD,DDLTD,          SEL,          ELMF,          IMYA,IMYB,TBA,TBB
COMMON YFACT,PCRY,FAT,NSFDF
DATA FUNIT/5H(KNS),5H(LBS),8H (KN-M) ,8H (LB-IN)/
TIME=LL*DT
C *****
C   COMPUTE ELEMENT FORCES IN LOCAL COORDINATE
C *****
DO 900 M=1,NMEM
NA=JA(M)
NB=JB(M)
NOEL(1)=3*NA-2
NOEL(2)=3*NA-1
NOEL(3)=3*NA
NOEL(4)=3*NB-2
NOEL(5)=3*NB-1
NOEL(6)=3*NB
DO 10 K=1,6

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N=NDEL(K)
ELMU(K,1)=U(N)
10 ELMDU(K,1)=DU(N)
CNA=CN(1)
SNA=SN(M)
*****
C CALL SUBROUTINE TRANS TO COMPUTE ELEMENT END LOADS IN ELEMENT
C COORDINATE SYSTEM
C *****
CALL TRANS(CNA,SNA,T)
DO 20 K=1,6
DO 20 KK=1,6
20 ELMT(K,KK)=SEL(M,K,KK)
DO 550 I=1,6
TELMD(I,1)=0.0
DO 550 J=1,6
550 TELMD(I,1)=TELMD(I,1)+T(I,J)*ELMU(J,1)
DO 560 I=1,6
C(I,1)=0.0
DO 560 J=1,6
560 C(I,1)=C(I,1)+T(I,J)*ELMDU(J,1)
DO 570 I=1,6
ELMDF(I,1)=0.0
DO 570 J=1,6
570 ELMDF(I,1)=ELMDF(I,1)+ELMT(I,J)*C(J,1)
*****
C CALL SUBROUTINE PDEL TO COMPUTE EQUIVALENT SHEARS FOR THE P-DELTA
C AND BEAM COLUMN EFFECTS
C *****
CALL PDEL(TELMD,C,ELMF,ELMDF,VL,R,ROR,NINTL,NDETL,NDOF,NDOEF,
1 T,M,NOEL,FAT)
TEMF1=ELMF(M,1)
TEMF3=ELMF(M,3)
TEMF4=-ELMF(M,4)
TEMF6=ELMF(M,6)
DO 30 K=1,6
30 ELMF(M,K)=ELMF(M,K)+ELMDF(K,1)
IF(NELAS)31,31,350
31 YM=YHY(M,1)
YP=PY(M)
EM=ELMF(M,3)
EP=ELMF(M,1)
IOLD=IOLD(M,1)
PCR=PCRY(M)
*****
C CALL SUBROUTINE YIELD TO DETERMINE WHETHER ELEMENT IS IN EITHER
C THE ELASTIC OR PLASTIC CONDITIONS
C *****
CALL YIELD(EM,EP,YM,YP,IX,TH,TP,IOLD,TEMF1,TEMF3,PCR,SUM1,SUMM)
SUMP(M)=SUM1
SUMA(M)=SUMM
IOLD(M,1)=IOLD
ILASTA(M)=IMYAT(M)
IMYAT(M)=IX
EM=ELMF(M,6)
IOLD=IOLD(M,2)

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YM=YM*(M,2)
EP=-ELMF(M,4)
CALL YIELD(EM,EP,YM,YP,IX,TM,TP,IOLD,TEMF4,TEMF6,PCR,SUM1,SUMH)
SUMB(M)=SUMH
IOLD(M,2)=IOLD
ILASTB(M)=IMYBT(M)
IMYBT(M)=IX
*****
C COMPUTE ELASTIC AND/OR PLASTIC ELEMENT END ROTATIONS
C *****
SKA=4.0*E*AI(M)/VL(M)
SKB=SKA/2.0
DALPH=(C(5,1)-C(2,1))/VL(M)
TEM1=C(6,1)-DALPH
TEM2=C(3,1)-DALPH
TEM1=TEM1*57.29577951
TEM2=TEM2*57.29577951
IF(IMYA(M)-1)80,65,65
60 IF(IMYB(M)-1)70,75,75
70 DBA =0.0
   DBB =0.0
   IF(TBA(M).NE.0.0)TBA(M)=0.0
   IF(TBB(M).NE.0.0)TBB(M)=0.0
45 TEBA(M)=TEBA(M)+TEM2
55 TEBB(M)=TEBB(M)+TEM1
   GO TO 100
75 DBA =0.0
   IF(TBA(M).NE.0.0)TBA(M)=0.0
   IF(MPA(M)-1)80,85,85
85 DBB =TEM1
   GO TO 100
80 IF(NSFDF(M)-1)81,85,81
81 IF(TEBB(M))84,83,84
83 DBB=TEM2*SKB/SKA
   GO TO 86
84 DBB =TEM1+TEM2*SKB/SKA
86 IF(C(3,1))82,100,82
82 TEBA(M)=TEBA(M)+TEM2
   GO TO 100
65 IF(IMYB(M)-1)90,95,95
95 DBB =TEM1
   DBA =TEM2
   GO TO 100
90 IF(MPB(M)-1)97,98,98
98 DBB =0.0
   DBA =TEM2
   GO TO 100
97 IF(NSFDF(M)-2)99,98,99
99 DBB =0.0
   IF(TBB(M).NE.0.0)TBB(M)=0.0
   IF(TEBA(M))103,102,103
102 DBA=TEM1*SKB/SKA
   GO TO 104
103 DBA =TEM2+TEM1*SKB/SKA
104 IF(C(6,1))101,100,101
101 TEBB(M)=TEBB(M)+TEM1

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100 TBA(M)=TBA(M)+DBA
    TBB(M)=TBB(M)+DBB
C *****
C RECORD MAXIMA AND MINIMA OF ELEMENT END ROTATIONS
C *****
399 IF(RMAX(M,1)-TEBA(M))210,210,210
200 IF(RMAX(M,2).GT.0.0)GO TO 210
    RMAX(M,1)=TEBA(M)
    TIM(M,1)=TIME
210 IF(RMAX(M,2)-TBA(M))220,230,230
220 RMAX(M,2)=TBA(M)
    TIM(M,2)=TIME
    RMAX(M,1)=TEBA(M)
    TIM(M,1)=TIME
    PYMAX(M,1)= ELMF(M,1)/PY(M)
230 IF(RMAX(M,3)-TEBB(M))240,250,250
240 IF(RMAX(M,4).GT.0.0)GO TO 250
    RMAX(M,3)=TEBB(M)
    TIM(M,3)=TIME
250 IF(RMAX(M,4)-TBB(M))260,270,270
260 RMAX(M,4)=TBB(M)
    TIM(M,4)=TIME
    RMAX(M,3)=TEBB(M)
    TIM(M,3)=TIME
    PYMAX(M,2)= -ELMF(M,4)/PY(M)
270 IF(RMIN(M,1)-TEBA(M))290,290,290
280 IF(RMIN(M,2).LT.0.0)GO TO 290
    RMIN(M,1)=TEBA(M)
    TMM(M,1)=TIME
290 IF(RMIN(M,2)-TBA(M))310,310,300
300 RMIN(M,2)=TBA(M)
    TMM(M,2)=TIME
    RMIN(M,1)=TEBA(M)
    TMM(M,1)=TIME
    PYMIN(M,1)= ELMF(M,1)/PY(M)
310 IF(RMIN(M,3)-TEBB(M))330,330,321
321 IF(RMIN(M,4).LT.0.0)GO TO 330
    RMIN(M,3)=TEBB(M)
    TMM(M,3)=TIME
330 IF(RMIN(M,4)-TBB(M))350,350,341
341 RMIN(M,4)=TBB(M)
    TMM(M,4)=TIME
    RMIN(M,3)=TEBB(M)
    TMM(M,3)=TIME
    PYMIN(M,2)= -ELMF(M,4)/PY(M)
C *****
C RECORD MAXIMA AND MINIMA OF ELEMENT END LOADS
C *****
310 IF(ELMF(M,3))400,405,405
400 IF(ELMF(M,3)-CMOMN(M,1))410,415,415
410 CMOMN(M,1)=ELMF(M,3)
    TMOYN(M,1)=TIME
    CMNP(M,1)=ELMF(M,1)
    CMNV(M,1)=ELMF(M,2)
    GO TO 415
405 IF(ELMF(M,3)-CMOMP(M,1))415,415,425

