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THEORETICAL BASIS FOR CTABS80: A COMPUTER PROGRAM FOR THREE-DIM-ETC(U)

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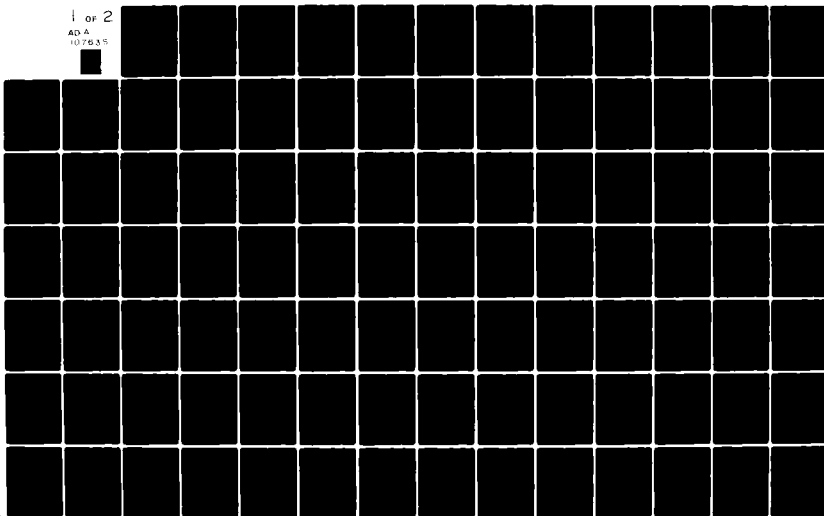
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TECHNICAL REPORT K-81-2

THEORETICAL BASIS FOR CTABS80: A COMPUTER PROGRAM FOR THREE-DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS

by

Edward L. Wilson, H. H. Dovey
Ashraf Habibullah

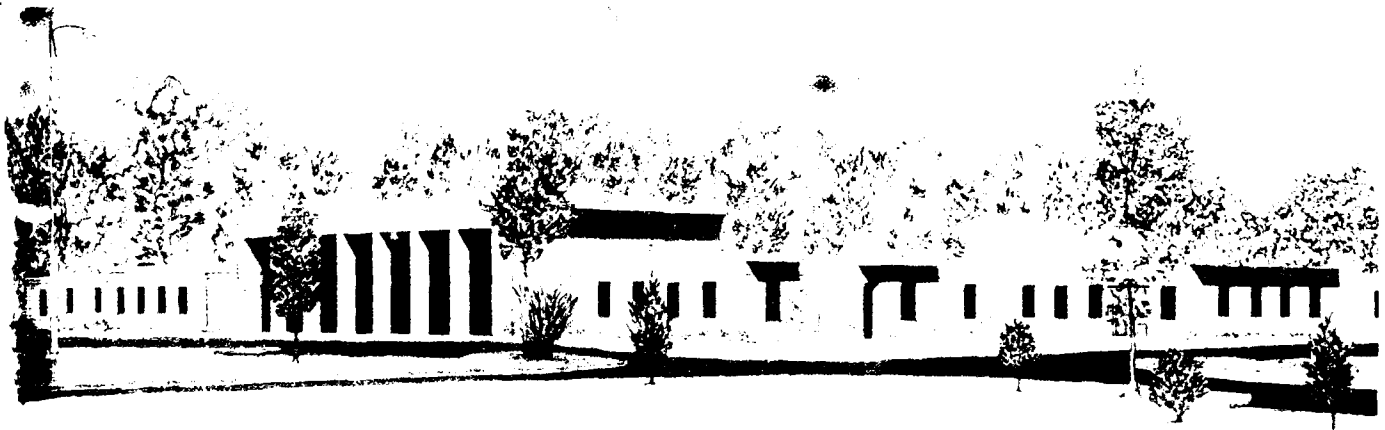
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P. O. Box 631, Vicksburg, Miss. 39180

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THEORETICAL BASIS FOR CTABS80

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20. ABSTRACT (Continued)

vertical columns (or piers) and horizontal beams (or spandrels). However, with special modeling techniques, very complex situations may be considered. A special shear panel element is developed to enable modeling of discontinuous shear walls and shear walls with arbitrary openings. A diagonal bracing system to model braced frames (X-braced, K-braced, or eccentrically braced systems) is also presented.

The column, shear panel, and diagonal formulations include the effects of bending, axial, and shear deformations. Bending and shear deformations are also included in the beam formulation; however, the effects of axial deformations are neglected.

The effects of the finite dimensions of the beams and columns on the stiffness of a frame or shear wall system are automatically included.

The buildings may be unsymmetrical and nonrectangular in plan. Torsional behavior and interstory compatibility are accurately reflected in the results.

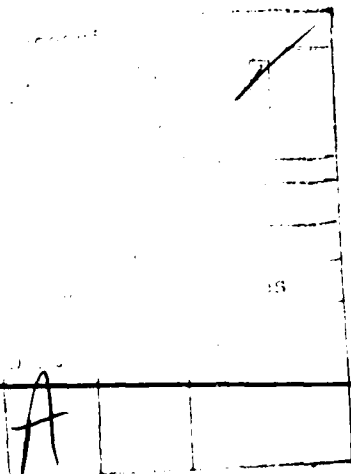
Four independent vertical and two independent lateral static load conditions are possible in any one run. These six static load conditions may be combined in any ratio to each other or to a lateral dynamic earthquake input which may be specified as a time-dependent ground acceleration or as an acceleration response spectrum.

Three-dimensional mode shapes and frequencies are evaluated.

The unique solution procedure used by CTABS80 considers the frame and shear walls as substructures, reduced with a modified wave front technique. This method results in a significant reduction in the program data preparation, computational effort, and storage requirements.

The consecutive levels of each of the individual frames can be arbitrarily connected to any (sequential by not necessarily consecutive) level of the structure, thereby making it possible for frames to bypass certain story levels. This option gives the program the capability to model partial diaphragms and multidiaphragms at any level.

A user's guide for the program is presented in Waterways Experiment Station Instruction Report K-81-9.



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PREFACE

This report presents the theoretical basis for a computer program called CTABS80 that can be used for static and dynamic analysis of multi-story frame and shear wall buildings. Dr. E. H. Wilson, University of California, Berkeley, was responsible for developing the original version of the program (TABS), sponsored mainly by a National Science Foundation Grant.

Modifications to the program to make it a more useful tool for Corps of Engineers' personnel were made by Mr. Ashraf Habibullah, Computers/Structures International, Oakland, Calif. His work was sponsored with funds provided to the Automatic Data Processing (ADP) Center, U. S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Miss., by the Military Programs Directorate of the Office, Chief of Engineers, U. S. Army (OCE), under the Computer-Aided Structural Engineering (CASE) Project. This report and a companion user's guide for CTABS80 are the work of Dr. Wilson and Messrs. H. H. Dovey and Habibullah.

Specifications for the modifications to TABS were provided by the members of the CASE Task Group on Building Systems. The following were members of the Task Group (though all may not have served for the entire period) during the period of modifications to the program:

- Mr. Dan Reynolds, Sacramento District (Chairman)
- Mr. Jerry Foster, Baltimore District
- Mr. Joseph Hartman, St. Louis District
- Mr. David Illias, Portland District
- Mr. Sefton Lucas, Memphis District
- Mr. Jun Ouchi, Pacific Ocean Division
- Mr. David Raisanen, North Pacific Division
- Mr. Pete Roszbach, Baltimore District
- Mr. James Simmons, Baltimore District
- Mr. Ollie Werner, Middle East Division
- Mr. Gene Wyatt, Mobile District

Dr. N. Radhakrishnan, Special Technical Assistant, ADP Center, WES, and CASE Project Manager, and Mr. Paul K. Senter, Computer-Aided Design Group (CADG), ADP Center, coordinated and monitored the work. Ms. Deborah K. Martin, CADG, supported the Task Group in changing the program to accept free-field input. Mr. Seymour Schneider, Military Programs Directorate, was the OCE point of contact. Mr. Donald L. Neumann was Chief, ADP Center.

Directors of WES during this period were COL J. L. Cannon, CE, COL N. P. Conover, CE, and COL T. C. Creel, CE. Technical Director was Mr. F. R. Brown.

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CONVERSION FACTORS, INCH-POUND TO METRIC (SI)
UNITS OF MEASUREMENT

Inch-pound units of measurement used in this report can be converted to metric (SI) units as follows:

<u>Multiply</u>	<u>By</u>	<u>To Obtain</u>
feet	0.3048	metres
inches	2.54	centimetres
kips (1000 lb force)	4.448222	kilonewtons
kips (force) per foot	14.593904	kilonewtons per metre
pounds (force) per square foot	47.880263	pascals
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre

THEORETICAL BASIS FOR CTABS80: A COMPUTER
PROGRAM FOR THREE-DIMENSIONAL ANALYSIS
OF BUILDING SYSTEMS

CHAPTER I: INTRODUCTION

A. Purpose

This report presents the theoretical basis for CTABS80, a computer program for the linear three-dimensional structural analysis of multistory frame and shear wall buildings subjected to static and dynamic loadings. A user's guide for the program is presented in Waterways Experiment Station (WES) Instruction Report K-81-9⁽¹⁸⁾.

B. General-Purpose Programs for Structural Analysis

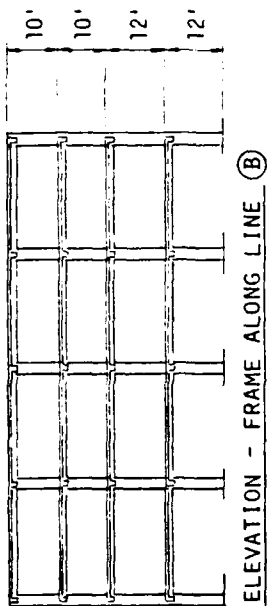
There are many two- and three-dimensional computer programs for the linear analysis of complex structures ^(1,2). Most of these programs can be used for the static and dynamic analysis of multistory frame and shear wall buildings. However, most of these programs do not give special recognition to the fact that building systems in themselves are a very special class of structures from an analytical point of view. The following are some of the characteristics that are inherent in the nature of a building analysis that a general-purpose analysis program may not recognize, thereby resulting in significant losses in man-hours, computer time, and possibly accuracy:

1. Most buildings are of simple geometry with horizontal beams and vertical columns. A simple rectangular grid can define such a geometry

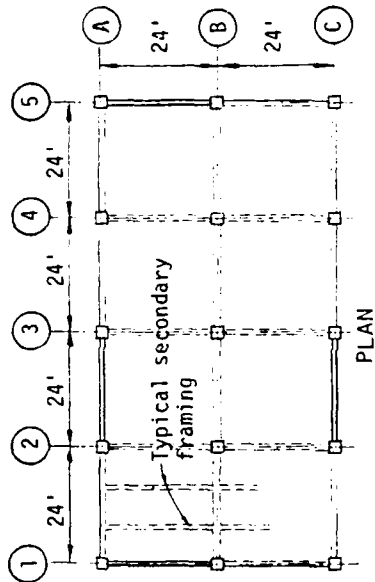
vertical columns. A simple rectangular grid can define such a geometry with minimal input. See Figure 1.

2. Many of the frames and shear walls are typical. Most general-purpose programs do not recognize this fact; therefore, the input may be large, and some internal calculations may be unnecessarily duplicated.
3. The in-plane stiffnesses of the floor systems of most buildings are very high. General-purpose programs do not necessarily recognize this, resulting in a set of equilibrium equations which may be very large, and thereby causing an increase in computation effort by a factor of 10 to 100. Also, numerical errors may be introduced since the in-plane floor stiffnesses are several orders of magnitude greater than the story-to-story stiffnesses of the structure. Since these two stiffnesses are added in a direct stiffness approach, double precision may be required in the solution.
4. The loading in building systems is of a restricted form. Loads, in general, are either vertically down (dead or live) or lateral (wind or seismic). The vertical loads are usually applied on the beams, and the lateral loads are generated at the floor levels.
5. In many buildings, the dimensions of the members are large and have a significant effect on the stiffness of the frame. Therefore, corrections need to be applied to the member stiffnesses. Most general-purpose programs work on center-line dimensions, and stiffness corrections are usually very tedious to implement.
6. In the dynamic analysis of buildings, the mass of the structure can be accurately lumped at the floor levels. Recognizing this fact

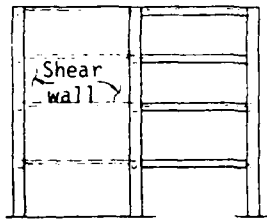
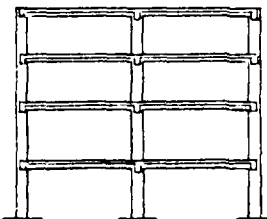
Horizontal rigid diaphragms
connecting the frames at
each level



ELEVATION - TYPICAL FRAME
ALONG LINES (1) AND (5)



ELEVATION - TYPICAL FRAME
ALONG LINES (2) (3) AND (4)



NOTE/

Structure has 4 typical
frames and 8 total frames

ELEVATION - TYPICAL FRAME ALONG LINES (A) AND (C)

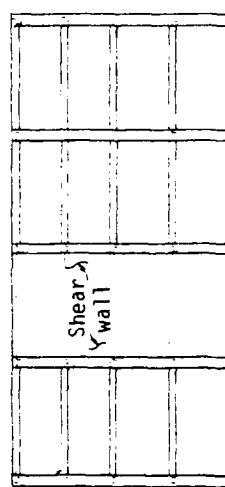


Figure 1. Typical frame and shear wall building

significantly reduces the size of the eigenvalue problem to be solved.

7. Various code loading requirements necessitate special options that allow convenient combinations of the vertical and lateral static and dynamic loadings. Also, the member forces need to be printed out at the support faces of the members. Such transformations are not automatic in general-purpose programs.
8. It is desirable to have a building analysis computer output printed in a special format; i.e., in terms of a particular frame, story, column, and beam. Also, special output such as story shears may be desirable.

In light of the above-mentioned and other reasons, the need for special-purpose programs for building analysis is apparent.

C. Special-Purpose Programs for Building Analysis

Various programs have been developed at the University of California at Berkeley for the linear analysis of multistory buildings in the past two decades (4,5,6). These programs have been used in the profession on many major structures in many different countries. One of the major reasons for the development of computer program TABS (1,2,3) was the direct "feedback" from the profession in the use of these programs.

The first of these programs, FRMSTC, is a static load analysis program for symmetrical buildings with parallel frames and shear walls. Lateral mode shapes and frequencies are also evaluated.

Program FRMDYN is the same as FRMSTC except that the load input is ground accelerations due to a specified earthquake. Time-dependent displacements and member forces are produced but are not combined with static loads.

Program LATERAL is an extension of FRMSTC to the static analysis of a system of frames and shear walls which are not parallel. Three degrees of freedom exist at each story level. This program does not have dynamic options.

The first version of TABS was released in 1972, with the intent of replacing the computer programs described above. CTABS80 is an enhanced version of the original version of TABS and is intended to supercede other enhanced versions such as XTABS and TABS77.

The computer program ETABS ⁽¹⁵⁾ was released in 1975. The program allows three-dimensional frame input in which common column compatibility is enforced. The input data are more complex than those of TABS, and use of this program is only recommended if common column compatibility is important.

For buildings with other complexities, such as discontinuous or flexible diaphragms, sloped diaphragm, nonrectangular framing systems, etc., a general-purpose program such as SAPIV ⁽¹²⁾ or EASE2 ⁽¹¹⁾ is still the most appropriate solution tool.