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425 CMOMP(M,1)=ELMF(M,3)
    CMPP(M,1)=ELMF(M,1)
    CMPV(M,1)=ELMF(M,2)
    TMOMP(M,1)=TIME
415 IF(ELMF(M,6))430,435,435
430 IF(ELMF(M,6)-CMOMN(M,2))450,455,455
450 CMOMN(M,2)=ELMF(M,6)
    CMNP(M,2)=ELMF(M,1)
    CMNV(M,2)=ELMF(M,5)
    TMOMN(M,2)=TIME
    GO TO 455
435 IF(ELMF(M,6)-CMOMP(M,2))455,455,465
465 CMOMP(M,2)=ELMF(M,6)
    CMPP(M,2)=ELMF(M,1)
    CMPV(M,2)=ELMF(M,5)
    TMOMP(M,2)=TIME
455 IF(ELMF(M,1))470,475,475
470 IF(ELMF(M,1)-CLOAD(M,2))480,900,900
480 CLOAD(M,2)=ELMF(M,1)
    CLNM(M,1)=ELMF(M,3)
    CLNV(M,1)=ELMF(M,2)
    CLNM(M,2)=ELMF(M,6)
    CLNV(M,2)=ELMF(M,5)
    TIMEL(M,2)=TIME
    GO TO 900
475 IF(ELMF(M,1)-CLOAD(M,1))900,490,490
490 CLOAD(M,1)=ELMF(M,1)
    CLPM(M,1)=ELMF(M,3)
    CLPM(M,2)=ELMF(M,6)
    CLPV(M,1)=ELMF(M,2)
    CLPV(M,2)=ELMF(M,5)
    TIMEL(M,1)=TIME
900 CONTINUE
    IF(IPRMP-1)600,610,610
610 IF(INDEXW-1)600,603,603
603 IF(IPRD-1)604,605,604
604 IF(NCOUN-ICOUN)607,606,606
606 WRITE(5,1001)TITLE
1001 FORMAT(1H1,20X,20A4)
    NCOUN=0
607 NCOUN=NCOUN + 1
605 WRITE(5,615)FUNIT(NUNIT,1),(FUNIT(NUNIT,K),K=1,2),(FUNIT(NUNIT,K),
1 K=1,2)
615 FORMAT( /5X,4HELEN,9X,2HPA,8X,5HPA/PG, 13X,2HVA,12X,2HMA,10X,
16HMA/MMA,10X,2HVB,12X,2HMB,10X,6HMB/MKB,/,17X,A5,23X,A5,8X,A8,21X,
2 A5,8X,A8)
C *****
C PRINT ELEMENT END LOADS AND ROTATIONS
C *****
DO 630 N=1,NMEM
WRITE(5,640)N,ELMF(N,1),SUMP(N),(ELMF(N,M),M=2,3),SUMA(N),
1(ELMF(N,M),M=5,6),SUMB(N)
640 FORMAT(8X,I2,8(3X,E11.4))
630 CONTINUE
IF(NELAS)1002,1002,600
1002 WRITE(5,1000)

```

```

                                ELEFD
. 1000 FORMAT( /25X,32HELEMENT END ROTATIONS IN DEGREES,/,24X,5HEND A,
123X,54END B,/,
                                /,6X,4HELEM,6X,7HELASTIC,7X,7H
2PLASTIC,7X,7HELASTIC,7X,7HPLASTIC,8X,4HP/PP)
DO 635 N=1,NMEM
RAT=ABS(ELMF(N,1)/PY(N))
WRITE(5,640)N, TEB(N),TBA(N), TEBB(N),TBB(N),RAT
635 CONTINUE
600 RETURN
END

```

```

SUBROUTINE PRINT
DIMENSION UMAX(30,3),UMIN(30,3),YMAX(30,3),TMIN(30,3)
DIMENSION R(60),RD(60),RDR(60),IOLD(30,2)
DIMENSION SAB(60,60),SAA(60,60),SBB0(60,60)
DIMENSION CMNP(30,2),CMNV(30,2),CMP(30,2),CMPV(30,2),CLPM(30,2),
1CLPV(30,2),CLNM(30,2),CLNV(30,2),PY(30),JA(30),JB(30)
DIMENSION CMOMP(30,2),CMOMN(30,2),CLOAD(30,2),TMOMP(30,2),
1TMOMN(30,2),TIMEL(30,2),YMY(30,2),          JOINT(2)
DIMENSION ILASTA(30),ILASTB(30)
DIMENSION TEBB(30),TEBA(30),DUTR(30,4)
DIMENSION TITLE(20)
DIMENSION VL(30),CN(30),SN(30),A(30),AI(30)
DIMENSION RMAX(30,4),RMIN(30,4),TIM(30,4),TMM(30,4)
DIMENSION FUNIT(2,2),XLUNIT(2),ISENSE(3)
DIMENSION PYMAX(30,2),PYMIN(30,2)
COMMON NUNIT,PYMAX,PYMIN
COMMON ICOUN,NCOUN,TITLE
COMMON RMAX,RMIN,TIM,TMM,TEBA,TEBB
COMMON ILASTA,ILASTB,NELAS
COMMON UMAX,UMIN,TMAX,TMIN
COMMON NSKIP,INDEXW
COMMON SAB,SAA,SBB0
COMMON R,RD,RDR,IOLD
COMMON PY,IPRD,IPRNP
COMMON NDT,LL,NMEM,YMY,          CLOAD,CMOMP,CMOMN,TMOMP,TMOMN,
1TIMEL,CMNP,CMNV,CMP,CMPV,CLPM,CLPV,CLNM,CLNV,NNODE,JA,JB
COMMON          VL,CN,SN,A,AI,E
DATA FUNIT/5H(KNS),5H(LBS),8H(KN-M),8H(LB-IN)/
DATA XLUNIT/4H(MM),4H(IN)/
DATA ISENSE/1HC,1HT,1H /
WRITE(5,1700)TITLE
1700 FORMAT(1H1,20X,20A4/)
WRITE(5,200)(FUNIT(NUNIT,J),J=1,2),FUNIT(NUNIT,1),FUNIT(NUNIT,2)
1 ,FUNIT(NUNIT,2),FUNIT(NUNIT,1),FUNIT(NUNIT,1)
NCOUN=3
ICOUN=57
C *****
C COMPUTE DUCTILITY RATIOS
C *****
DO 500 M=1,NMEM
DO 3 J=1,4
3 DUTR(M,J)=1.0
IF(AI(M))1,1,19
1 AI(M)=1.0
19 IF(NELAS)6,6,9
6 IF(RMAX(M,1))14,15,14
14 DUTR(M,1)=1.0 + ABS(RMAX(M,2)/RMAX(M,1))
15 IF(RMIN(M,1))16,5,16
16 DUTR(M,2)=1.0+ABS(RMIN(M,2)/RMIN(M,1))
5 IF(RMAX(M,3))21,22,21
21 DUTR(M,3)=1.0 +ABS(RMAX(M,4)/RMAX(M,3))
22 IF(RMIN(M,3))23, 9,23
23 DUTR(M,4)=1.0 + ABS(RMIN(M,4)/RMIN(M,3))
9 JOINT(1)=JA(M)
JOINT(2)=JB(M)
C *****
C PRINT TABULATION OF MAXIMA AND MINIMA OF ELEMENT END LOADS

```

PRINT

```
C *****
DO 10 K=1,2
R TO1=CMOMP(M,K)/YMY(M,K)
RATO2=CMOMN(M,K)/YMY(M,K)
WRITE(5,100)M,JOINT(K),CLOAD(M,1),CLPH(M,K),CLPV(M,K),TIMEL(M,1),
1YMY(M,K),CMOMP(M,K),CMPP(M,K),CMPV(M,K),THOMP(M,K),RATO1
10 WRITE(5,110)CLOAD(M,2),CLNM(M,K),CLNV(M,K),TIMEL(M,2),YMY(M,K),
1CMOMN(M,K),CMNP(M,K),CMNV(M,K),THOMN(M,K),RATO2
NCOJN=NCOUN+6
IF(NCOUN-ICOUN)500,499,499
499 IF(M-NMEM)498,500,500
498 WRITE(5,120)
WRITE(5,1700)TITLE
WRITE(5,200)(FUNIT(NUNIT,J),J=1,2),FUNIT(NUNIT,1),FUNIT(NUNIT,2)
1 ,FUNIT(NUNIT,2),FUNIT(NUNIT,1),FUNIT(NUNIT,1)
NCOUN=6
500 CONTINUE
200 FORMAT(35X,51HTABULATION OF MAXIMUM AND MINIMUM ELEMENT END LOADS,
1 //, 1X,4HELEM,1X,4HNODE,3X,4HMAXP,7X,7HASSOC.M,5X,7HASSOC.V,6X,
14HTIME,10X,2HMM,8X,5HMAX M,6X,7HASSOC.P,5X,7HASSOC.V,5X,4HTIME,5X,
27HMAXM/MM, /,13X,A5,6X,A8,5X,A5,6X,5H(SEC),8X,A1,3X,A8,
35X,A5,7X,A5,5X,5H(SEC))
100 FORMAT(1H0,1X,I2,1X,I2,1X,9E12.4,F6.2,2X,F6.2)
110 FORMAT(1H ,7X,9E12.4,F6.2)
WRITE(5,120)
120 FORMAT(1H0,94HA PLUS SIGN (+) FOR MAXP INDICATES COMPRESSION AND A
1 MINUS SIGN (-) FOR MAXP INDICATES TENSION)
300 FORMAT( 1X,33HMAXIMUM NODAL DISPLACEMENT TABLE )
310 FORMAT(1H0,4HNODE,6X,4HX(+),6X,4HTIME,6X,4HX(-),6X,4HTIME,6X,
14HY(+),6X,4HTIME,6X,4HY(-),6X,4HTIME,6X,4HR(+),6X,4HTIME,6X,
24HR(-),6X,4HTIME,/,11X,A4,6X,5H(SEC),5X,A4,6X,5H(SEC),5X,A4,
36X,5H(SEC),5X,A4,6X,5H(SEC),5X,5H(DEG),5X,5H(SEC),5X,5H(DEG),
45X,5H(SEC))
IF(NELAS)901,901,401
901 WRITE(5,1700)TITLE
WRITE(5,1000)
1000 FORMAT( 26X,29HMAXIMUM ELEMENT END ROTATIONS,31X,29HMINIMUM ELE
1MENT END ROTATIONS,/,21X,4HELAS,3X,4HTIME,6X,4HPLAS,3X,4HTIME,2X,
1 5HP/PP*,3X,15HDUCTILITY RATIO,5X,
24HELAS,3X,4HTIME,6X,4HPLAS,3X,4HTIME,2X,5HP/PP*,3X,15HDUCTILITY RA
3TIO/)
NCOJN=6
ICOUN=60
C *****
C PRINT TABULATION OF MAXIMA AND MINIMA OF ELEMENT END ROTATIONS
C *****
DO 400 I=1,NMEM
IS=ISENSE(1)
IS1=ISENSE(1)
IF(PYMAX(I,1).LT.0.0)IS=ISENSE(2)
IF(PYMAX(I,1).EQ.0.0)IS=ISENSE(3)
IF(PYMIN(I,1).LT.0.0)IS1=ISENSE(2)
IF(PYMIN(I,2).EQ.0.0)IS1=ISENSE(3)
PYMX=ABS(PYMAX(I,1))
PYMN=ABS(PYMIN(I,1))
WRITE(5,1500)I,JA(I),(RMAX(I,J),TIM(I,J),J=1,2),PYMX,IS,OUTR(I,1),
```