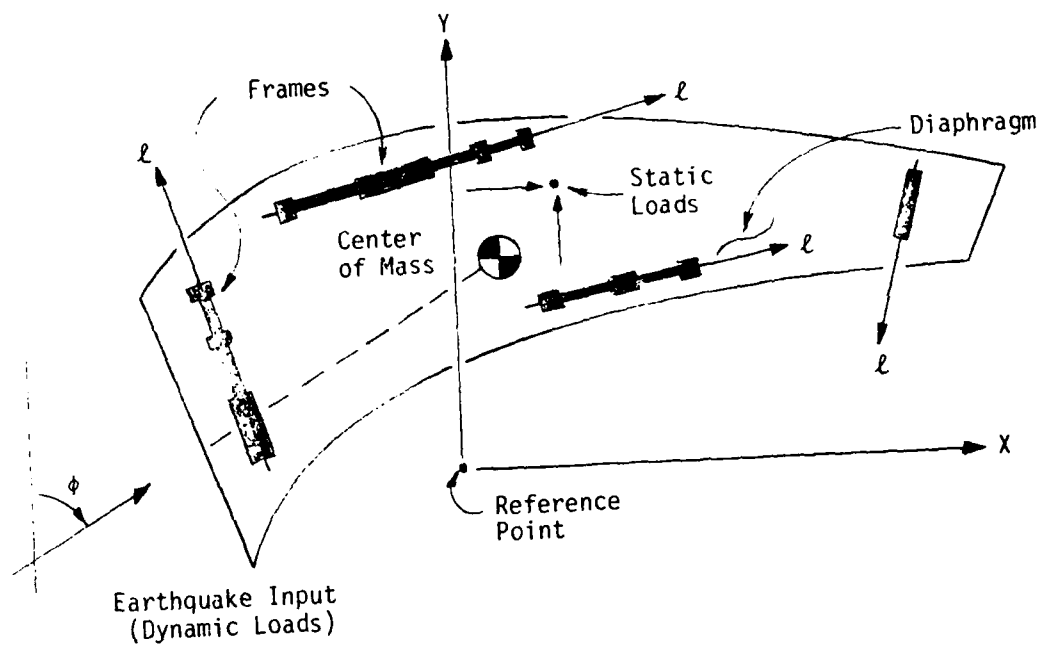
D. Disclaimer

Considerable time, effort, and expense have gone into the development and documentation of CTABS80, and the program has been thoroughly tested and used. In using the program, however, the user accepts and understands that no warranty is expressed or implied, either by the sponsors, the developers, or the distributors, as to the accuracy or the reliability of the program. The user must clearly understand the basic assumptions of the program and must verify his own results.

CHAPTER II: STRUCTURAL IDEALIZATION

An exact three-dimensional structural analysis is required for only a limited number of buildings. For the majority of buildings the following approximations can be made. These approximations greatly simplify the preparation of input data and significantly reduce the computational efforts associated with the analysis of the structure.

1. The structure is idealized as an assemblage of vertical planar "frames". A frame consists of m columns and $(m-1)$ beams. As long as shear and bending deformations are included in all members there is no need to distinguish between a shear wall/spandrel versus a beam/column system. See Figure 2.
2. The out-of-plane stiffnesses of all frames are assumed to be zero. Therefore each column at every floor has two degrees of freedom, a vertical displacement and a rotation. In addition, there is one lateral degree of freedom at every floor level of the frame.
3. Each floor is modeled as a horizontal diaphragm. This diaphragm is assumed to be infinitely stiff in-plane. The out-of-plane stiffness of this diaphragm is neglected. Bending stiffness of the floors may be included approximately in the modeling of the individual frames. It is apparent that axial deformation is not permitted in the beams. Floor levels must be the same for all frames. See Figure 2.
4. The floor diaphragm connects all the frames together at the corresponding level. The connection is only in a lateral sense. The frames otherwise are completely independent of each other. This also means that compatibility is not enforced with regard to



PLAN

Figure 2. Typical story level

displacements at columns which are common to more than one frame.

Thus axial deformations in common columns will not be the same. As for joint rotations, if the frames with common members are perpendicular in plan view, then the rotations are uncoupled. This assumption invalidates the program for use in the analysis of structures in which the tubular effect or common column compatibility is important.

5. Vertical loads are applied to each frame on a tributary area basis. The diaphragm will not transfer any vertical load from one frame to another. However, no frame can sidesway independently without engaging the other frames.
6. Lateral loads are applied as loads for the complete floor at each level. The loads are applied at specified locations on the floor diaphragms and get distributed to the various frames in accordance with their corresponding stiffnesses and locations.

A. The Frame Substructure

The elevation of a typical frame is shown in Figure 3. The frame is basically of rectangular geometry with vertical column center lines and horizontal floor levels as the basic reference lines for the description of the frame.

The frame is an assemblage of column, beam, panel, and bracing elements. Vertical loading is applied to the individual frames by means of loading patterns associated with each beam.

The column and beam elements have options for rigid offsets at each end to compensate for the effects of the finite dimensions of the members on

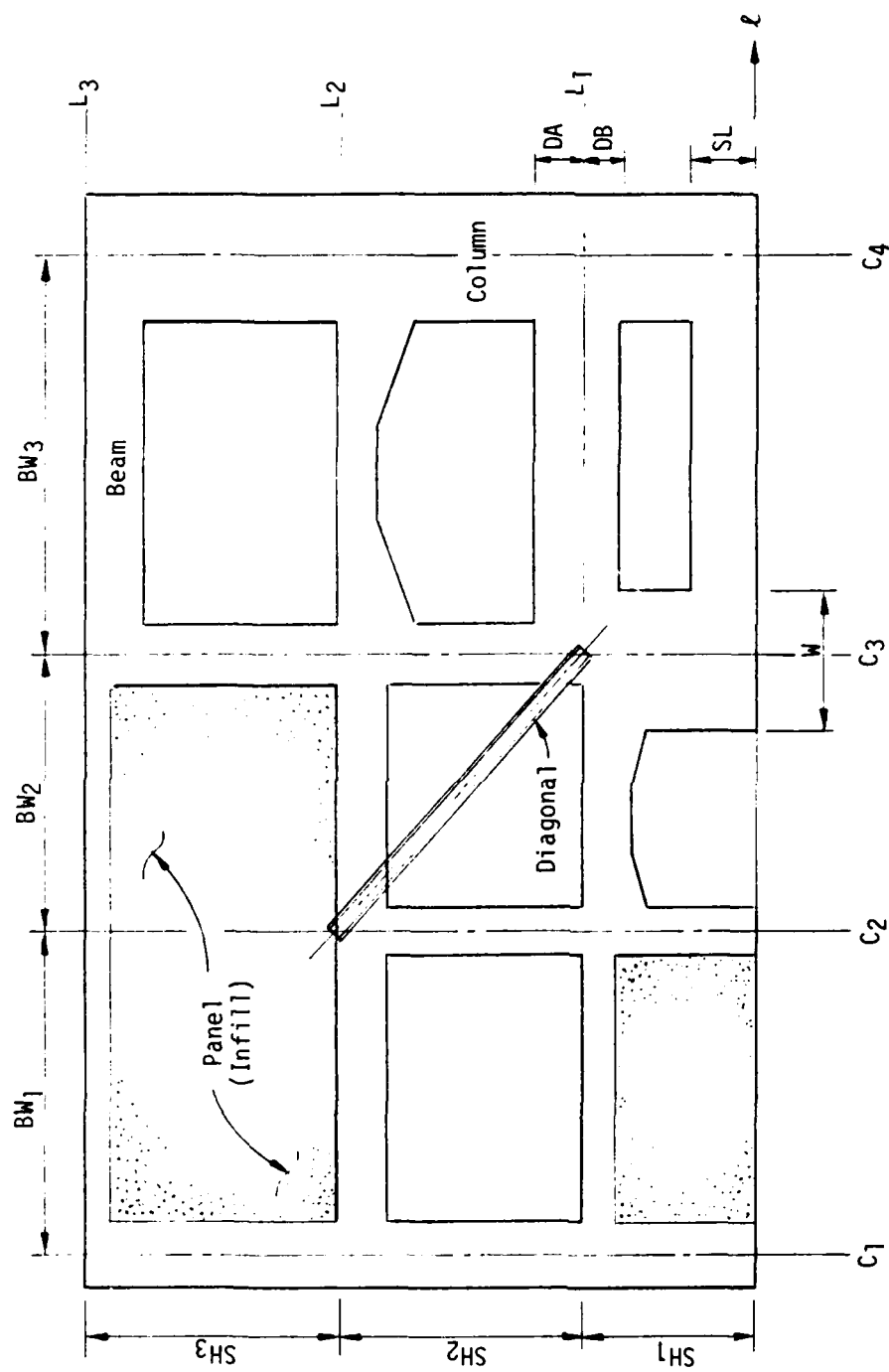


Figure 3. Elevation of typical frame

the stiffness of the system. The procedure used to set the lengths of these rigid offsets is presented later in this report.

(i). Individual Member Stiffnesses

The complete stiffness matrix of each frame is assembled by the direct stiffness technique. This involves calculating the local stiffness matrix, $\underline{k}$, for each member along with a transformation matrix, $\underline{a}$, which transforms the local displacements and forces, $\underline{\phi}$, $\underline{S}$, to global displacements and forces, $\underline{r}$, $\underline{R}$

$$\begin{array}{ll} \text{or: } \underline{\phi} = \underline{a} \underline{r} & \text{also: } \underline{S} = \underline{k} \underline{\phi} \\ \underline{S} = \underline{a} \underline{R} & \text{and: } \underline{R} = \underline{K} \underline{r} \end{array}$$

where $\underline{K}$ is the stiffness matrix in global coordinates.

Substituting $\underline{\phi} = \underline{a} \underline{r}$ and $\underline{S} = \underline{a} \underline{R}$ into $\underline{S} = \underline{k} \underline{\phi}$ we get:

$$\underline{a} \underline{R} = \underline{k} \underline{a} \underline{r}$$

Premultiply both sides by $\underline{a}^T$ and recognizing $\underline{a}^T \underline{a} = \underline{I}$ we get:

$$\underline{a}^T \underline{a} \underline{R} = \underline{a}^T \underline{k} \underline{a} \underline{r}$$

$$\underline{R} = \underline{a}^T \underline{k} \underline{a} \underline{r}$$

$$\text{As } \underline{R} = \underline{k} \underline{r} \text{ we get } \underline{K} = \underline{a}^T \underline{k} \underline{a}$$

Thus knowing the local stiffness matrix $\underline{k}$ and the coordinate transformation matrix $\underline{a}$ the global stiffness matrix may be evaluated.

The $\underline{a}$ and $\underline{k}$ matrices for the column, beam, panel, and brace elements are presented in Figures 4, 5, 6, and 7, respectively.

$$\begin{Bmatrix} M_i \\ M_j \\ S \end{Bmatrix} = \begin{bmatrix} S_a & S_b & 0 \\ S_b & S_a & 0 \\ 0 & 0 & S_c \end{bmatrix} \begin{Bmatrix} \phi_i \\ \phi_j \\ \delta \end{Bmatrix}$$

$$S_c = k_c \quad \phi_c$$

FORCE/DEFORMATION TRANSFORMATION

$$\begin{Bmatrix} \phi_i \\ \phi_j \\ \delta \end{Bmatrix} = \begin{bmatrix} 1 + \frac{b}{h} & \frac{1}{h} & \frac{a}{h} & -\frac{1}{h} & 0 & 0 \\ \frac{b}{h} & \frac{1}{h} & 1 + \frac{a}{h} & -\frac{1}{h} & 0 & 0 \\ 0 & 0 & 0 & 0 & 1 & -1 \end{bmatrix} \begin{Bmatrix} \theta_B \\ u_B \\ \theta_T \\ u_T \\ v_B \\ v_T \end{Bmatrix}$$

$$\phi_c = \begin{bmatrix} a_c & r_c \end{bmatrix}$$

DEFORMATION/DISPLACEMENT TRANSFORMATION

$$k_c = \begin{bmatrix} a_c^T & k_c & a_c \end{bmatrix}$$

COLUMN STIFFNESS MATRIX (6x6)

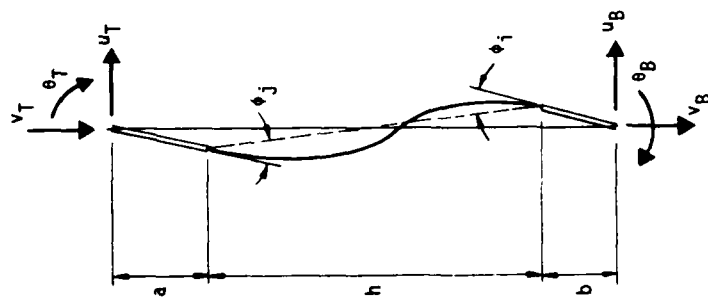


Figure 4

$$\begin{Bmatrix} M_i \\ M_j \end{Bmatrix} = \begin{bmatrix} s_a & s_b \\ s_b & s_a \end{bmatrix} \begin{Bmatrix} \phi_i \\ \phi_j \end{Bmatrix}$$

$$s_b = k_b \quad \phi_b$$

FORCE/DEFORMATION TRANSFORMATION

$$\begin{Bmatrix} \phi_i \\ \phi_j \end{Bmatrix} = \begin{bmatrix} 1 + \frac{b}{L} & \frac{1}{L} & \frac{a}{L} & -\frac{1}{L} \\ \frac{b}{L} & \frac{1}{L} & 1 + \frac{a}{L} & -\frac{1}{L} \end{bmatrix} \begin{Bmatrix} \theta_L \\ v_L \\ \theta_R \\ v_R \end{Bmatrix}$$

$$\phi_b = \begin{bmatrix} a_b & r_b \end{bmatrix}$$

DEFORMATION/DISPLACEMENT TRANSFORMATION

$$k_b = \begin{bmatrix} a_b^T & k_b & a_b \end{bmatrix}$$

BEAM STIFFNESS MATRIX (4x4)

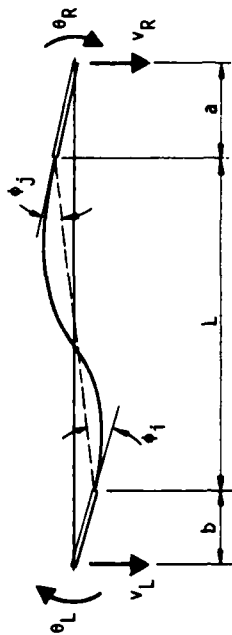


Figure 5

$$\begin{Bmatrix} M_i \\ M_j \\ S \end{Bmatrix} = \begin{bmatrix} S_a & S_b & 0 \\ S_b & S_a & 0 \\ 0 & 0 & S_c \end{bmatrix} \begin{Bmatrix} \phi_i \\ \phi_j \\ \delta \end{Bmatrix}$$

$$S_c = k_c \phi_c$$

FORCE/DEFORMATION TRANSFORMATION

$$\begin{Bmatrix} \phi_i \\ \phi_j \\ \delta \end{Bmatrix} = \begin{bmatrix} \frac{1}{h} & -\frac{1}{h} & -\frac{1}{L} & \frac{1}{L} & 0 & 0 \\ \frac{1}{h} & \frac{1}{h} & -\frac{1}{h} & 0 & -\frac{1}{L} & \frac{1}{L} \\ 0 & 0 & \frac{1}{2} & \frac{1}{2} & -\frac{1}{2} & -\frac{1}{2} \end{bmatrix} \begin{Bmatrix} u_B \\ u_T \\ v_{LB} \\ v_{RB} \\ v_{LT} \\ v_{RT} \end{Bmatrix}$$

$$f_p = \begin{bmatrix} a_p \\ -p \\ r_p \end{bmatrix}$$

DEFORMATION/DISPLACEMENT TRANSFORMATION

$$k_p = \begin{bmatrix} a_p^T & k_p & a_p \end{bmatrix}$$

PANEL STIFFNESS MATRIX
(6x6)

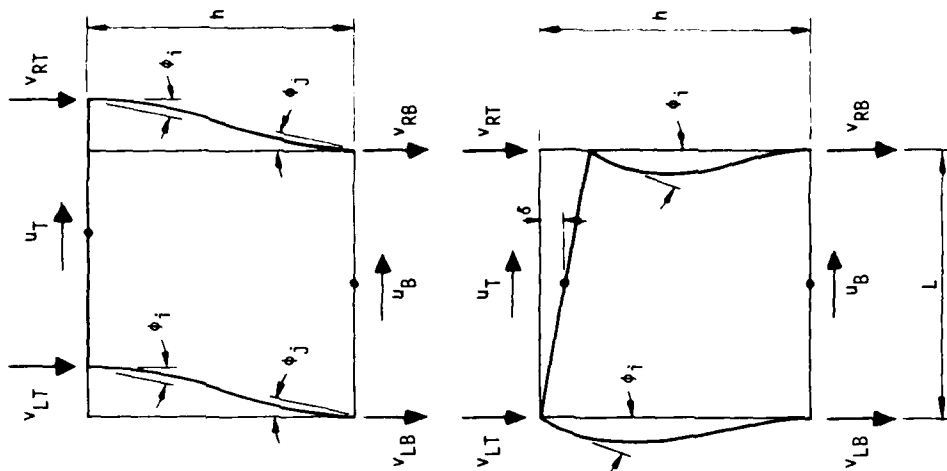


Figure 6

$$\begin{Bmatrix} M_i \\ M_j \\ S \end{Bmatrix} = \begin{bmatrix} s_a & s_b & 0 \\ s_b & s_a & 0 \\ 0 & 0 & s_c \end{bmatrix} \begin{Bmatrix} \phi_i \\ \phi_j \\ \delta \end{Bmatrix}$$

$$s_D = k_D \quad \phi_D$$

FORCE/DEFORMATION TRANSFORMATION

$$\begin{Bmatrix} \phi_i \\ \phi_j \\ \delta \end{Bmatrix} = \begin{bmatrix} 1 & \frac{h}{DD} & 0 & -\frac{h}{DD} & \frac{L}{DD} & -\frac{L}{DD} \\ 0 & \frac{h}{DD} & 1 & -\frac{h}{DD} & \frac{L}{DD} & -\frac{L}{DD} \\ 0 & -\frac{L}{D} & 0 & \frac{L}{D} & -\frac{h}{D} & \frac{h}{D} \end{bmatrix} \begin{Bmatrix} \theta_B \\ u_B \\ \theta_T \\ u_T \\ v_B \\ v_T \end{Bmatrix}$$

$$\phi_D = \phi_D \quad r_D$$

DEFORMATION/DISPLACEMENT MATRIX

$$k_D = \begin{bmatrix} a_D^T & k_D & a_D \end{bmatrix}$$

DIAGONAL (BRACE) STIFFNESS MATRIX

(6x6)

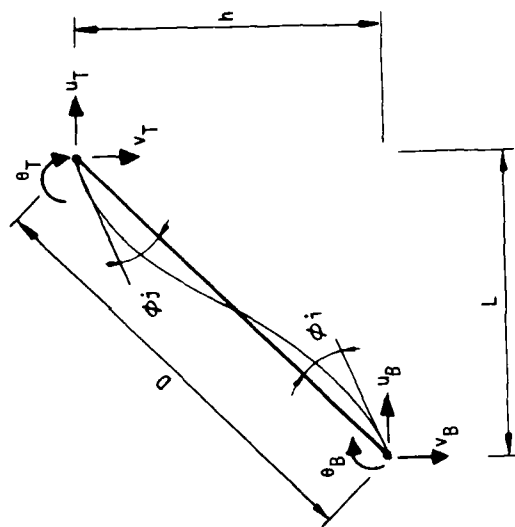


Figure 7

The column element formulation accounts for bending, axial and shear deformations. The basic stiffness matrix for such an element is shown in Figure 8. The column ends have options for rigid offsets. Figure 4 shows the six degrees of freedom associated with the column element and the deformation displacement transformation matrix linking the frame joint displacements to the column end deformations.

The beam element formulation is similar to that of the column except that the axial force component is dropped leaving a stiffness and transformation matrix as shown in Figure 5.

The panel element formulation is basically the same as that of the column except that each rotational degree of freedom is transformed into the two vertical displacements of the column lines bounding the panel element at each corresponding level. The axial degrees of freedom of the panel are also transformed as being an average of the vertical degrees of freedom of the two column lines bounding the panel element at each corresponding level. The degrees of freedom of the frame associated with the panel are therefore all translational. No rigid offsets are used. see Figure 6.

The diagonal element formulation is exactly the same as that of the column except that the brace is inclined and no rigid offsets are used. See Figure 7.

$$\begin{Bmatrix} M_i \\ M_j \\ S \end{Bmatrix} = \begin{bmatrix} S_a & S_b & 0 \\ S_b & S_a & 0 \\ 0 & 0 & S_c \end{bmatrix} \begin{Bmatrix} \phi_i \\ \phi_j \\ \delta \end{Bmatrix}$$

FORCE/DEFORMATION TRANSFORMATION

Where: $S_a = \frac{2EI}{L} \left(\frac{2 + \beta}{1 + 2\beta} \right)$

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

$$S_c = \frac{AE}{L}$$

$$\beta = \frac{6EI}{L^2 \bar{A} G}$$

A = axial area

$\bar{A}$ = effective shear area

I = moment of inertia

E = elastic modulus

G = shear modulus

L = length

FLEXURAL MEMBER WITH AXIAL DEFORMATIONS

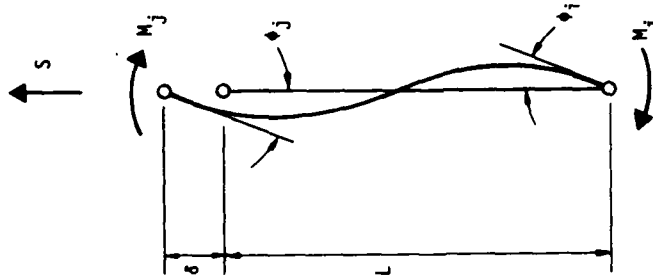


Figure 8

The complete stiffness matrix for each frame has two degrees of freedom for each beam-column intersection and one lateral degree of freedom per story.

(ii). Lateral Frame Stiffness

With the frame degrees of freedom appropriately ordered, the frame equilibrium equations have the form shown in Figure 9 . Where N is the number of stories in the frame, r_n is the vector of joint displacements (that is vertical displacement and rotation) at story level n and r_L is the vector of lateral story displacements. Lateral loads are applied to the complete structure and are considered when the lateral stiffness matrix for the complete building is assembled. Gaussian elimination is performed on the full system up to and including the equations:

$$R_N = C_{N-1} r_{N-1} + K_N r_N + E_N r_L$$

The last N equations (r_L is a vector of order N) may now be written as:

$$R_L = K_L r_L$$

The vector R_L is the lateral load submatrix of the frame and is modified by the elimination process due to vertical loading on the frame. These terms represent the sidesway effects under vertical loading. The matrix K_L clearly represents the frame lateral stiffness matrix; i.e., the stiffness matrix of the frame in terms of only the lateral story displacements.

Within the computer program the following approach is adopted in order to reduce storage requirements. The assembly and reduction process is

								SIZE	
R_1	K_1	C_1		E_1	r_1			2NC	
R_2	C_1^T	K_2	C_2	E_2	r_2			2NC	
R_3		C_2^T	K_3	C_3	r_3			2NC	
$\vdots$			$\vdots$	$\vdots$	$\vdots$				
$\vdots$			$\vdots$	$\vdots$	$\vdots$				
R_n			K_n	C_n	E_n	r_n		2NC	
R_{n+1}			C_n^T	K_{n+1}	E_{n+1}	r_{n+1}		2NC	
$\vdots$					$\vdots$	$\vdots$			
$\vdots$					$\vdots$	$\vdots$			
$\vdots$					$\vdots$	$\vdots$			
R_{N-1}				K_{N-1}	C_{N-1}	E_{N-1}	r_{N-1}	2NC	
R_N				C_{N-1}^T	K_N	E_N	r_N	2NC	
R_L	E_1^T	E_2^T	$\vdots$	E_n^T	E_{n+1}^T	$\vdots$	E_{N-1}^T	E_N^T	K_L
							r_L		NS+1

The equations in core at any one time are blocked out above.

NC = No. of Column Lines NS = No. of Stories

Figure 9. Complete equation system of frame substructure

carried out systematically story by story from the top of the structure such that at any level, n , we consider the system shown below:

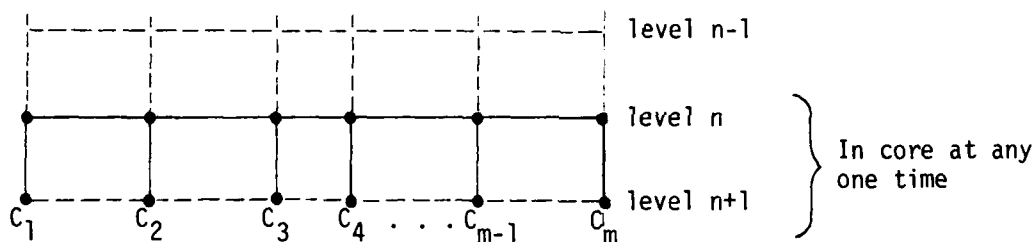
$$\begin{Bmatrix} \underline{R}'_n \\ \underline{R}'_{n+1} \\ \underline{R}'_L \end{Bmatrix} = \begin{bmatrix} \underline{K}'_n & \underline{C}'_n & \underline{E}'_n \\ \underline{C}'_n{}^T & \underline{K}'_{n+1} & \underline{E}'_{n+1} \\ \underline{E}'_n{}^T & \underline{E}'_{n+1}{}^T & \underline{K}'_L \end{bmatrix} \begin{Bmatrix} \underline{r}_n \\ \underline{r}_{n+1} \\ \underline{r}_L \end{Bmatrix}$$

where the prime indicates that the submatrices may have been modified by previous elimination.

At each level the following steps are performed:

- a. Add in the individual member stiffnesses for level n .

These are shown below:



- b. Perform the elimination on the equations of the uppermost partition in the equations above
- c. Save these reduced equations for subsequent back-substitution.
- d. Rearrange the submatrices in the equation above appropriately in order to proceed to the next level. This rearrangement is as follows:

$$\begin{Bmatrix} \underline{R}'_{n+1} \\ \underline{0} \\ \underline{R}'_L \end{Bmatrix} = \begin{bmatrix} K'_{n+1} & \underline{0} & E'_{n+1} \\ \underline{0} & \underline{0} & \underline{0} \\ E'^T_{n+1} & \underline{0} & K'_L \end{bmatrix} \begin{Bmatrix} \underline{r}_{n+1} \\ \underline{r}_{n+2} \\ \underline{r}_L \end{Bmatrix}$$

- e. Repeat the above steps for the next level. Thus after the elimination is completed for joint displacements at all story levels, we are left with the lateral stiffness matrix for the frame.

(iii). Rigid Joint Offset For Beams and Columns

The deformations within the joint, an area bounded by the finite dimensions of any beam and column intersection (shown shaded in Figure 10) are neglected. In other words, this area is assumed to be an infinitely rigid rectangular diaphragm.

This is achieved by providing rigid offsets at the ends of the beams equal in length to one half of the widths of the column below at each corresponding end. Rigid offsets are also provided at each end of the columns equal to the depth of the larger of the beams on either side of the column at the corresponding level.

It has been found that, in general, a reduction in the lengths of the rigid offsets to compensate for some deformation that may exist in the joint is justifiable and gives better results, especially in cases where the member dimensions are substantial. See Reference 9.

Reduction of the rigid link dimension has been coupled to the size of the member. In other words, the rigid link is calculated as described above and then is reduced by 25% of the dimension of the member, at each end.

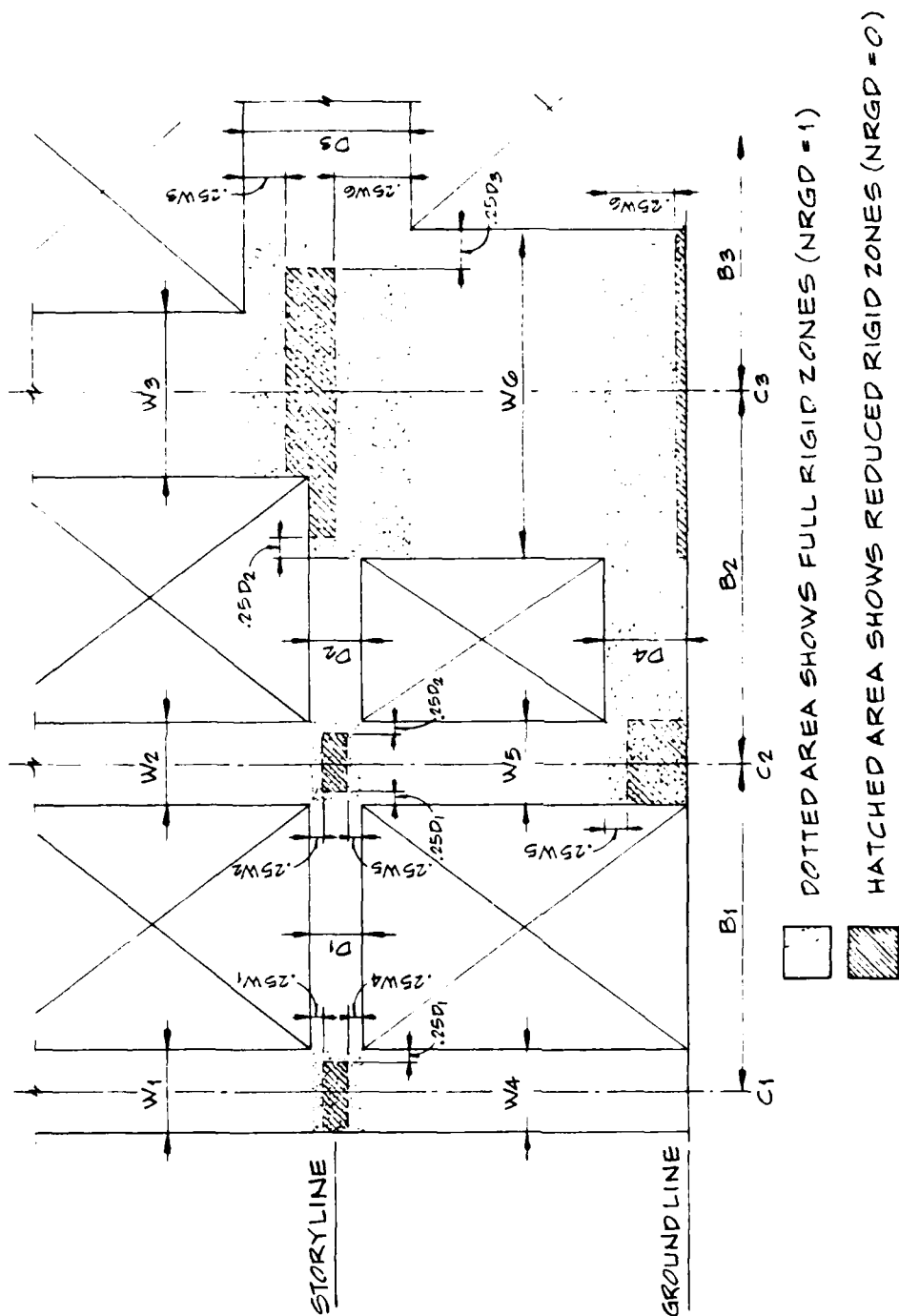


Figure 10. Illustration of rigid zone reduction methodology

Thus the beam rigid links are reduced by 25% of the beam depth and the column rigid links are reduced by 25% of the column width at each end. The reduction cannot, of course, result in a negative rigid link length. This reduction procedure is optional. If no column widths or beam lengths are input the rigid link lengths degenerate to zero and the analysis is carried out on the frame grid line basis.

B. The Complete Structure

In order to combine the frame lateral stiffness matrices into a complete structure lateral stiffness matrix, each of the frame stiffnesses must be transformed to a common displacement coordinate system (which will be referred to as the global system). The global system chosen is two translations and one rotation per story. The origin of these global displacement coordinates at each story level is taken at the center of mass of that story segment. This position may vary from story to story. Such a formulation will degenerate the mass matrix to a diagonal form, thus simplifying the eigen-value problem in the dynamics.