```

                                PRINT
1  (RMIN(I,J),TMM(I,J),J=1,2),PYMN,IS1,DUTR(I,2)
  IS=ISENSE(1)
  IS1=ISENSE(1)
  IF(PYMAX(I,2).LT.0.0)IS=ISENSE(2)
  IF(PYMAX(I,2).EQ.0.0)IS=ISENSE(3)
  IF(PYMIN(I,2).LT.0.0)IS1=ISENSE(2)
  IF(PYMIN(I,2).EQ.0.0)IS1=ISENSE(3)
  PYMX=ABS(PYMAX(I,2))
  PYMN=ABS(PYMIN(I,2))
  WRITE(5,1600)JB(I),(RMAX(I,J),TIM(I,J),J=3,4),PYMX,IS,DUTR(I,3),
1  (RMIN(I,J),TMM(I,J),J=3,4),PYMN,IS1,DUTR(I,4)
  NCOUN=NCOUN+5
  IF(NCOUN-ICOUN)400,398,398
398 NCOUN=6
  IF(I-NMEM)399,400,400
399 WRITE(5,1800)
  WRITE(5,1700)TITLE
1800 FORMAT(1X,46H*NOTE - THE LETTER C INDICATES COMPRESSION AND,/
1  9X,30H*THE LETTER T INDICATES TENSION)
  WRITE(5,1000)
400 CONTINUE
  WRITE(5,1800)
500 FORMAT(1X,4HELEM,I2,1X,4HNODE,I3,2(3X,F7.4,2X,F5.3),2X,F5.3,
1  A1,6X,F7.3,4X,
2  2(3X,F7.4,2X,F5.3),2X,F5.3,A1,5X,F7.3)
1600 FORMAT( 8X,4HNODE,I3,2(3X,F7.4,2X,F5.3),2X,F5.3,
1  A1,6X,F7.3,4X,
2  2(3X,F7.4,2X,F5.3),2X,F5.3,A1,6X,F7.3//)
401 WRITE(5,1700)TITLE
  WRITE(5,300)
  WRITE(5,310)XLUNIT(NUNIT),XLUNIT(NUNIT),XLUNIT(NUNIT),XLUNIT(NUNIT
1)
  NCOUN=5
  ICOUN=60
C *****
C PRINT TABULATION OF MAXIMA AND MINIMA OF NODAL DISPLACEMENTS
C *****
  DO 350 N=1,NNODE
  IF(NUNIT.EQ.2)GO TO 347
  DO 346 K=1,2
  UMAX(N,K)=1000.0*UMAX(N,K)
346 UMIN(N,K)=1000.0*UMIN(N,K)
347 UMAX(N,3)=57.29577951*UMAX(N,3)
  UMIN(N,3)=57.29577951*UMIN(N,3)
  WRITE(5,320)N,(UMAX(N,K),TMAX(N,K),UMIN(N,K),TMIN(N,K),K=1,3)
  NCOUN=NCOUN+2
  IF(NCOUN-ICOUN)350,348,348
348 IF(N-NNODE)349,350,350
349 NCOU4=5
  WRITE(5,1700)TITLE
  WRITE(5,300)
  WRITE(5,310)XLUNIT(NUNIT),XLUNIT(NUNIT),XLUNIT(NUNIT),XLUNIT(NUNIT
1)
350 CONTINUE
320 FORMAT(1HC,I4,12(1X,F9.5))
  RETURN

```

END

PRINT



```

C
C
C
SUBROUTINE TRANS(CNA,SNA,T)
*****
ROTATIONAL TRANSFORMATION MATRIX T
*****
DIMENSION T(6,6)
DO 75 K=1,6
DO 75 L=1,6
75 T(K,L)=J,C
T(1,1)=CNA
T(2,2)=CNA
T(4,4)=CNA
T(5,5)=CNA
T(1,2)=SNA
T(2,1)=-SNA
T(4,5)=SNA
T(5,4)=-SNA
T(3,3)=1.0
T(6,6)=1.0
RETURN
END

```

	SUBROUTINE MINV(A,N,D,L,M)	
C	
C	COMPUTE INVERSE OF A MATRIX	
C	
	DIMENSION A(1),L(1),M(1)	MINV 2
C	SEARCH FOR LARGEST ELEMENT	MINV 3
	D=1.0	MINV 4
	NK=-N	MINV 5
	DO 00 K=1,N	MINV 6
	NK=NK+N	MINV 7
	L(K)=K	MINV 8
	M(K)=K	MINV 9
	KK=NK+K	MINV 10
	BIGA=A(KK)	MINV 11
	DO 20 J=K,N	MINV 12
	IZ=N*(J-1)	MINV 13
	DO 20 I=K,N	MINV 14
	IJ=IZ+I	MINV 15
	10 IF(ABS(BIGA)- ABS(A(IJ))) 15,20,20	MINV 16
	15 BIGA=A(IJ)	MINV 17
	L(K)=I	MINV 18
	M(K)=J	MINV 19
	20 CONTINUE	MINV 20
C	INTERCHANGE ROWS	MINV 21
	J=L(K)	MINV 22
	IF(J-K) 35,35,25	MINV 23
	25 KI=K-N	MINV 24
	DO 30 I=1,N	MINV 25
	KI=KI+N	MINV 26
	HOLD=-A(KI)	MINV 27
	JI=KI-K+J	MINV 28
	A(KI)=A(JI)	MINV 29
	30 A(JI)=HOLD	MINV 30
C	INTERCHANGE COLUMNS	MINV 31
	35 I=M(K)	MINV 32
	IF(I-K) 45,45,38	MINV 33
	38 JP=N*(I-1)	MINV 34
	DO 40 J=1,N	MINV 35
	JK=NK+J	MINV 36
	JJ=JP+J	MINV 37
	HOLD=-A(JK)	MINV 38
	A(JK)=A(JJ)	MINV 39
	40 A(JJ)=HOLD	MINV 40
C	DIVIDE COLUMN BY MINUS PIVOT (VALUE OF PIVOT ELEMENT IS	MINV 41
C	CONTAINED IN BIGA)	MINV 42
	45 IF(ABS(BIGA)-1.E-20)46,46,48	MINV 43
	46 D=0.0	MINV 44
	RETURN	MINV 45
	48 DO 55 I=1,N	MINV 46
	IF(I-K) 50,55,50	MINV 47
	50 IK=NK+I	MINV 48
	A(IK)=A(IK)/(-DIGA)	MINV 49
	55 CONTINUE	MINV 50
C	REDUCE MATRIX	MINV 51
	DO 65 I=1,N	MINV 52
	IK=NK+I	MINV 53
	HOLD=A(IK)	MINV 54

	IJ=I-N	MINV	54
	DO 65 J=1,N	MINV	55
	IJ=IJ+N	MINV	56
	IF(I-K) 60,65,60	MINV	57
60	IF(J-K) 62,65,62	MINV	58
62	KJ=IJ-I+K	MINV	59
	A(IJ)=HOLD*A(KJ)+A(IJ)		
65	CONTINUE	MINV	61
C	DIVIDE ROW BY PIVOT	MINV	62
	KJ=K-N	MINV	63
	DO 75 J=1,N	MINV	64
	KJ=KJ+N	MINV	65
	IF(J-K) 70,75,70	MINV	66
70	A(KJ)=A(KJ)/BIGA	MINV	67
75	CONTINUE	MINV	68
C	PRODUCT OF PIVOTS	MINV	69
	D=D*BIGA	MINV	70
C	REPLACE PIVOT BY RECIPROCAL	MINV	71
	A(KK)=1.0/BIGA	MINV	72
80	CONTINUE	MINV	73
C	FINAL ROW AND COLUMN INTERCHANGE	MINV	74
	K=N	MINV	75
100	K=K-1	MINV	76
	IF(K) 150,150,105	MINV	77
105	I=L(K)	MINV	78
	IF(I-K) 120,120,100	MINV	79
100	JQ=N*(K-1)	MINV	80
	JR=N*(I-1)	MINV	81
	DO 110 J=1,N	MINV	82
	JK=JQ+J	MINV	83
	HOLD=A(JK)	MINV	84
	JJ=JR+J	MINV	85
	A(JK)=-A(JJ)	MINV	86
110	A(JI) =HOLD	MINV	87
120	J=M(K)	MINV	88
	IF(J-K) 100,100,125	MINV	89
125	KI=K-N	MINV	90
	DO 130 I=1,N	MINV	91
	KI=KI+N	MINV	92
	HOLD=A(KI)	MINV	93
	JJ=KI-K+J	MINV	94
	A(KI)=-A(JJ)	MINV	95
130	A(JI) =HOLD	MINV	96
	GO TO 100	MINV	97
150	RETURN	MINV	98
	END	MINV	99