The first step is to develop the transformation between the frame lateral displacements and the global displacements. With reference to Figure 11, the transformation at any level, n , is as follows:

$$r_{Ln} = \begin{bmatrix} \cos \alpha & \sin \alpha & -d_n \end{bmatrix} \begin{Bmatrix} r_{xn} \\ r_{yn} \\ r_{\theta n} \end{Bmatrix}$$

$$\text{or:} \quad r_{Ln} = a_n r_n$$

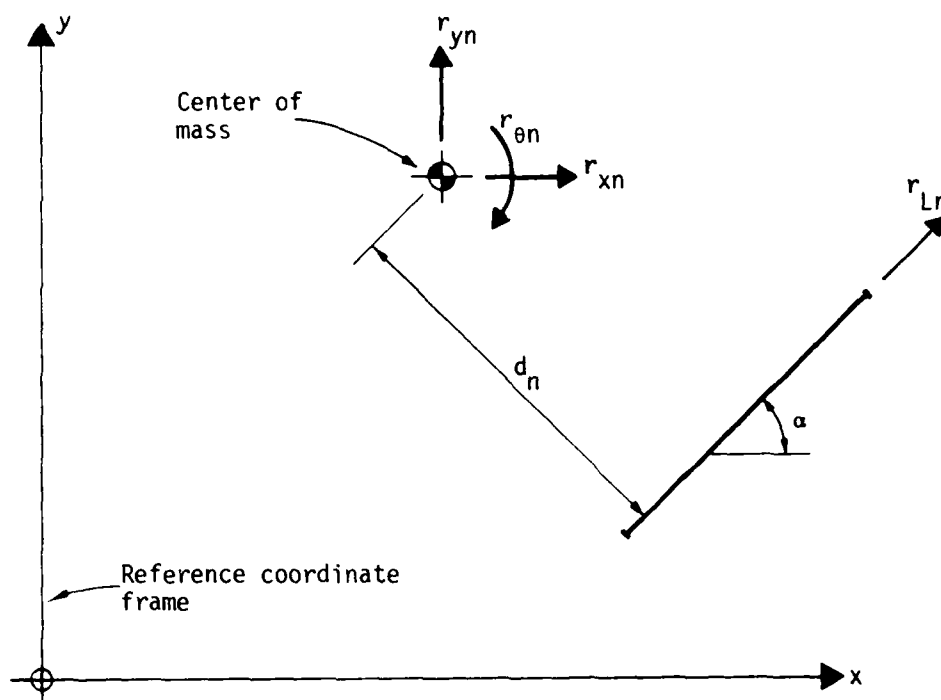


Figure 11. Structural global lateral displacements and frame local lateral displacements

Assembling the transformations for all floors, we obtain the complete transformation between frame lateral displacements and global displacement as follows:

$$\begin{Bmatrix} r_{L1} \\ r_{L2} \\ \vdots \\ r_{Ln} \\ \vdots \\ r_{LN} \end{Bmatrix} = \begin{bmatrix} a_1 & & & & \\ & a_2 & & & \\ & & \ddots & & \\ & & & a_n & \\ & & & & \ddots \\ & & & & & a_N \end{bmatrix} \begin{Bmatrix} r_1 \\ r_2 \\ \vdots \\ r_n \\ \vdots \\ r_N \end{Bmatrix}$$

or:

$$r_{Li} = A_i r$$

r is the complete vector of global displacements. The frame lateral stiffness is transformed to the global system and becomes:

$$K_i = A_i^T K_{Li} A_i$$

where the subscript i denotes the i th frame.

The structure lateral stiffness is assembled by the addition of components from all frames: i.e.,

$$K = \sum_i K_i$$

The frame lateral load vector from sidesway effects must also be transformed to the global system. This transformation is shown by:

$$R_i = A_i^T R'_{Li}$$

The global load vector is formed by the summation of frame sway effects and the addition of externally applied lateral loads $\underline{F}$, i.e.:

$$\underline{R} = \sum_i \underline{R}_i + \underline{F}$$

The global forces $\underline{F}$ are specified; however, they are also given by:

$$\underline{F} = \sum A_i^T P_{Li}$$

Expanding the tri-matrix product:

$$\underline{K}_i = \underline{A}_i^T \underline{K}_{Li} \underline{A}_i$$

we get:

$$\begin{bmatrix} K_{11} & K_{12} & \cdot & \cdot & \cdot & \cdot \\ K_{21} & K_{22} & \cdot & \cdot & \cdot & \cdot \\ \cdot & \cdot & \cdot & \cdot & \cdot & \cdot \\ \cdot & \cdot & K_{ij} & \cdot & \cdot & \cdot \\ \cdot & \cdot & \cdot & \cdot & \cdot & \cdot \\ \cdot & \cdot & \cdot & \cdot & \cdot & K_{NN} \end{bmatrix} = \begin{bmatrix} a_1^T \\ a_2^T \\ \cdot \\ a_i^T \\ \cdot \\ a_N^T \end{bmatrix} \begin{bmatrix} k_{11} & k_{12} & \cdot & \cdot \\ \cdot & \cdot & \cdot & \cdot \\ \cdot & \cdot & \cdot & \cdot \\ \cdot & \cdot & \cdot & \cdot \\ \cdot & \cdot & \cdot & \cdot \\ \cdot & \cdot & \cdot & k_{NN} \end{bmatrix} \begin{bmatrix} a_1 \\ a_2 \\ \cdot \\ a_j \\ \cdot \\ a_N \end{bmatrix}$$

It is worth noting that a typical 3 x 3 submatrix $\underline{K}_{ij}$ within $\underline{K}_i$ has the form $a_i^T \underline{k}_{ij} a_j$. Obviously this product may be formed independently for each term in and added directly into $\underline{K}$. Hence the global equilibrium equations are formed.

$$\underline{R} = \underline{K} \underline{r}$$

It may be noted that the global stiffness $\underline{K}$ is a full matrix, but it is of course relatively small compared to the total number of degrees of freedom associated with all the frames in the structure.

CHAPTER III: STATIC ANALYSIS

The static analysis equations:

$$R = k r$$

are solved directly by Gaussian elimination giving a vector of global lateral displacements, r . Next, for each frame, the lateral displacements, r_{Li} are computed using:

$$r_{Li} = A_i r$$

To complete the solution for each frame, the following system is considered.

$$R'_n = \begin{bmatrix} K'_n & C'_n & E'_n \end{bmatrix} \begin{bmatrix} r_n \\ r_{n+1} \\ r_L \end{bmatrix}$$

Note that these are the equations which were reduced, then saved at each level, n , of the frame. That is, K'_n was triangularized. At any stage, n , r_{n+1} and r_L are known and so r_n is computed by back substitution. To start this sequence, we simply note that for $n = N$ (the number of stories in the structure) r_{N+1} represents the displacements at the foundation which are zero since columns are assumed rigidly connected to the foundation. Thus the frame joint displacements are computed successively story by story and individual member forces may be computed at the same time from the force/deformation transformations previously presented.

A. Vertical Loads Analysis

The vertical loads are applied on each individual frame as beam span loads. Four independent vertical loading conditions are possible. The self weight of the frames can be automatically calculated by the program and added to the load vector of the first load condition. Typically the first load condition is used for the dead load analysis of the structure; the second load condition is used for the live load analysis of the structure. The third and fourth load conditions may be used for skip live loading or left unused.

B. Lateral Load Analysis

The lateral static loads are applied as forces acting at a particular point on each floor level. Two independent lateral loading conditions are possible. The lateral loads may be due to wind or earthquake. The wind loads have to be calculated and input by the user, based upon the wind pressure and the exposed tributary area of the building at each level of the structure. The seismic static equivalent loads may be automatically calculated by the program, based upon the requirement of Reference 14. The modal participation factors calculated by the program are used to determine the predominant directions of the modes and the time periods of the predominant modes are used in calculating the seismic loads in the corresponding directions.

The program has options to calculate the dynamic properties, such as the mass and mass moment of inertia of each floor level based upon simplified user input.

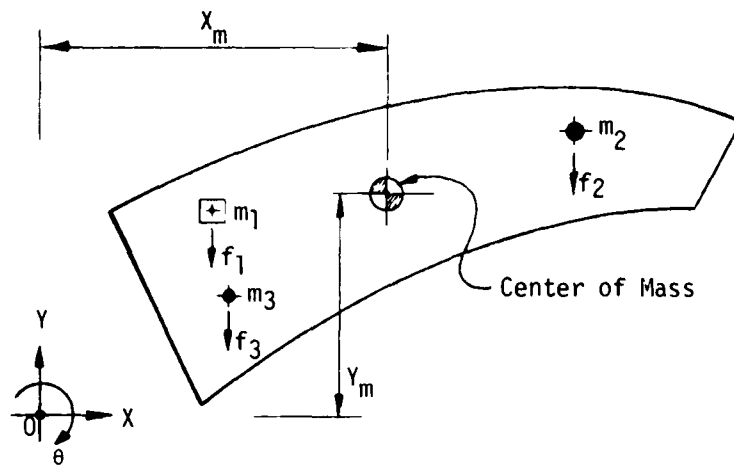
CHAPTER IV: DYNAMIC ANALYSIS

The exact formulation of the dynamic response of a structure involves an infinite number of degrees of freedom. For most structures, however, the response may be adequately captured by a limited number of discrete points (or joints) within the system. In the buildings considered here, the response may be described by the lateral motions of each floor level, as previously described for the formation of the lateral structure stiffness matrix. The center of mass is used as the master constraint location at each level in the generation of the lateral stiffness matrix. The tributary mass of each story level is lumped at the center of mass of the level along with the mass moment of inertia of the floor about a vertical axis through the center of mass to compensate for the rotational aspects of the lumping process. The resulting mass matrix is of diagonal form. With this lumped parameter idealization, equilibrium of the structure is described by a set of ordinary second order differential equations.

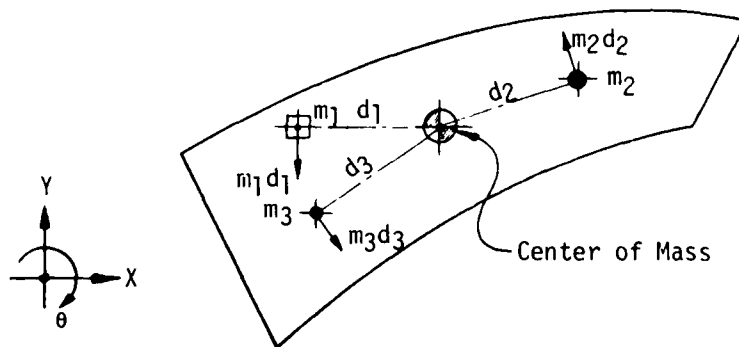
A. Mass Approximation, Mass and Mass Moment of Inertia

In the diaphragm shown in Figure 12, there are various lumped masses ($m_1, m_2, m_3 \dots$ etc.) and other distributed masses associated with the diaphragm level.

When the diaphragm is subjected to a unit translational acceleration in the Y-direction, inertia forces opposing the direction of the acceleration will be generated, i.e. $f_1 = m_1 \times 1$, $f_2 = m_2 \times 1$, $f_3 = m_3 \times 1 \dots$. The resultant of all these forces and line of action of the resultant can be determined. The magnitude of the resultant is found to be $= f_1 +$



(a) DEFINITION OF MASS AND CENTER OF MASS



(b) DEFINITION OF MASS MOMENT OF INERTIA

Figure 12

$f_2 + f_3 \dots = m_1 + m_2 + m_3 \dots = \text{total mass associated with the diaphragm.}$ The resultant is parallel to the Y-direction and passes through a point at a distance X_m from 0.

Similarly, a unit translation in the X-direction will give a resultant of the same magnitude but parallel to the X-direction and passing through a point a distance Y_m from 0.

The coordinates X_m, Y_m define the location of a point known as the center of mass.

Redefining the term, Mass: "The mass of a diaphragm may be defined as the force generated when the center of mass of the diaphragm undergoes a unit translational acceleration. This force acts at the center of mass, resulting in no associated moment."

$$\text{Mass} = \sum_{i=1}^n m_i$$

Similarly, defining the term, Mass Moment of Inertia (or Rotational Mass): "The mass moment of inertia of a diaphragm may be defined as the moment generated when the center of mass of the diaphragm undergoes a unit rotational acceleration about a vertical axis. No resultant translational force is associated with the couple."

The radial distances from the center of mass of the lumped masses $m_1, m_2, m_3 \dots$ are $d_1, d_2, d_3 \dots$ respectively, as shown in Figure 12.

Due to a unit rotational acceleration of the center of mass about a vertical axis, $m_1, m_2, m_3 \dots$ will have translational accelerations of $d_1 \times 1, d_2 \times 1, d_3 \times 1 \dots$. Thereby giving corresponding inertia forces of

$m_1 d_1, m_2 d_2, m_3 d_3 \dots$ The moments of these forces about a vertical axis through the center of mass are $m_1 d_1^2, m_2 d_2^2, m_3 d_3^2 \dots$

$$\therefore MMI = m_1 d_1^2 + m_2 d_2^2 + m_3 d_3^2 \dots$$

$$= \sum_{i=1}^n m_i d_i^2$$

= Polar Moment of Inertia of all Masses, about a vertical axis through the center of mass

B. Dynamic Equilibrium Equations

The equilibrium equations for a structure, including dynamic effects, may be written in the following form:

$$\underline{M} \ddot{\underline{r}}_a + \underline{C} \dot{\underline{r}} + \underline{K} \underline{r} = \underline{P}(t) \dots (a)$$

where: $\underline{M}$ = mass matrix

$\underline{C}$ = damping matrix

$\underline{K}$ = stiffness matrix

$\underline{P}(t)$ = applied load vector, which may be time dependent

$\underline{r}$ = displacement vector of deformation relative to support motion

$\ddot{\underline{r}}_a$ = absolute acceleration vector

$\underline{r}$ and $\underline{r}_a$ are related in the following fashion:

$$\underline{r}_a = \underline{v}_g + \underline{r}$$

where $\underline{v}_g$ is the vector of pseudo-static displacements due to support movement. Also:

$$\ddot{\underline{r}}_a = \ddot{\underline{v}}_g + \ddot{\underline{r}}$$

These vectors have the following form for a typical floor, of a building shown in Figure 13.

$$\begin{Bmatrix} r_{xa} \\ r_{ya} \\ r_{\theta a} \end{Bmatrix}_n = \begin{Bmatrix} v_{gx} \\ v_{gy} \\ v_{g\theta} \end{Bmatrix} + \begin{Bmatrix} r_{xn} \\ r_{yn} \\ r_{\theta n} \end{Bmatrix} = \begin{Bmatrix} \sin \beta \\ \cos \beta \\ 0 \end{Bmatrix} v_g + \begin{Bmatrix} r_{xn} \\ r_{yn} \\ r_{\theta n} \end{Bmatrix}$$

and:

$$\begin{Bmatrix} \ddot{r}_{xa} \\ \ddot{r}_{ya} \\ \ddot{r}_{\theta a} \end{Bmatrix}_n = \begin{Bmatrix} \sin \beta \\ \cos \beta \\ 0 \end{Bmatrix} \ddot{v}_g + \begin{Bmatrix} \ddot{r}_{xn} \\ \ddot{r}_{yn} \\ \ddot{r}_{\theta n} \end{Bmatrix}$$

i.e.:

$$\underline{r}_{na} = \underline{b} \underline{v}_g + \underline{r}_n$$

Or, for all floors:

$$\underline{r}_a = \underline{B} \underline{v}_g + \underline{r}$$

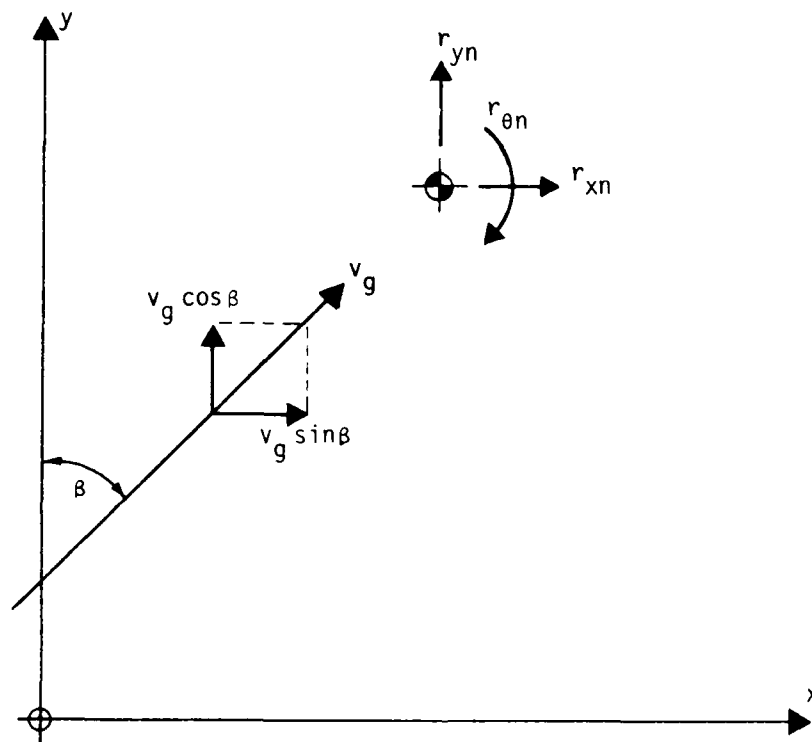


Figure 13. Ground and structural displacements

where:

$$\underline{B} = \begin{Bmatrix} b_1 \\ b_2 \\ b_3 \\ \vdots \\ b_N \end{Bmatrix} ; \quad b_1 = b_2 \text{ etc.}$$

In the case of seismic analysis, there are no externally applied loads; i.e., $P(t) = 0$. Then equation (a) may be written as:

$$\underline{M} (\ddot{\underline{r}} + \underline{B} \ddot{v}_g) + \underline{C} \dot{\underline{r}} + \underline{K} \underline{r} = \underline{0}$$

or:

$$\underline{M} \ddot{\underline{r}} + \underline{C} \dot{\underline{r}} + \underline{K} \underline{r} = - \underline{M} \underline{B} \ddot{v}_g \quad \dots \dots (b)$$

This coupled set of equations may be solved simultaneously with an appropriate numerical technique. Another approach, which will be used here, is to find a transformation which uncouples the equations so that they may be solved independently. This transformation, of course is via the eigen-vectors or mode shapes of the system.

C. Mode Shapes and Frequencies

The vibration mode shapes represent the solution of the undamped free vibration problem given by:

$$\underline{M} \ddot{\underline{r}} + \underline{K} \underline{r} = \underline{0}$$

The eigen-value problem to be solved is written as:

$$\underline{K} \underline{\phi} = \underline{\omega}^2 \underline{M} \underline{\phi}$$

where: $\underline{\phi}$ = mode shapes

$\underline{\omega}$ = frequencies

The mode shapes are normalized such that:

$$\underline{\phi}^T \underline{M} \underline{\phi} = \underline{I}$$

then also:

$$\underline{\phi}^T \underline{K} \underline{\phi} = \underline{\omega}^2$$

Also, it is assumed that the damping matrix $\underline{C}$ is of a form that is uncoupled by the mode shapes; specifically it is assumed that:

$$\underline{\phi}^T \underline{C} \underline{\phi} = [2\lambda_m \quad \omega_m]$$

so that λ_m represents the damping of the m th mode.

The actual displacements, $\underline{r}$, are now expressed as a linear combination of the mode shapes.

$$\underline{r} = [\underline{\phi}_1 \quad \underline{\phi}_2 \quad \underline{\phi}_3 \quad \cdots \quad \underline{\phi}_N] \begin{bmatrix} z_1(t) \\ z_2(t) \\ \cdot \\ \cdot \\ z_N(t) \end{bmatrix} \quad \dots (c)$$

i.e.: $\underline{r} = \underline{\phi} \underline{z}$

also $\dot{\underline{r}} = \underline{\phi} \dot{\underline{z}}$

and: $\ddot{\underline{r}} = \underline{\phi} \ddot{\underline{Z}}$

where $Z_m(t)$ represents the response of the m th mode.

D. Time History Analysis

Using equations (b), equation (c) may be rewritten as:

$$\underline{M} \underline{\phi} \ddot{\underline{Z}} + \underline{C} \underline{\phi} \dot{\underline{Z}} + \underline{K} \underline{\phi} \underline{Z} = - \underline{M} \underline{B} \ddot{v}_g$$

Premultiplication by $\underline{\phi}^T$ yields the uncoupled set of second order equations:

$$\underline{M}^* \ddot{\underline{Z}} + \underline{C}^* \dot{\underline{Z}} + \underline{K}^* \underline{Z} = \underline{P}^* \ddot{v}_g$$

where:

$$\underline{M}^* = \underline{\phi}^T \underline{M} \underline{\phi} = \underline{I}$$

$$\underline{C}^* = \underline{\phi}^T \underline{C} \underline{\phi} = [2\lambda_m \omega_m]$$

$$\underline{K}^* = \underline{\phi}^T \underline{K} \underline{\phi} = [\omega_m^2]$$

$$\underline{P}^* \ddot{v}_g = \underline{\phi}^T \underline{M} \underline{B} \ddot{v}_g$$

to find the form of $\underline{P}^*$, consider:

$$\underline{M} \underline{B} = \begin{bmatrix} m_1 & & & & & \\ & m_1 & & & & \\ & & J_1 & & & \\ & & & m_2 & & \\ & & & & m_2 & \\ & & & & & J_2 \\ & & & & & & \ddots \\ & & & & & & & J_N \end{bmatrix} \begin{Bmatrix} \sin \beta \\ \cos \beta \\ 0 \\ \sin \beta \\ \cos \beta \\ 0 \\ \vdots \\ 0 \end{Bmatrix}$$

where: m_1 = mass of story 1

J_1 = rotational mass moment of inertia of story 1

i.e.:

$$\underline{M} \underline{B} = \begin{bmatrix} m_1 \sin \beta \\ m_1 \cos \beta \\ 0 \\ m_2 \sin \beta \\ m_2 \cos \beta \\ 0 \\ \cdot \\ \cdot \\ \cdot \end{bmatrix}$$

So, a typical term of $\underline{P}^*$ has the form:

$$\begin{aligned} P_m^* &= \underline{\phi}_m^T \underline{M} \underline{B} \\ &= \langle \phi_{1x} \ \phi_{1y} \ \phi_{1\theta} \ \phi_{2x} \ \phi_{2y} \ \phi_{2\theta} \ \dots \rangle \begin{Bmatrix} m_1 \sin \beta \\ m_1 \cos \beta \\ 0 \\ m_2 \sin \beta \\ m_2 \cos \beta \\ 0 \\ \cdot \\ \cdot \\ \cdot \end{Bmatrix} \end{aligned}$$

$$P_m^* = \sum_{n=1}^N m_n \{ \sin \beta \ \phi_{nx} + \cos \beta \ \phi_{ny} \}$$

Now a typical equation governing the response in the m th mode has the form:

$$\ddot{Z}_m + 2\lambda_m \omega_m \dot{Z}_m + \omega_m^2 Z_m = P_m^* \ddot{v}_g \quad \dots \dots \dots (d)$$

For any earthquake, the ground acceleration, $\ddot{v}_g$ is specified as a set of discrete values and linear interpolation is used for intermediate values. On any linear portion then:

$$\ddot{v}_g = A + Bt$$

where A and B are computed from the end values as shown in Figure 14.

On any linear segment t_1, t_2 then:

$$\ddot{Z}_m + 2\lambda_m \omega_m \dot{Z}_m + \omega_m^2 Z_m = P_m^* (A + Bt)$$

The solution to this equation is summarized in Figure 14.

At rest initial conditions are used for the first linear portion. The values of displacement and velocity at the end of any linear portion form the initial conditions for the following linear segment and so on. Repetition gives the complete solution over the required time span. With solutions for each mode, equation (c) is used to give a set of structure displacements r at each output time step.

The backsubstitution procedure used for the time history analysis is exactly the same as that described for the static analysis in Chapter III. Backsubstitution for each time step is equivalent to one static load backsubstitution. The frame displacements and member forces are determined at each time step and the maxima of these parameters over the time span are output as dynamic load condition 3.

For the equation: $\ddot{z}_m + 2\lambda_m \omega_m \dot{z}_m + \omega_m^2 z_m = p_m^* (A + Bt)$

On any linear segment such as t_1, t_2 , the solution is given by:

$$\begin{aligned} z_m(t) = & p_m^* e^{-\lambda_m \omega_m t} \left\{ \left[z_m(t_1) - \frac{A}{2} + \frac{2\lambda_m B}{3\omega_m} \right] \cos \omega_{Dm} t \right. \\ & + \frac{1}{\omega_{Dm}} \left[\dot{z}_m(t_1) + \lambda_m \omega_m z_m(t_1) - \frac{\lambda_m A}{\omega_m} + \frac{B(2\lambda_m^2 - 1)}{2\omega_m} \right] \sin \omega_{Dm} t \\ & + p_m^* \left[\frac{A}{2} - \frac{2\lambda_m B}{3\omega_m} + \frac{Bt}{2} \right] \end{aligned}$$

42

and:

$$\begin{aligned} \dot{z}_m(t) = & p_m^* e^{-\lambda_m \omega_m t} \left\{ \left[\dot{z}_m(t_1) - \frac{B}{2\omega_m} \right] \cos \omega_{Dm} t \right. \\ & + \left[A - \omega_m^2 z_m(t_1) - \lambda_m \omega_m (\dot{z}_m(t_1) + \frac{t_1}{2}) \right] \frac{\sin \omega_{Dm} t}{\omega_{Dm}} \\ & + p_m^* \frac{B}{2\omega_m} \end{aligned}$$

where:

$$A = \ddot{v}_g(t_1)$$

$$B = \frac{\ddot{v}_g(t_2) - \ddot{v}_g(t_1)}{t_2 - t_1}$$

$$\omega_{Dm} = \omega_m (1 - \lambda_m^2)^{1/2}$$

$z_m(t_1), \dot{z}_m(t_1)$ are the initial conditions for the segment t_1, t_2

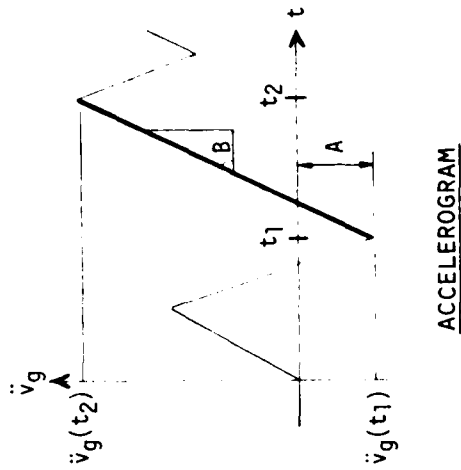


Figure 14. Closed form time integration scheme of CTABS80

E. Response Spectrum Analysis

Unless actual histories of displacements and forces are required for a specific earthquake a more realistic and economical approach for dynamic analysis is via the response spectrum method. For a particular ground motion history $\ddot{v}_g(t)$, the spectrum curve is defined as follows:

The response of a single mass system with damping λ , and circular frequency ω , subjected to a ground motion history $\ddot{v}_g(t)$ is governed by the equation:

$$\ddot{u}(t) + 2\lambda\omega \dot{u}(t) + \omega^2 u(t) = \ddot{v}_g(t)$$

Let $u_{\max}$ be the maximum absolute value that $u(t)$ attains. A plot of this maximum displacement versus the frequency ω for each λ is by definition the displacement response spectrum (Sd) for the earthquake $\ddot{v}_g(t)$. A plot of $u_{\max}\omega$ is the pseudo-velocity spectrum (PSv) and a plot of $u_{\max}\omega^2$ is the pseudo-acceleration spectrum (PSa). These pseudo-velocity and acceleration spectra are of the same physical interest but are not an essential part of a response spectrum analysis.

Recalling equation (d), if the dynamic loading on the structure is specified in terms of the pseudo-acceleration spectrum, then the maximum response for the m th mode is given by:

$$Z_{m\max} = p_m^* \frac{\text{PSa}(\omega_m, \lambda_m)}{\omega_m^2}$$

Therefore, the maximum contribution of mode m to the total three dimensional response of the structure is:

$$r_m = Z_{m_{\max}} \phi_m$$

For all modes S_d is, by definition, positive. The maximum modal displacement r_m is proportional to the mode shape ϕ ; and the sign of the proportionality constant is given by the sign of the modal participation factor, P_m^* . Therefore, each maximum modal displacement has a unique sign. Also, the maximum internal modal forces, which are consistently evaluated from the maximum modal displacements, have unique signs.

A complete analysis is performed down to the member force level with the maximum modal displacements of the structure for each individual mode using the backsubstitution procedure described in Chapter II.

The maxima in each mode will generally occur at different times. The combination of the modal components of the displacements and member forces to give resultant values for design purposes is performed at the design parameter level by the following methods.

1. The Square-Root-of-the-Sum-of-the-Squares (SRSS)⁽¹³⁾ method
2. The Absolute Sum (ABS)⁽¹³⁾ method
3. The Complete Quadratic Combination (CQC)⁽¹⁰⁾ method

The SRSS method and the ABS method entirely neglect the signs of the modal contributions. The SRSS method in general gives good approxi-

mations of the dynamic response in structures with well separated frequencies. The ABS method is basically for interest to give an upper bound on the maximum values.

In structures with closely spaced modes or multiple frequencies, the fact that the SRSS method neglects the signs of the modal components may cause the design parameters to be dramatically overestimated in some elements while being significantly underestimated in other elements. The CQC method overcomes this difficulty and it is recommended as the best of the three methods for obtaining the most realistic results.

F. Dynamic Options

The dynamic options currently available in CTABS80 are:

1. Calculation of mode shapes and periods (frequencies)
2. Response spectrum analysis for any acceleration spectrum supplied by the user using the:
 - a. SRSS modal combination as Dynamic load condition 1
 - b. Sum of absolute value modal combinations as Dynamic load condition 2
 - c. Complete Quadratic combinations as Dynamic load condition 3
3. Time history analysis maxima for any ground motion supplied by the user as Dynamic load condition 3

Either dynamic analysis condition may be combined with any static load condition.

1. SRSS COMBINATION

$$F_{(1 \times 1)} = \sqrt{f_{(1 \times n)}^T \quad \underline{I}_{(n \times n)} \quad f_{(n \times 1)}}$$

Where $\underline{I}$ is an identity matrix

2. ABS COMBINATION

$$F_{(1 \times 1)} = f_{(1 \times n)}^T \text{ sign } f_{(n \times 1)}$$

Where sign f is a unit matrix containing the signs of the corresponding elements of matrix f

3. CQC COMBINATION

$$F_{(1 \times 1)} = \sqrt{f_{(1 \times n)}^T \quad \underline{C}_{(n \times n)} \quad f_{(n \times 1)}}$$

Where $\underline{C}$ is the matrix of modal cross-correlation coefficients given by:

$$C_{ij} = \frac{8\lambda^2(1+r)r^{3/2}}{(1-r^2)^2 + 4\lambda^2r(1+r)^2}$$

NOTES/

f = vector of modal components

F = combined resultant

n = number of modes

where $r = \omega_i/\omega_j$, the ratio of the circular frequencies of the coupling modes and λ is the damping associated with the response spectrum curve being used.

Figure 15. Summary of modal combination techniques used in CTABS80

CHAPTER V: GENERAL OBSERVATIONS

A. Program Application

The effective application of a computer program for the analysis of practical situations involves a considerable amount of experience. The most difficult phase of the analysis is in assembling an appropriate model which captures the major characteristics of the structural behavior of the building. No computer program can replace the engineering judgement of an experienced engineer. It is well said that an incapable engineer cannot do with a ton of computer output what a good engineer can do on the back of an envelope. Correct output interpretation is just as important as the preparation of a good structural model. Verification of unexpected results needs a good understanding of the basic assumptions and the mechanics of the program. Static equilibrium checks are necessary not only to check the computer output but to understand the basic structural behavior of the building.

B. Static Seismic Analysis of Buildings

At the present time, the seismic design of most buildings in California and other earthquake regions of the United States is based upon the Uniform Building Code. The UBC method allows the seismic loads to be approximated by an equivalent set of lateral static loads. The magnitude of the loads is based upon the seismic zone, the structural system, and the fundamental period of the structure. Corrections to compensate for local soil conditions and the physical importance of the structure are also defined.

An approximate formula, specified in the UBC, may be used to estimate the fundamental period. The period associated with the predominant structural mode obtained via the TABS program is more accurate and appropriate. The suggested UBC distribution of the lateral loads over the height of the building is triangular with some correction to allow for higher mode effects. Behavior of structures that have dynamically decoupled regions due to stiffness and/or mass discontinuities, causing significantly non-triangular inertia load patterns are not adequately covered by the code. By examining the structural modes produced by a TABS analysis such structural complexities can be isolated.