```

SUBROUTINE YIELD(EM,EP,YM,YP,IX,TM,TP,IOLD,XP,XM,PCR,SUMP,SUMM)
*****
C MONITOR ELEMENT END LOADS TO DETERMINE WHETHER ELEMENT IS IN
C EITHER THE ELASTIC OR PLASTIC CONDITIONS
C *****
IF(EP)75,75,50
50 P=PCR
XMY=YM
C XMY=(1.0-ABS(EP)/P)*YM
GO TO 90
75 P=YP
XMY=YM
90 IF(XMY.NE.0)GO TO 95
AB1=0.0
GO TO 96
95 AB1=ABS(EM)/XMY
96 AB2=ABS(EP)/P
SUMP=AB2
SUMM=AB1
SUM1=AB1+AB2
IF(IOLD-1)100,200,300
200 DM=EM-XM
OP=EP-XP
ABM=ABS(XM)
ABP=ABS(XP)
IF(XM.NE.0)GO TO 225
RM=0.0
GO TO 230
225 RM=XM*DM/(ABM*XMY)
230 RP=XP*OP/(ABP*P)
SUM=RM+RP
IF(SJM)300,350,350
350 IF(SJM-1.0)300,600,600
600 IX=1
TM=EM
TP=EP
RETURN
300 IX=0
TM=EM
TP=EP
RETURN
100 IF(SUM1-1.0) 300,400,400
400 IOLD=1
IX=1
TM=EM
TP=EP
RETURN
END

```