The determination of the minimum horizontal torsional design moments, as specified by the UBC for the design of structures having rigid diaphragms, requires the location of a center of rigidity of the structure at each level. The definition of torsional moments on such a basis for multi-story structures is technically vague. It is only meaningful in single story structures where there are no stories above or below to affect the rotation of the level under consideration.

The UBC lateral loads are only a small fraction of the loads developed during a significant earthquake, and must therefore be considered as minimum requirements. As a result of the above mentioned inadequacies the need for a more comprehensive code earthquake analysis methodology is apparent to most structural engineers⁽⁸⁾.

C. Computer Methods Versus Hand Methods

High speed digital computers and the development of computer programs such as TABS have given engineers the capability to consider aspects of

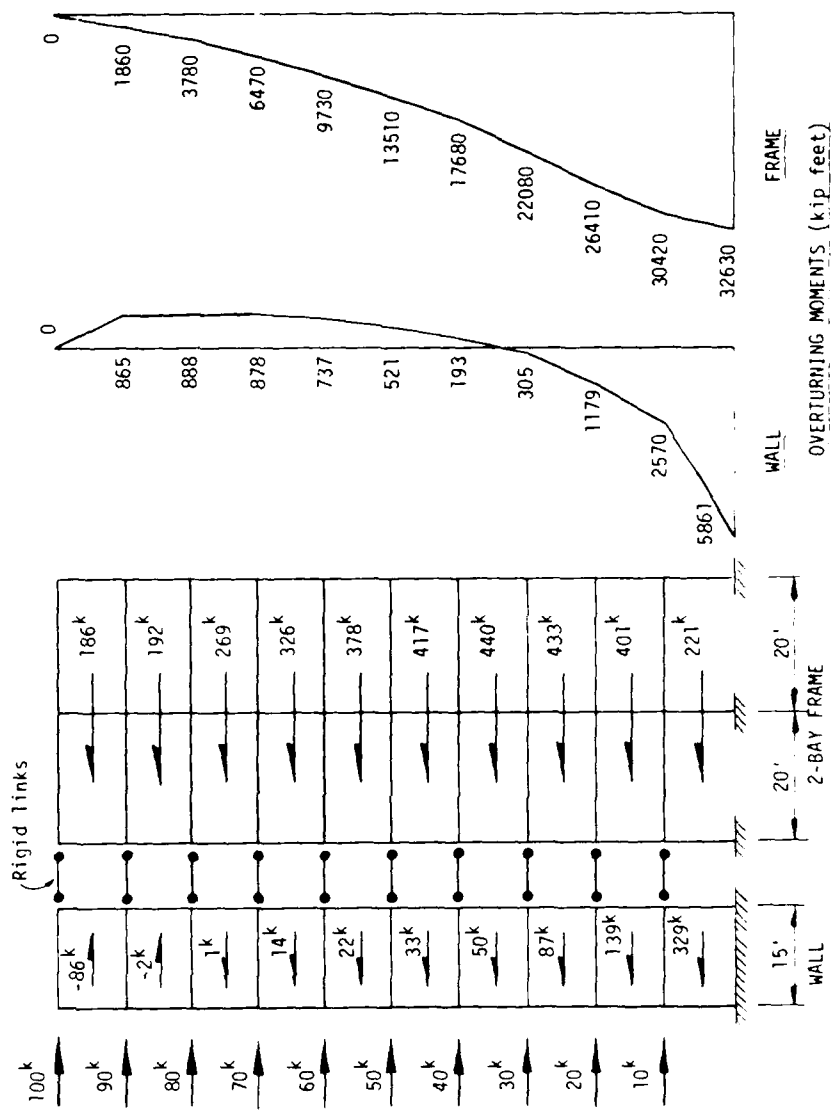
structural behavior that conventional hand analysis techniques have traditionally neglected. Hand analysis techniques used by practicing engineers for the lateral analysis of multistory structures have been shown to violate joint statics and compatibility.

The following examples demonstrate the degree of error that could be present in a conventional hand analysis, by comparison with a TABS analysis; i.e., one that completely satisfies statics, compatibility, and boundary. Examples presented are of simple symmetrical multistory buildings with symmetrical loading. The stories, therefore, translate under lateral load without rotating, thereby keeping the problems clear and demonstrative. The proportions of the structures and magnitudes of the forces have been chosen to generate problems of a nature that a conventional structural engineer is commonly faced with in practice.

(i). Example 1

This is a classic example of shear wall-frame interaction. A 10 story shear wall is connected in parallel with a ten story frame at each story level through a rigid link. The axial deformations in the beams are neglected, thus simulating a rigid diaphragm. Therefore, the lateral displacements of the respective stories of the frame and shear wall are equal. See Figure 16.

Consider the top story. Based on a conventional hand analysis, one would, in general, tend to ignore the stiffness of the frame and conclude that the shear wall takes close to 100% of the applied 100 K, and that the frame being relatively flexible gets a negligible amount of the shear.



NOTE/

Walls are 12" thick
Columns are 24" x 24" typ.
Beams are 12" x 24" typ.
Story height = 10' typ.

Figure 16

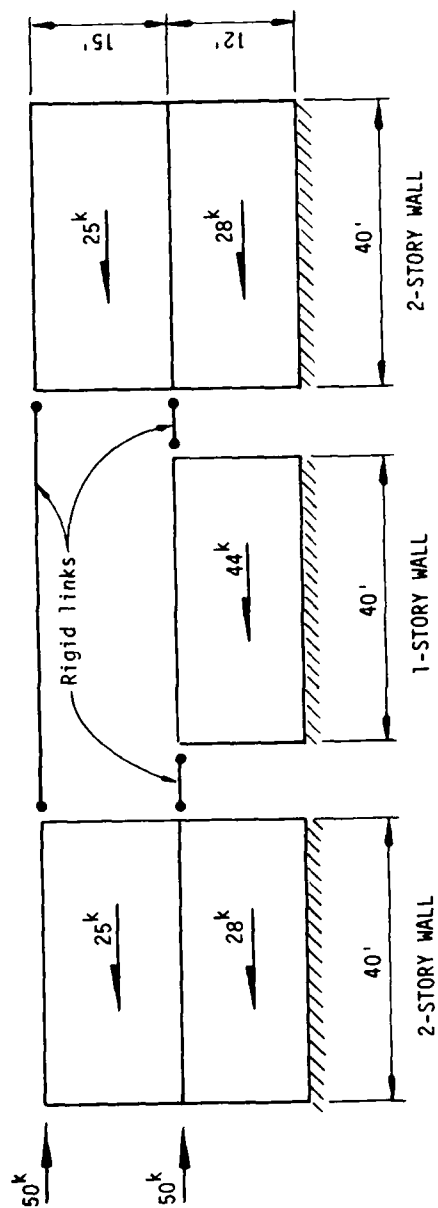
A "correct" analysis, however, reveals that the frame has a total shear of 186 K at that level. Thus, the frame, besides carrying the total 100 K applied load, is laterally supporting the shear wall which puts an additional 86 K on the frame. The shear wall is in effect acting as a propped cantilever supported by the frame at the upper story levels.

The phenomenon may be explained as follows: If the frame and shear wall are loaded independently with the load, and the lateral displacements of the respective floors compared, it will be observed that the shear wall has larger lateral displacements in the upper levels, whereas the frame has larger displacements in the lower stories and vice versa. Compatibility of joint rotations will have a significant effect on these displacement patterns. When loaded together, the constraint of equal story displacements is enforced, thus resulting in this unique shear distribution. Notice that in the lower stories the shear gradually shifts to the shear wall.

This example demonstrates the importance of the interaction of all the elements on one another and that the hand analysis method of analyzing an n-story structure as n 1-story structures stacked one over the other with no interaction of one on the other can lead to highly unreliable results.

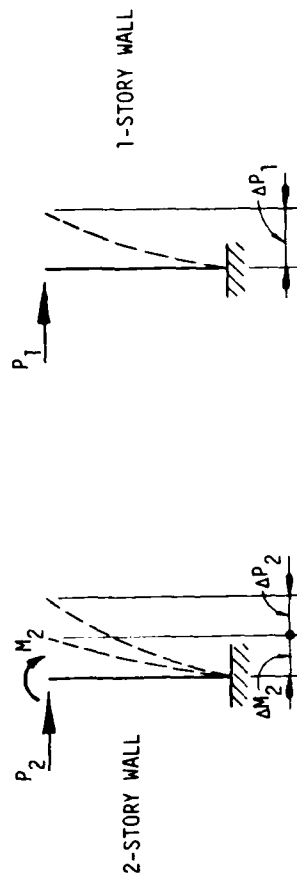
(ii). Example 2

This problem consists of two 2-story walls and one 1-story wall. See Figure 17a. Again, as in Example 1, the walls are connected by rigid links at the floor levels to simulate a rigid diaphragm and to enforce



NOTE/
All walls 12" thick

(a) STRUCTURE



(b) FREE BODY DIAGRAM OF LOWER LEVEL

Figure 17

equal lateral story displacements in all the walls. The walls are loaded with a total story load of 50 K at each level. As expected in a conventional hand analysis, the 2-story walls equally share the 50 K lateral load at the upper level.

Now let us consider the shear distribution in the bottom story. The base shear is 100 K. In the light of the fact that there are three resisting elements at this level, all 40' long and 1' thick, a conventional hand analysis would conclude that the base shear will be carried equally by all three elements; that is 33 K each.

A "correct" analysis, however, indicates that the 1-story wall takes over 50% more shear than each 2-story wall. In Figure 17b are presented the free body diagrams of the lower levels of the 1-story wall and the 2-story walls. Consider the lateral story displacements of the 1st level in each wall. In the 1-story wall the lateral displacement, ΔP_1 , is due to P_1 , the shear force in the wall. In the 2-story wall the lateral displacement is due to two factors. Firstly, ΔP_2 , that is due to P_2 , the shear force in the wall and, secondly, ΔM_2 , due to M_2 , the moment at the top of the wall due to the fact that the wall is 2 stories high. Now for the lateral displacements to be equal:

$$\Delta P_1 = \Delta P_2 + \Delta M_2$$

Therefore $\Delta P_1 > \Delta P_2$

so that $P_1 > P_2$

The discrepancy between the conventional hand analysis method and the "correct" method here, again, is due to the fact that the effect of the upper story on the lower story is accounted for incompletely.

(iii). Example 3

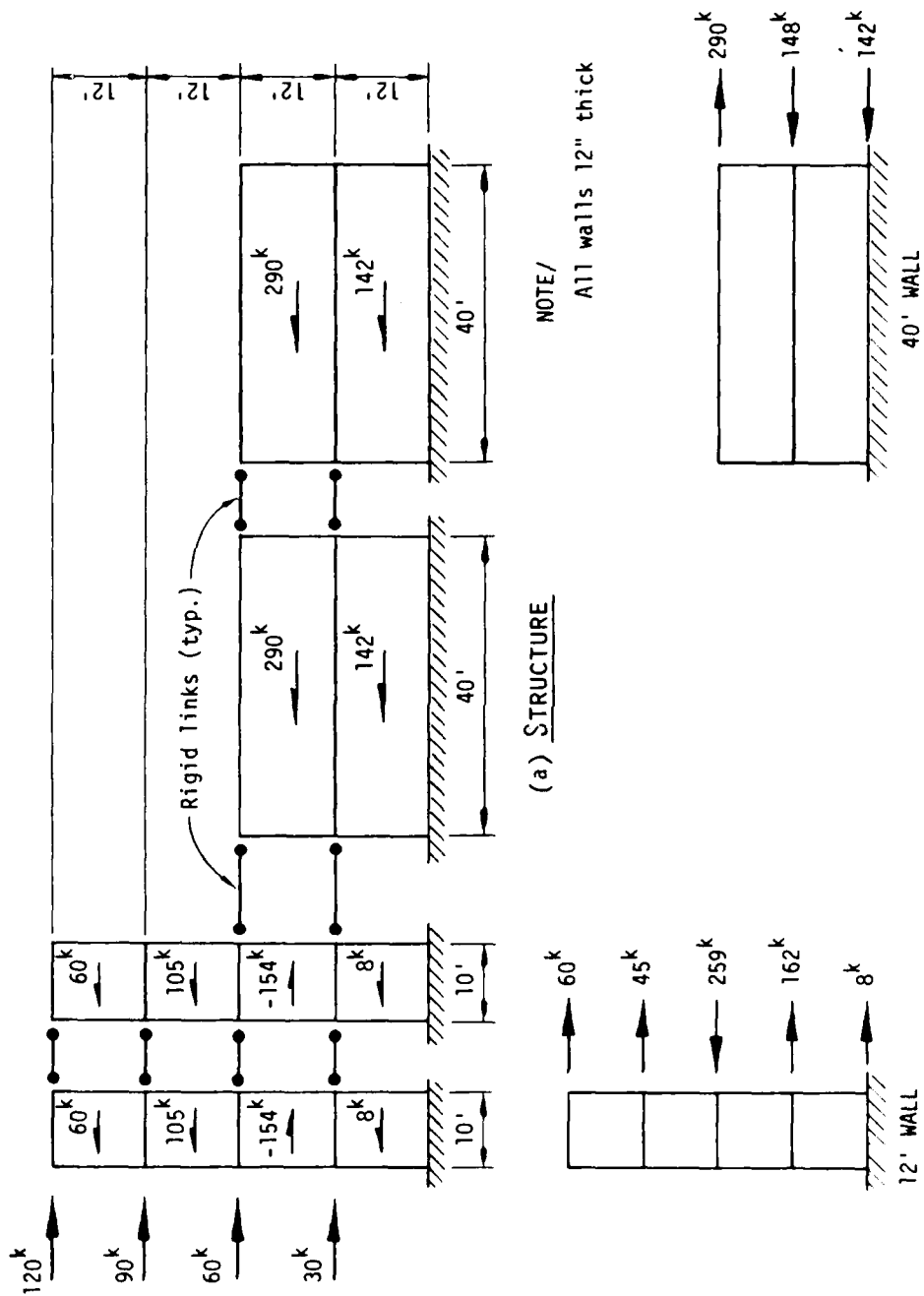
This structure consists of 4 walls. Two 4-story walls and two 2-story walls. See Figure 18a. Again, the walls are connected by rigid links at the floor levels to simulate a rigid diaphragm.

In the "correct" analysis the shear forces in the 10 foot walls in the 3rd and 4th levels are as would be expected in a conventional hand analysis. Note the shear distribution at the 2nd level. At this level there is a considerable increase in the story stiffness due to the two 40-foot walls. This restricts the lateral diaphragm movement to the extent that the 10' walls are in effect laterally supported by the diaphragm at this level, and, therefore, behave like over-hanging cantilevers as shown in Figure 18b. This explains why these walls have a negative shear of 154 K each at this level.

The conventional hand analysis method for such problems completely disregards the possibility of negative shear forces occurring in the walls, thereby always assuming that the walls support the diaphragm laterally, and not recognizing that at times the diaphragm may actually be the support for certain walls at certain levels.