```

SUBROUTINE PDEL (TELM ,C,ELMF,ELMDF,VL,RD,RDR,NINTL,NOETL,NINOF,
1 NOEDF,T,NMEM,NOEL,FAT)
C *****
C COMPUTE EQUIVALENT SHEARS FOR THE P-DELTA AND BEAM COLUMN EFFECTS
C *****
DIMENSION TELMD(6,1),C(6,1),ELMF(38,6),ELMDF(6,1),TELM(6,1),
1 VL(38),RD(60),RDR(60),NINOF(60),NOEDF(60),T(6,6),TT(6,6),F(6),
2 NOEL(6),FAT( 90, 90)
DO 5 I=1,6
5 TELMD(I,1)=TELM(I,1)-C(I,1)
DELV= ELMF(NMEM,1)*(C(2,1)-C(5,1)) + ELMDF(1,1)*((TELMD(2,1) +
1 C(2,1)) - (TELMD(5,1) + C(5,1)))
DELV=-DELV/VL(NMEM)
DO 10 I=1,6
DO 10 J=1,6
10 TT(J,I)=T(I,J)
DO 20 I=1,6
20 F(I)=(TT(I,2)-TT(I,5))*DELV
DO 75 I=1,6
DO 50 J=1,NINTL
IF(NOEL(I)-NINOF(J))50,25,50
25 RD(J)=RD(J)+F(I)
GO TO 75
50 CONTINUE
DO 60 J=1,NOETL
IF(NOEL(I)-NOEDF(J))60,56,60
56 DO 58 K=1,NINTL
58 RDR(K)=RDR(K) + FAT(K,J)*F(I)
GO TO 75
60 CONTINUE
75 CONTINUE
1000 FORMAT(10E12.5)
RETURN
END

```

APPENDIX E
LIST OF SYMBOLS

APPENDIX E

LIST OF SYMBOLS USED IN TEXT

A	Area of cross-section (cm^2)
A_b	Cross-sectional area of bracing member (cm^2)
A_1, A_2	Tributary areas assigned to mass points on blastward wall (Fig 31), roof (Fig 32a) and leeward wall (Fig 34 and 35) (m^2)
a	Maximum dimension of tributary area measured parallel to direction of propagation of shock front (m)
b_f	Flange width (mm)
b_h	Tributary width for horizontal loading on exterior column of blastward wall (m)
b_v	Tributary width for vertical loading on roof girder (m)
C	Coefficient for computing natural period of bending mode of vibration of beam (Eq (35))
C	Ratio of exterior column-to-girder moment capacities (Table 1 through 3)
C_c	Limiting slenderness ratio for frame member (Eq (4.1), Ref 1)
C_D	Drag coefficient (Ref 3)
$C_{r\alpha}$	Peak-reflected pressure coefficient at angle of incidence α_f
C_1	Ratio of interior column-to-girder moment capacities (Tables 1 through 3)
C_2	Frame stiffness factor constant (Table 8)
[C]	Damping matrix of system (kN-sec/m)

c	Dynamic increase factor
D	Distance from the blastward surface of the frame to the leading edge of the tributary area measured parallel to the direction of the blastwave (m)
D	Girder-to-column stiffness coefficient (Table 8)
DF	Distribution factor
DLF	Dynamic load factor for preliminary design
DOF	Degree of freedom
D_H	Horizontal translation of nodal point (m)
D_V	Vertical translation of nodal point (m)
d	Web depth (mm)
d	Horizontal distance from blastward wall to leading edge of tributary area on roof (Fig 32b) (m)
d_2	Horizontal distance from edge of tributary width supported by frame to edge of tributary area on blastward column (Fig 31) (m)
d_2	Distance from blastward wall to leading edge of tributary area on roof (Fig 32a) (m)
E	Young's modulus of elasticity (kPa)
F_a	Maximum compressive stress permitted in the absence of bending moment (kPa)
F_{dy}	Dynamic yield stress (kPa)
F_y	Specified minimum static yield stress (kPa)
$\{F(t)\}$	Matrix of applied transient loads (kN)
$\{F^*\}_T$	Equivalent force matrix (Eq (6), Appendix C) (KN)
g	Acceleration due to gravity (9.8 m/sec ²)

H	Story height (m)
H	Height of wall panel (m)
h_1	Vertical distance from roof to upper boundary of tributary area on leeward wall (Fig 33) (m)
h_2	Height of tributary area (Fig 33) (m)
I	Moment of inertia (cm^4)
I_x, I_y	Moments of inertia about the x- and y-axes, respectively (cm^4)
i_r	Unit positive normal reflected impulse (psi)
i_s	Unit positive incident impulse (psi)
K	Stiffness factor for single-story, multi-bay rigid frame subjected to uniform horizontal loads (Table 8) (kN/m)
K	Relative rotational stiffness of frame member (Eq (31) kN/m)
K_L	Load factor that modifies stiffness factor, K, for single-story, multi-bay rigid frame
K_b	Stiffness of horizontal bracing system of a braced frame (kN/m)
[K]	Stiffness matrix of system (kN/m)
k_c	Total stiffness of element (kN/m)
z_{ir}	Horizontal distance from blastward end of roof to mass point on roof (Eq (17) and (18)) (m)
z_{iz}	Vertical distance from roof to mass point i on leeward wall (Eq (19) and (20)) (m)
z_1	Horizontal distance from blastward wall to leading edge of tributary area on roof (Fig 32a) (m)
z_2	Length of tributary area (Fig 32a) (m)

l/r	Slenderness ratio in the plane of bending
M_A, M_B	Bending moments at ends A and B of element (kN-m) (Fig 63)
M_R	Mass of roof panel (kg)
M'_R	One-half the mass of decking within roof panel area $(W'_1 + W'_2) \times L$ plus mass of girder (Fig 19) (kg)
M_W	Mass of wall panel (kg)
M'_W	One-half of mass of siding within wall panel area $(W'_1 + W'_2) \times H$ plus mass of column (Fig 17) (kg)
M_e	Concentrated mass at end of a member (kg)
M_i	Summation of the masses at all intermediate mass points on a member (kg)
M_{mn}	Moment about the n-axis that can be resisted in the absence of axial load (kN-m)
M_{mx}, M_{my}	Moments about the x- and y-axes that can be resisted in the absence of axial load (kN-m)
$\bar{M}_{mx}, \bar{M}_{my}$	Reduced bending capacities about the x- and y-axes that can be resisted in the absence of axial load (kN-m)
M_p	Required plastic moment capacity (kN-m)
M_{px}, M_{py}	Design plastic bending capacities about the x- and y-axes (kN-m)
M_x, M_y	Peak-applied moments about the x- and y-axes (kN-m)
$(M_{EH})_R$	Portion of roof panel mass assigned to horizontal dynamic-degrees-of-freedom at ends of girder (kg)
$(M_{EH})_W$	Portion of wall panel mass assigned to horizontal dynamic-degrees-of freedom at ends of column (kg)

$(M_{EV})_R$	Portion of roof panel mass assigned to vertical dynamic-degrees-of-freedom at ends of girder (kg)
$(M_{EV})_W$	Portion of wall panel mass assigned to vertical dynamic-degrees-of-freedom at ends of column (kg)
$(M_{IV})_R$	Portion of roof or floor panel mass assigned to vertical-dynamic-degrees-of-freedom at intermediate mass points on girder (kg)
$(M_{IH})_W$	Portion of wall panel mass assigned to horizontal dynamic-degrees-of-freedom at intermediate mass point on column (kg)
$(M_p)_n$	Required plastic bending capacity about n-axis (kN-m)
$\begin{matrix} (M_p)_x \\ (M_p)_y \end{matrix}$	Required plastic bending capacities about x- and y-axes (kN-m)
$(M_u)_i$	Unbalanced moment on element at time i due to P- Δ and beam column effects (Eq (10), Appendix C) (kN-m)
[M]	Mass matrix of system (kg)
[M*]	Equivalent mass matrix (Eq (6), Appendix C) (kg)
m	Number of braced bays in multi-bay frame
m_e	Equivalent mass of a frame for sideway natural period (Eq (36)) (kg)
m_g	Mass of one girt within wall panel (kg)
m_p	Mass of one purlin within roof panel (kg)
NDT	Number of integration time increments for analysis specified in the input
N_{DOFH}	Number of horizontal dynamic-degrees-of-freedom at intermediate mass points within wall panel (Fig 17)
N_{DOFV}	Number of vertical dynamic-degrees-of-freedom at intermediate mass points within roof panel (Fig 19)

n	Number of bays in a multi-bay frame
n	Number of diagonal braces
n_g	Number of girts within wall panel (Fig 17)
n_p	Number of purlins within roof panel (Fig 19)
P	Applied axial load (kN)
P_A, P_B	Axial loads at ends A and B of element (Fig 63) (kN)
P_C	Axial load at yield (kN)
P_p	Ultimate tensile load capacity of member (kN)
P_{pk}	Peak pressure initially acting at mass point (kPa)
P_r	Peak positive normal reflected pressure (kPa)
P_{ra}	Peak-reflected pressure at angle of incidence α_i (kPa)
P_s	Incident pressure at time t_c (kPa)
P_{s0}	Peak positive incident pressure (kPa)
P_u	Ultimate compressive load capacity of member (kN)
$(P_{pk})_{AVG}$	Average peak pressure acting on tributary area (kPa)
$\{P(t)\}$	Matrix of applied blast loads (Eg (9), Appendix C) (kN)
p	Infinitely elastic fraction of total element stiffness
P_h	Peak reflected blast pressure on blastward wall (kPa)
P_v	Peak pressure on roof (kPa)
q	Elasto-plastic fraction of total element stiffness