(iv). Example 4

This example demonstrates the effect of axial deformations on the distribution of shear to a series of walls. Consider the structure shown in Figures 19a and 19b. The structure consists of two solid walls, and one wall terminating on two columns. In case A, the columns are 12" square, see Figure 19a. Again, the walls are connected by rigid



(b) FREE BODY DIAGRAMS (SHEAR LOADS)

Figure 18

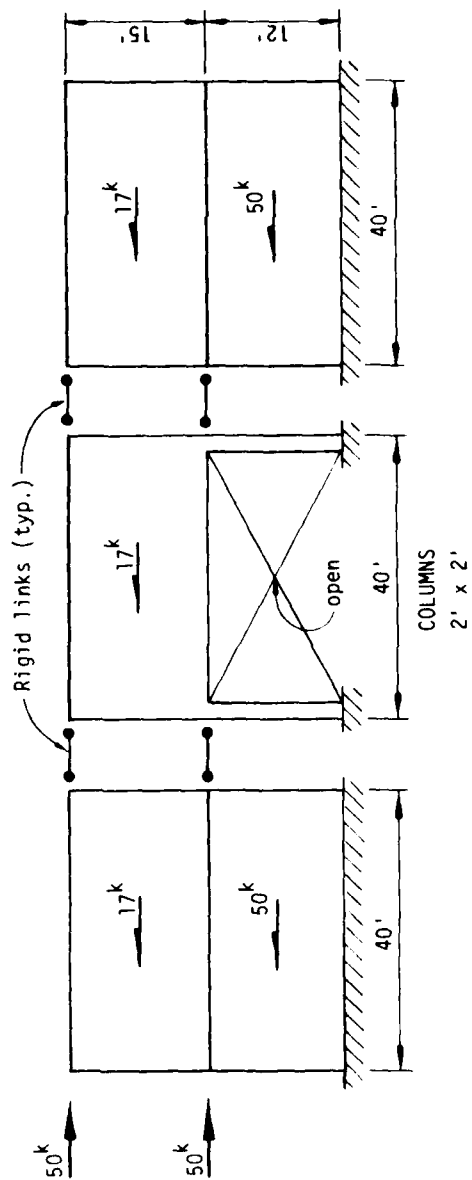


FIGURE 19a: CASE A

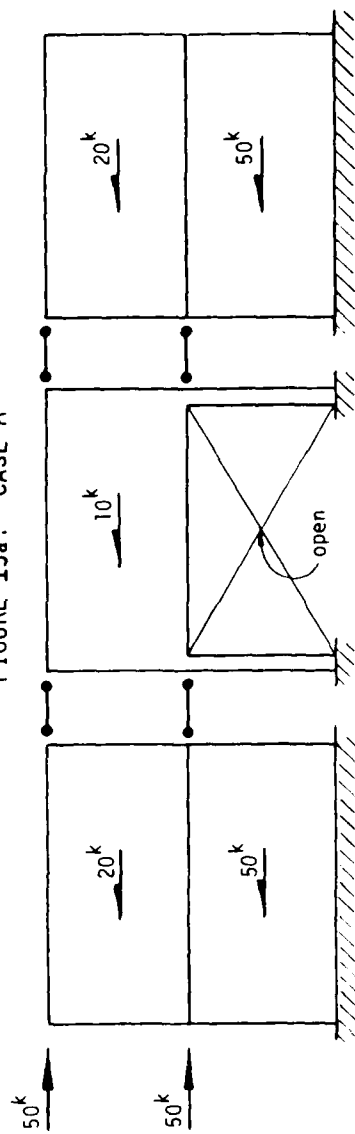


FIGURE 19b: CASE B

NOTE/
All walls 12" thick

Figure 19

links at the story levels.

Consider the shear distribution in the upper level. In case A, a "correct" analysis indicates that the 50 K story shear is distributed approximately equally among the 3 walls. A conventional hand analysis would lead us to the same conclusion. In case B, however, the shear taken by the wall terminating on columns is 50% of that taken by the solid walls. The reason being that the smaller axial area of the columns gives larger axial deformations which, in turn, reduce the lateral rigidity of the wall in the story above.

A conventional hand analysis neglects the effects of axial deformations, and, therefore, would give an equal shear distribution in all three walls regardless of the size of the columns.

(v). Example 5

This example clearly demonstrates the analytical discrepancy in analyzing an n-story structure as n 1-story structures. The structure is a shear wall with the same story height, wall dimensions and openings at every level. See Figure 20.

A conventional hand analysis (analysis of the structure as four 1-story structures) would indicate that the location of the point of contra-flexure of the piers in all the stories would be at a constant distance vertically from the corresponding diaphragm. Also, the ratio of the shear force in pier A to that in pier B in all stories will be constant.

A "correct" solution of the structure, however, shows that there is no point of contra-flexure in pier A in the first three stories. Also, the

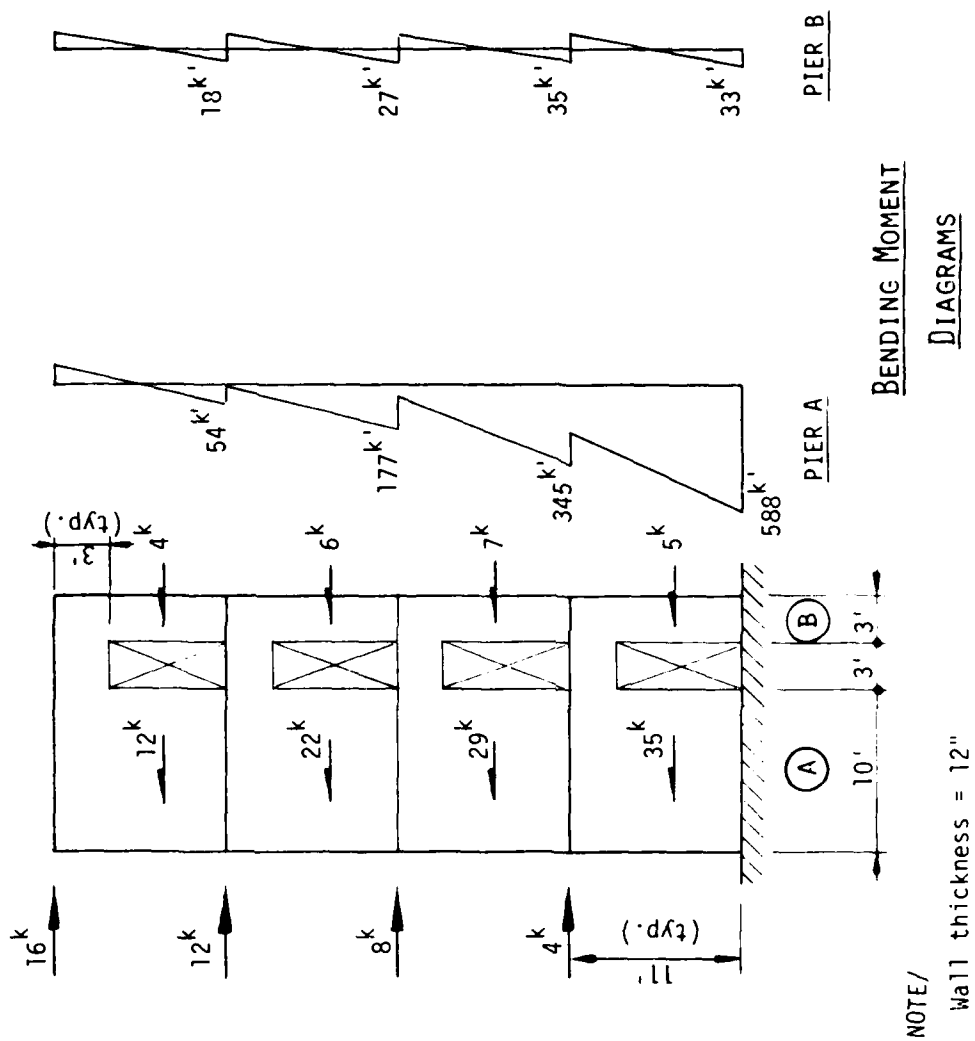


Figure 20

percentage of the story shear carried by pier B is not constant in all stories but decreases as we move into the lower stories.

Joint rotation incompatibility from story to story is the main cause for this discrepancy between the hand analysis and the "correct" analysis.

(vi). Example 6

This is another example demonstrating the effect of axial deformations on the shear distribution. The structure consists of slender piers framing into relatively stiff spandrels and is 8 stories tall. See Figure 21. For all practical purposes, the piers may be considered fixed in rotation at both ends. Since all the piers are of the same size, a conventional hand analysis would indicate that the story shear is distributed equally among the 5 piers.

A "correct" analysis, as we can see, indicates that the piers closer to the center take a higher percentage of the story shear. In the top story for instance, the center pier takes over 65% higher shear than the end pier. This discrepancy is due to the fact that the "correct" analysis considers axial deformations in the piers, whereas the hand analysis does not.

This behavior may be explained by the following analogy. Consider the lateral displacements in a vertical cantilever with a rectangular cross section and lateral loading. If the shear deformations are negligible compared to the bending deformations in the cantilever, the distribution of the shear stress across the section is parabolic with the maximum at the center. However, if the deformation pattern is one of pure shear,

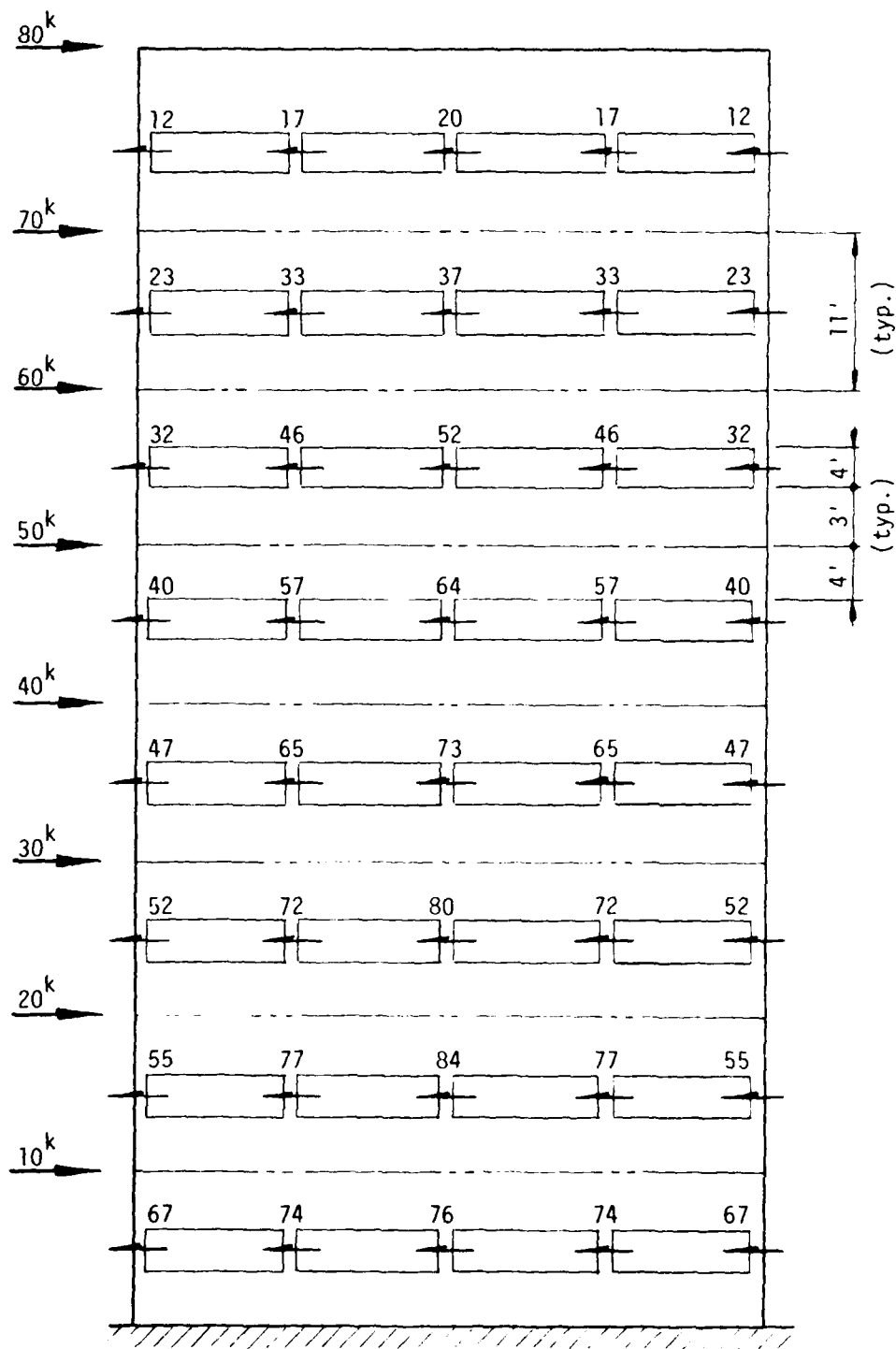


Figure 21

such that the bending deformations are negligible, the shear stress distribution is constant over the section and not parabolic.

The axial deformations of the piers in this example correspond to the bending deformations in the analogy above. If the piers were axially stiff to the extent that there were no axial deformations, the shear distributions in the piers would be equal. However, as the piers deform axially, the piers closer to the middle have heavier shear. This also explains why the shear distribution is more uniform as we move into the lower stories.

D. Dynamic Seismic Analysis of Buildings

The deficiencies of the present seismic design procedures are clearly summarized in Reference 8. It is apparent that the present code is a very approximate method based on the first mode only. The foundation factors discussed later are not considered. Another factor which is important in an elastic analysis is the damping. Spectra for damping of 2 and 10% are shown in Figure 22. It is clear that the Uniform Building Code seismic loads are very small compared to the force produced in recorded earthquakes. It has been estimated that earthquakes of the Parkfield magnitude can be expected about once per year at some point in California, and earthquakes of the El Centro magnitude may be expected every five or six years.

The selection of a design spectrum for the response spectrum analysis of a particular building will depend on the geographical area, the local soil condition, the type of construction material and the intended use of the

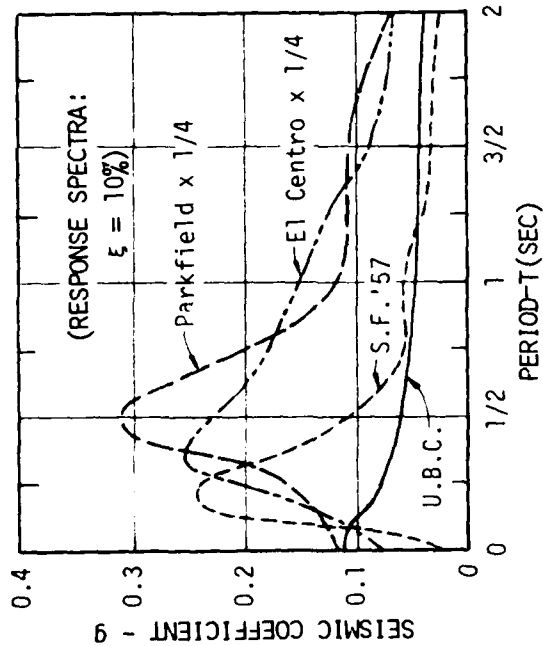
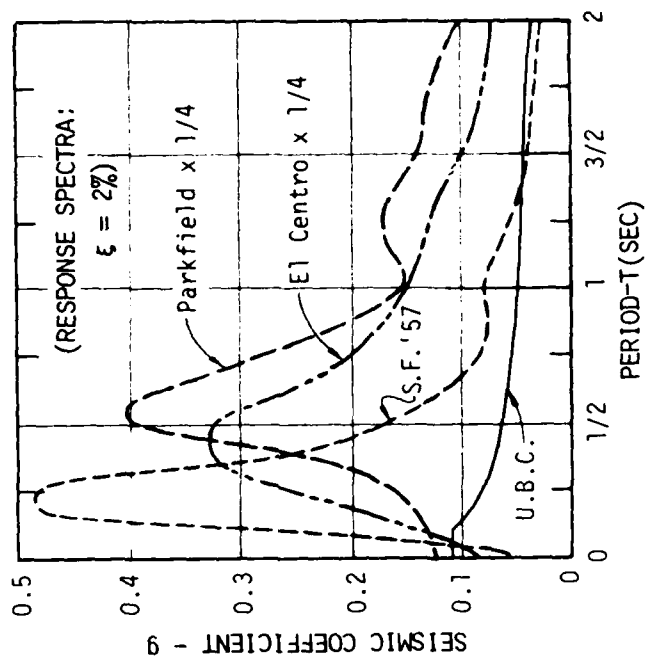


Figure 22. Response spectra

building. Many Soils Engineering firms now specialize in the dynamics of soil systems to evaluate specific sites and recommend shapes and intensities for dynamic response spectra. Most lending agencies are requiring dynamic analysis of structures as part of their financing terms for buildings in major metropolitan areas of California.