q_h	Peak horizontal load on frame (kN/m)
q_0	Peak dynamic pressure (kPa)
q_v	Peak vertical load on frame (kN/m)
R	Slant distance from explosive charge to point of interest on structure (m)
R_A	Normal distance from explosive charge to point of interest on structure (m)
R_C	Rotation capacity of beam column
R_n	Nodal reduction factor for bi-axial bending
R_α	DLF reduction factor for quartering loads
$\{R_i\}_\tau, \{R_d\}_\tau$	Resistance of system at time τ (Eq(3), Appendix C) (kN)
r_b	Radius of gyration of bracing member (m)
r_x, r_y	Radius of gyration about the x-and y-axes (m)
SF	Safety factor
T_N	Natural period of vibration (sec)
T_a	Natural period of vibration of the primary extensional mode of a frame member (sec)
T_b	Natural period of a bending mode of vibration of a beam (sec)
T_f	Desired duration of response (sec)
T_s	Sideway natural period of vibration of a frame (sec)
t	t (sec)
t_a	Arrival time of blast wave (sec)
t_c	Clearing time required to relieve the reflected pressure (sec)

t_c	Fictitious clearing time of reflected pressure (sec)
t_f	Flange thickness (mm)
t_{DI}	Duration of pressure-time history (sec)
t_{dr}	Total duration of blast pressure (sec)
t_{pk}	Time at which pressure $(P_{pk})_{AVG}$ occurs (sec)
t_R	Fictitious duration of the reflected pressure (sec)
t_{of}	Positive phase pressure duration (sec)
t_{rt}	Rise time (sec)
t_T	Time at which the pressure decays to zero (sec)
t_w	Web thickness (mm)
U	Shock front velocity (m/sec)
U_{AVG}	Average shock front velocity between blastward end of frame and mass point (m/sec)
u	Nodal point displacement (mm)
\dot{u}	Nodal point velocity (mm/sec)
\ddot{u}	Nodal point acceleration (mm/sec ²)
V	Equivalent shear computed for second order effects (kN)
V_A, V_B	Shears at ends A and B of element (Fig 63) (kN)
W	Charge weight (kg)
W_E	Effective charge weight (kg)
W_T	Tributary width of structure supported by frame (m)
W_1, W_2	One-half of widths of framing bays adjacent to frame (m)

W'_1, W'_2	Distances between frame and adjacent girts and/or purlins (m)
w	Load per unit length (kN/m)
w	Width of tributary area on blastward wall (Fig 31), roof (Fig 32) and leeward wall (Fig 33) (m)
w_b	Equivalent static load for beam mechanism (kN/m)
w_p	Equivalent static load for panel mechanism (kN/m)
w_1	Horizontal distance from end of building to nearest edge of tributary strip supported by frame (Fig 34 and 35) (m)
w_2	Horizontal distance between edge of strip supported by frame and tributary area (Fig 34 and 35) (m)
w_3	Width of tributary area (Fig 34 and 35) (m)
y_A, y_B	Displacements at ends A and B of element (mm)
Z	Scaled distance ($m/kg^{1/3}$)
Z	Plastic section modulus (cm^3)
Z_x, Z_y	Plastic section moduli about the x- and y-axes (m^3)
Z_n	Plastic section modulus about n-axis (m^3)
α	Ratio of horizontal-to-vertical load on a frame
α	Rigid body component of element end rotation (deg)
α_j	Angle of incidence between the shock front and blastward surface (deg)
β	Base fixity factor for single story frame (Table 8)
β	Plastic rotation component of element end rotation (deg)
β_{max}	Maximum plastic member or element end rotation (deg)

γ	Total element end rotation (deg)
γ	Angle between bracing member and horizontal plane (deg)
Δ	Differential displacement between ends of element (mm)
Δt	Integration time increment (sec)
δ	Frame sideways deflection (mm)
θ	Support or member end rotation angle or chordal angle (deg)
θ	Nodal point rotation (deg)
θ_{max}	Maximum permitted support or member end rotation or chordal angle (deg)
μ	Ductility ratio
μ_{max}	Maximum permitted ductility ratio
μ_{ult}	Ultimate ductility ratio at failure
ϕ	Elastic component of element end rotation (deg)
ϕ_{max}	Maximum elastic member or element end rotation (deg)

LIST OF SYMBOLS USED IN INPUT TO DYNFA

A	Cross sectional area of element (m^2)
ADIR	Direction of element uniform loads specified in terms of the axis of global system in the direction of the loads
AFX	Tributary area associated with dynamic-degree-of-freedom in X-direction (m^2)
AFY	Tributary area associated with dynamic-degree-of-freedom in Y-direction (m^2)
DAMP	Percentage of damping expressed as a decimal
DT	Integration time interval (sec)
E	Modulus of elasticity (kPa)
ELEM NO	Element identification number
END	End of data indicator
FX	Static load in X-direction (kg)
FY	Static load in Y-direction (kg)
I	Moment of inertia of element (cm^4)
IMPV	Input parameter for requesting printing of element end loads
INDXX	Identification number of pressure waveform associated with load in X-direction
INDYY	Identification number of pressure waveform associated with load in Y-direction
IPP	Input parameter for requesting printing of nodal point displacement-time histories
JA	Nodal point at end A of element
JB	Nodal point at end B of element

M_{mA}	Ultimate bending capacity at end A of element (kN-m)
M_{mB}	Ultimate bending capacity at end B of element (kN-m)
MX	Nodal mass assigned to dynamic degree-of-freedom in X-direction (kg)
MY	Nodal mass assigned to dynamic degree-of-freedom in Y-direction (kg)
MZ	Static moment around Z-axis (kg-m)
NDEAD	Input parameter for omitting static analysis
NDT	Number of integration time increments
NELAS	Input parameter for omitting inelastic behavior from analysis
NELM	Number of elements
NLNODE	Number of nodal points with static loads
NNODE	Number of nodal points
NNOF	Number of nodal points subjected to blast loads
NNOR	Number of nodal points with restraints
NNOW	Number of nodal points with assigned masses (number of mass points)
NODE NO	Nodal point number
NPOINT	Number of points in pressure waveform
NSKIP	Number of integration time increments to be skipped between printout of structure's response
NU	Input parameter for specifying U.S. System of Units for input and output of program
NWF	Number of pressure waveforms

P_A	Pin code for end A of element
P_B	Pin code for end B of element
P_i	Pressure of point i of pressure waveform (kPa)
P_p	Ultimate dynamic load capacity in axial tension (kN)
P_u	Ultimate dynamic load capacity in axial compression (kN)
r_x	X-direction nodal restraint
r_y	Y-direction nodal restraint
r_θ	Rotational nodal restraint
T_i	Time of point i of pressure waveform (sec)
WF NO	Pressure waveform identification number
WUNIF	Uniform static load acting on element (kg/m)
X	X-direction nodal coordinate (m)
Y	Y-direction nodal coordinate (m)
YFACT	Fraction of elastic stiffness utilized after yielding

APPENDIX F

INTERNATIONAL SYSTEM OF UNITS

APPENDIX F

INTERNATIONAL SYSTEM OF UNITS

F.1 General

This appendix deals with the conversion of quantities from the U. S. System of measurement to the International System of Units which is officially abbreviated as SI in all languages. It includes units most frequently used in the various fields of science and industry and conforms to the Metric Practice Guide as presented in the American Society for Testing and Materials Standard E 380.

F.2 SI Units and Prefixes

SI consists of seven base units, two supplementary units, a series of derived units consistent with the base and supplementary units, and a series of approved prefixes for the formation of multiples and sub-multiples of various units. A summary of the base units, supplementary units, derived units and prefixes is given below:

Base Units

Quantity	Unit	Symbol
Length	meter	m
Mass	kilogram	kg
Time	second	s
Electric current	ampere	A
Thermodynamic temperature	kelvin	K
Amount of substance	mole	mol
Luminous intensity	candela	cd

Supplementary Units

Quantity	Unit	Symbol
Plane angle	radian	rad
Solid angle	steradian	sr

Derived Units

Quantity	Unit	Symbol or Formula
Acceleration	meter per second squared	m/s^2
Activity (radioactive source)	disintegration per second	(disintegration)/s
Angular acceleration	radian per second squared	rad/s^2
Angular velocity	radian per second	rad/s
Area	square meter	m^2
Density	kilogram per cubic meter	kg/m^3
Electric capacitance	farad	F or A.s/V
Electrical conductance	siemens	S or A/V
Electric field strength	volt per meter	V/m
Electric inductance	henry	H or V.s/A
Electric potential difference	volt	V or W/A
Electric resistance	ohm	Ω or V/A
Electromotive force	volt	V or W/A
Energy	joule	J or N.m
Entropy	joule per kelvin	J/K
Force	newton	N or $kg.m/s^2$
Frequency	hertz	Hz or (cycle)/s
Illuminance	lux	lx or lm/m^2
Luminance	candela per square meter	cd/m^2
Luminous flux	lumen	lm or $cd.sr$
Magnetic field strength	ampere per meter	A/m
Magnetic flux	weber	Wb or V.s
Magnetic flux density	tesla	T or Wb/m^2
Magnetomotive force	ampere	A
Power	watt	W or J/s
Pressure	pascal	Pa or N/m^2
Quantity of electricity	coulomb	C or A.s
Quantity of heat	joule	J or N.m
Radiant intensity	watt per steradian	W/sr
Specific heat	joule per kilogram-kelvin	J/kg.K
Stress	pascal	Pa or N/m^2
Thermal conductivity	watt per meter-kelvin	W/m.K
Velocity	meter per second	m/s
Viscosity, dynamic	pascal-second	Pa.s
Viscosity, kinematic	square meter per second	m^2/s
Voltage	volt	V or W/A

Derived Units (continued)

Quantity	Unit	Symbol or Formula
Volume	cubic meter	m ³
Wavenumber	reciprocal meter	(wave)/m
Work	joule	J or N.m

Prefixes

Multiplication	Prefix	Symbol
1,000,000,000,000 = 10 ¹²	tera	T
1,000,000,000 = 10 ⁹	giga	G
1,000,000 = 10 ⁶	mega	M
1,000 = 10 ³	kilo	k
100 = 10 ²	hecto*	h
10 = 10 ¹	deka*	da
0.1 = 10 ⁻¹	deci*	d
0.01 = 10 ⁻²	centi*	c
0.001 = 10 ⁻³	milli	m
0.000,001 = 10 ⁻⁶	micro	μ
0.000,000,001 = 10 ⁻⁹	nano	n
0.000,000,000,001 = 10 ⁻¹²	pico	p
0.000,000,000,000,001 = 10 ⁻¹⁵	femto	f
0.000,000,000,000,000,001 = 10 ⁻¹⁸	atto	a

*To be avoided where possible

Selection of prefixes representing steps of 1,000 is recommended for most multiple and submultiple prefixes. An exception is made in the case of area and volume used alone. When expressing a quantity by a numerical value and a unit, prefixes should preferably be chosen so that the numerical value lies between 0.1 and 1,000, except where certain multiples or submultiples have been agreed to for particular use. The same unit, multiple or submultiple is used for tabular values even though the series exceeds the preferred range 0.1 to 1,000. Double prefixes should not be used. Prefixes should not be used in the denominator of compound units, except for the kilogram which is a base unit of SI. However, prefixes may be applied to numerator of a combined unit. With SI units of higher order such as m², m³, etc., the prefix is also raised to the same order; e.g., mm³ is 10⁻⁹m³ and not 10⁻³m³. In such cases, the use of cm², cm³, dm², dm³ and similar nonpreferred prefixes is acceptable.

F.3 Mass, Force and Weight

The principal departure of SI from the gravimetric form of metric engineering units is the separate and distinct units for mass and force. The kilogram is restricted to the unit of mass. The newton is the unit of force and should be used in place of kilogram-force. Likewise, the newton instead of kilogram-force should be used in combination units which include force; e.g., pressure or stress ($N/m^2=Pa$), energy ($N.m=J$), and power ($N.m/s=W$).

Considerable confusion exists in the use of the terms mass and weight. Mass is a property of matter to which it owes its inertia. If a body or particle of matter at rest on the earth's surface is released from the forces holding it at rest, it will experience the acceleration of free fall (or acceleration of gravity, g). The force required to restrain it against free fall is commonly called weight. This force is proportional to the mass of the body and is often expressed in mass units (kg), but as it is a force it should be expressed in force units (N). The acceleration of free fall (g) varies in time and space; weight (which is proportional to it) does too, although mass does not. Further confusion arises in the measuring of weight because of the buoyant effect of the medium in which the weighing is performed. In common parlance the term weight is used where the technically correct word is mass and, therefore, the use of the term weight should be avoided in technical practice.

F.4 Conversion and Rounding Rules

Conversion of quantities should be handled with careful regard to the implied correspondence between the accuracy of the data and the given number of digits. In all conversions, the number of significant digits retained should be such that accuracy is neither sacrificed nor exaggerated. Proper conversion procedure is to multiply the specific quantity by the conversion factor exactly as given and then round to the appropriate number of significant digits.

The following table contains conversion factors that give exact or seven-figure accuracy where the nature of the dimension makes this degree of accuracy practical:

Selected Conversion Factors

To convert from	to	Multiply by
atmosphere (technical = 1 kfg/cm ²)	pascal (Pa)	9.806 650*x10 ⁴
bar	pascal (Pa)	1.000 000*x10 ⁵
board foot	meter ³ (m ³)	2.359 737x10 ⁻³
British thermal unit (In- ternational Table)	joule (J)	1.055 056x10 ³
Btu (International Table) - in./s-ft ² -°F (k, thermal conductivity)	watt/meter-kelvin (W/m.K)	5.192 204x10 ²
Btu (International Table) /hour	watt (W)	2.930 711x10 ⁻¹
calorie (International Table)	joule (J)	4.186 800*
centipoise	pascal-second (Pa.s)	1.000 000*x10 ⁻³
centistokes	meter ² /second (m ² /s)	1.000 000*x10 ⁻⁶
degree (angle)	radian (rad)	1.745 329x10 ⁻²
degree Fahrenheit	degree Celsius	t°C=(t°F-32)/1.8
fluid ounce (U.S.)	meter ³ (m ³)	2.957 353x10 ⁻⁵
foot	meter (m)	3.048 000*x10 ⁻¹
foot ²	meter ² (m ²)	9.290 304*x10 ⁻²
foot ³ (volume and section modulus)	meter ³ (m ³)	2.831 685x10 ⁻²
foot/second	meter/second (m/s)	3.048 000*x10 ⁻¹
foot-pound-force	joule (J)	1.355 818
foot-pound-force/second	watt (W)	1.355 818
foot/second ²	meter/second ² (m/s ²)	3.048 000*x10 ⁻¹
gallon (U.S.liquid)	meter ³ (m ³)	3.785 412x10 ⁻³
horsepower (electric)	watt (W)	7.460 000*x10 ²
inch	meter (m)	2.540 000*x10 ⁻²
inch ²	meter ² (m ²)	6.451 600*x10 ⁻⁴
inch ³ (volume and section modulus)	meter ³ (m ³)	1.638 706x10 ⁻⁵
inch of mercury (60°F)	pascal (Pa)	3.376 85x10 ³
inch of water (50°F)	pascal (Pa)	2.488 4 x10 ²
kilogram-force (kgf)	newton (N)	9.806 650*
kilogram-force/millimeter ²	pascal (Pa)	9.806 650*x10 ⁶
kilogram-mass	kilogram (kg)	1.000 000*
kip (1000 lbf)	newton (N)	4.448 222x10 ³
kip/inch ² (ksi)	pascal (Pa)	6.894 757x10 ⁶
lux	lumen/meter ² (lm/m ²)	1.000 000*
minute (angle)	radian (rad)	2.908 882x10 ⁻⁴
ounce-force (avoirdupois)	newton (N)	2.780 139x10 ⁻¹

*Exact

Selected Conversion Factors (Continued)

To convert from	to	Multiply by
ounce-mass (avoirdupois)	kilogram (kg)	2.834 952x10 ⁻²
ounce-mass/yard ²	kilogram/meter ² (kg/m ²)	3.390 575x10 ⁻²
ounce (avoirdupois) (mass)/inch ³	kilogram/meter ³ (kg/m ³)	1.729 994x10 ³
ounce (U.S. fluid)	meter ³ (m ³)	2.957 353x10 ⁻⁵
pint (U.S. liquid)	meter ³ (m ³)	4.731 765x10 ⁻⁴
pound-force (lbf avoirdupois)	newton (N)	4.448 222
pound/force/inch ² (psi)	pascal (Pa)	6.894 757x10 ³
pound-mass (lbm avoirdupois)	kilogram (kg)	4.535 924x10 ⁻¹
pound-mass-inch ² (moment of inertia)	kilogram-meter ² (kg-m ²)	2.926 397x10 ⁻⁴
pound-mass/inch ³	kilogram/meter ³ (kg/m ³)	2.767 990x10 ⁴
quart (U.S. liquid)	meter ³ (m ³)	9.463 529x10 ⁻⁴
second angle	radian(rad)	4.848 137x10 ⁻⁶
ton (short, 2000 lbm)	kilogram (kg)	9.071 847x10 ²
watt-hour	joule (J)	3.600 000*x10 ³
yard	meter (m)	9.144 000*x10 ⁻¹
yard ²	meter ² (m ²)	8.361 274x10 ⁻¹
yard ³	meter ³ (m ³)	7.645 549x10 ⁻¹

*Exact

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