For certain types of earthquakes it has been observed that the vertical accelerations are comparable in magnitude to the lateral accelerations. However, all building have been designed elastically for a minimum of $1g$ in the vertical direction; therefore, these additional vertical forces very often do not cause direct damage to the structure. Of course, they should be considered in the design of members in addition to the lateral earthquake loads. For most structures the stiffness in the vertical direction is very large; hence, the vertical periods will be very small. Therefore, a dynamic analysis in the vertical direction may not be required. A direct increase in dead load stresses may be a good method to approximate the effects of vertical earthquake loads.

E. Foundation - Building Interaction

Foundation modeling has always been an area of particular concern. The vertical and rotational stiffnesses under each column can be easily input by providing an extra "dummy" story. However, the assigning of accurate stiffness values for these soil springs can be difficult.

In recent years considerable research has been conducted in the area of foundation - building interaction. However, very little of this work has been of direct value to the profession involved in the earthquake analysis

of buildings. Several of the suggested approaches have been difficult to apply in case of complex buildings, or they have had serious theoretical restrictions.

Before foundation interaction effects are included in the analysis it is necessary to define the exact location of the earthquake input. If the design criteria states that the input is at the base of the building then it is impossible to say that the building will modify the input, and it is impossible to include interaction effects.

A large amount of research in this area has been associated with machines vibrating on an infinite foundation where the term radiation damping has been used. This work has little value in earthquake engineering since the energy source is not at the base of the building. It is easy to show that the energy stored in the building is very small compared to the energy stored in the immediate foundation area in the case of earthquake input. Also, the machine vibration problem is a steady state phenomenon, whereas earthquakes produce a transient loading.

The continuous foundation contains an infinite number of degrees of freedom. Therefore, any approach which suggest representing the lateral behavior of the foundation with a simple spring, dashpot and mass system is a very gross approximation. In fact, this technique can produce a filtering effect on the earthquake input and cause serious errors. For lateral earthquake input, this type of approximation is only acceptable in the representation of the rotational stiffness at the base of columns and shear walls.

The most significant factor to consider is the modification of the basic

earthquake rock motion by the layers of soil material under the building⁽⁷⁾. For certain earthquakes and locations this may be a factor of 2 or 3 in amplification. Therefore, it is very important that the dynamic behavior of the site is studied independently of the building. The results of such a study will result in a suggested acceleration spectrum to be used in the analysis of the building. Figure 23 indicates the type of results which can be expected from such a site analysis.

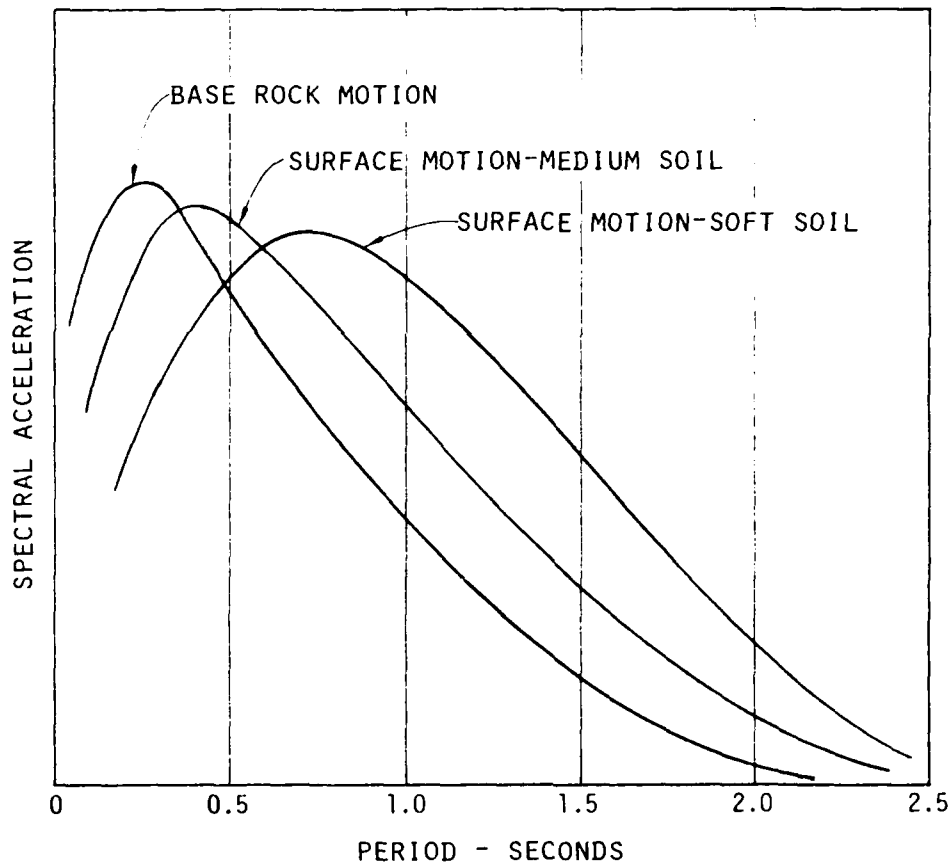


Figure 23. Spectral variation due to soil conditions

CHAPTER VI: INTERNAL ORGANIZATION

An outline of the subroutine structure of CTABS80 is presented in Figure 24. There are seven major calls from CTABS80 associated with the seven major blocks of the program.

1. The first operation is to read the basic control information. The data associated with the complete building (story data and structural lateral loads) is then rolled in via subroutine TABI.
2. The next operation involves reading in the frame data of every different frame in the structure. The frame elevations are plotted, if requested. In non-data check modes the frame stiffnesses are formulated and reduced and the frame lateral stiffness matrices and back-substitution equations are written sequentially on disc. This operation is implemented by the call to subroutine TABF.
3. The call to subroutine TABL reads the frame location data and formulates the complete lateral structural stiffness matrix of the whole building.
4. Subroutine SFRAME causes a plan view of the building to be plotted, showing the frame locations and the directions of their local axes.
5. The call to subroutine TABE gives the modeshapes and frequencies of the structure (TABM) and triggers the automatic UBC lateral seismic load calculation (TUBC). Also the dynamic analysis control information is read in by this call. Structural lateral displacements due to the static loads (TABQ) and response spectrum dynamic loads are obtained at this stage.

6. Subroutine TABDY reads in the time history earthquake ground motion data and causes the structural lateral displacements to be calculated for each time step.
7. Finally, subroutine TABP is called. This subroutine calls TABC to read the load case definition data for each frame. Then TABU is called to print the frame lateral displacements and TABO is called to calculate the frame joint displacements for each static load condition and each spectral mode or response time increment from the back substitution equations previously saved.

As the displacements are calculated the member forces are also evaluated and printed according to the load case definition data (TABW).

CHAPTER VII: CONCLUSIONS

A general computer program for the elastic three-dimensional static and dynamic analysis of frame and shear wall buildings has been presented. For buildings which can be approximated by independent frames and shear walls the program is very economical and easy to use as compared to a general purpose three-dimensional structural analysis program.

Many new options have been implemented in this release to make the program a more practical and useful engineering tool.

The program is based on linear theory. Non-linear behavior such as $P-\Delta$ effects and material plasticity are not captured by the program.

If non-linear effects are to be considered a step-by-step response analysis is required; however, this involves a significant increase in computational effort and will be justified for only a limited number of buildings. In addition, the non-linear material properties both for most structural and non-structural members have not been established accurately from experimental work.

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APPENDIX A: FORTRAN IV LISTING OF CTABS80


```

STOP 77777
150 CONTINUE
CALL SECOND (T111)
SETTING RIGID ZONE REDUCTION FACTOR
RIGID=25
IF (INQD.NE.O) RIGID=O.
C
NIMP=1
NOUT=1
IF (INOPT.EQ.3) NIMP=O
IF (INOPT.EQ.2) NIMP=O
IF (INOPT.EQ.2) NOUT=O
C
I3=3
I3=O
IF (INQD.EQ.O) GO TO 50
I3=MSD-1
I3=MSD-1
50 MSS=ASTO13
IF (INQD.EQ.3) MSS=MS
IF (INQD.EQ.3) MSS=O
CALL NUPAJ (L1,Z7,TESTE)
WRITE (HOUT,20001) MST,NDF,ATF,MO,MAT,NEQ,MSD,MUPT,NBOD,MOSP,
NURC,NPA7,NP17
C
IF (INQD.EQ.1) NAL=1
IF (INQD.EQ.1) NAL=1
BEHIND RTAPE
WRITE (RTAPE) INED,DATE
WRITE (RTAPE) MST,INDE,ATF,MSTF
C
STRESS TRANSFORMATION DATA
READ (INIMP,10021) ANI,AMP
IF (ANI.EQ.O) ANI=1.0
IF (AMP.EQ.O) AMP=1.0
WRITE (HOUT,10031) ANI,AMP
C
BEHIND 1
BEHIND 2
BEHIND 3
READ AND PRINT OF STUDY INFORMATION

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TAB580 101 NO=1
TAB580 102 M1=NO*ASTOND
TAB580 103 M2=N1+15*NST
TAB580 104 M3=N2+2*NST
TAB580 105 M4=N3+7*NST
TAB580 106 M5=N4+NPAT
TAB580 107 M6=N5+NPAT
TAB580 108 M7=NO*NPAT
TAB580 109 M8=N7+NPAT
TAB580 110 M9=NO*NST
TAB580 111 IF (NO*CT-MTGT) CALL TAPP (INO-MIDT),13
TAB580 112 CALL TABE (AIN1,AIN2,AIN3),AIN1,AIN2,AIN3,AIN7,AIN8,
N1,NPAT)
TAB580 113
TAB580 114 READ AND PRINT OF FRAME PROPERTIES AND FORM LATERAL STIFFNESS
TAB580 115 C
TAB580 116 DO 200 I=1,MDF
TAB580 117
TAB580 118 READ (INIMP,10011) FMD,F,MC,MS,MCP,MFP,MFEF,MCOM,MFAN,MOTIG,IPLT,
TAB580 119 N1,NPAT) MSCC
TAB580 120
TAB580 121 IF (INPLT.EQ.O) IPLT=O
TAB580 122 IF (INPLT.EQ.3) IPLT=O
TAB580 123 IF (MSCC.NE.O) MSCC=1
TAB580 124 IF (INIMP.EQ.O) GO TO 250
TAB580 125 CALL SECOND (T11)
TAB580 126 LINE=99
TAB580 127 CALL NUPAJ (L1,Z7,TESTE)
TAB580 128 WRITE (HOUT,20011) FMD,F,MC,MS,MCP,MFP,MFEF,MCOM,MFAN,MOTIG,IPLT,
TAB580 129 MSCC
TAB580 130
TAB580 131 C 250 M8=MC-1
TAB580 132 MCP=MCP-1
TAB580 133 MFP=MFP-1
TAB580 134 M8=MC+NST
TAB580 135 M9=N3+RNCP
TAB580 136 M3=N2+NR
TAB580 137 M5=N4+RNCP
TAB580 138 M6=N5+RNCP
TAB580 139 M7=N6+RNCP
TAB580 140 M8=N7+RNCP
TAB580 141 M9=N8+RNCP
TAB580 142 M10=N9+RNCP
TAB580 143 M11=N10+RNCP
TAB580 144 M12=N11+RNCP
TAB580 145 M13=N12+RNCP
TAB580 146 M14=N13+RNCP
TAB580 147 M15=N14+RNCP
TAB580 148 M16=N15+RNCP
TAB580 149
TAB580 150

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TAB580 197
TAB580 198
TAB580 199
TAB580 200

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N17=N16*NNNN
N18=N17*NNNN
N19=N18*NNNN
N20=N19*NNNN
IF (IN70-CT.MTOT) CALL TABR(IN70-MTOT,1)
CALL TABR
* IATN01,AIN11,AIN21,AIN31,AIN41,AIN51,AIN61,AIN71,AIN81,AIN91,AIN101,AIN111,AIN121,AIN131,AIN141,AIN151,AIN161,
* AIN171,AIN181,AIN191,
* NST=NC,NB,NS,NCP,NBP,NPEF,MCDML,NFAM,MDIC,NW,I,MDPT,
* JPLT,NAL,NSEC)
C
IF (IN1P.EQ.0) GO TO 200
CALL HUPAJ(1,5,ITEST)
CALL SECOND (TTTT)
WRITE (ENDUT,2003) 1,ITTY
200 CONTINUE
* MDC=N0*NST*(MDF+2)-1
WRITE (XTAPE) (A11,I=MD,MLOC)
C
CALL SECOND (T123)
C
ASSEMBLE STRUCTURAL STIFFNESS AND LATERAL LOAD VECTORS
N1=N2+N55*NS
N4=N3+6*NS
N5=N4*(INST+1)*NST/2
N6=N5+6*NST
N7=N6+NST*NF
N8=N8+6*NF
IF (IN70-CT.MTOT) CALL TABR(IN70-MTOT,1)
CALL TABR
* AIN01,AIN11,AIN21,AIN31,AIN41,AIN51,AIN61,AIN71,AIN81,AIN91,AIN101,AIN111,AIN121,AIN131,AIN141,AIN151,AIN161,
* AIN171,AIN181,AIN191,
CALL SECOND (T133)
C
PLOTING PLAN VIEW / FRAME LOCATIONS
IF (NPLT.EQ.0) GO TO 550
CALL SFRAPE
550 CONTINUE
* URC LATERAL LOAD DATA

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300 TABS80 300

A6


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1025 FORMAT (4SM,M=1,NGT GREATER THAN ZERO AT LEVEL,110)
1026 FORMAT (50M,NUMBER OF SPECTRUM PERIOD CARDS SHOULD BE 1,200,110)
1027 FORMAT (4SM,DAMPING RATIO IS GREATER THAN 1 FOR MODE,110)
1028 FORMAT (4SM,DAMPING RATIO IS LESS THAN ZERO FOR MODE,110)
1029 FORMAT (4SM,INCONSISTENT ACCELERATION DATA AT POINT,110)
1030 FORMAT (4SM,TIME FOR PERIOD MUST INCREASE SEQUENTIALLY,110)
1031 FORMAT (4SM,ACCELERATION HISTORY TIME SPAN IS LESS THAN,110)
1032 FORMAT (4SM,REQUESTED STRUCTURAL RESPONSE TIME,110)
1033 FORMAT (4SM,STRUCTURAL LEVEL NUMBER FOR FRAME CONNECTIVITY,110)
2000 FORMAT (10M,OUT OF RANGE IN SEQUENCE AT LEVEL NUMBER,110)
9999 FORMAT (4SM,EXCESSIVE ERRORS OR IMMEDIATELY FATAL ERROR,110)
*
END

SUBROUTINE TAB1 (SD,PA,PB,XM,XI,XZ,XY,INST,NST,NPAT)
READ AND PRINT STORY DATA
DIMENSION SD(NST,15),PA(NST,2),PB(NST,2),XI(NST,15),XZ(NST,15),XY(NST,15)
COMMON /HMDR/ (HMDR(28),IDATE(2),NPAGE(2),PAGE(2),MAXLIN,LINE
COMMON /ETAP/ KIMP,KIMP,KOUT,ISTR,MTAPE,RTAPE
COMMON /NDPT/ NIMP,NOUT
IF (NPAT.LE.0) GO TO 900
DO 775 MT=1,NPAT
XI(MT)=0.
XZ(MT)=0.
XY(MT)=0.
XI(MT)=0.
READ (KIMP,1001) P,NSEC,SF
IF (NSEC.LE.0) NSEC=1
IF (SF.LE.0) SF=1.
IF (NIMP.EQ.0) GO TO 650
LINE=99
CALL NUPAJ (1,9,ITEST)
WRITE (KOUT,8000) MT,NSEC,SF
WRITE (KOUT,2000)
650 DO 700 NG=1,NSEC
READ (KIMP,1002) AM,EC,YC,XD,YD
IF (NIMP.EQ.0) GO TO 675

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TABS80 589
TABS80 590
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TABS80 592
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TABS80 630
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TABS80 680
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TABS80 682
TABS80 683
TABS80 684

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A12

A14

A15


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IF (IPLT.EQ.0) GO TO 980
P6=0.0
IF (M.EQ.1) YI=Y(N+1)
J015)=MC
CALL ELPLDT (XEM1,XEJ1,LI,YI,VINI,DD,CPI5,MC),JD,IPLT,3)
910 CONTINUE
IF (INDPT.EQ.1) GO TO 900
D=0.0
DO 740 I=M,J
740 D=D+BI(I)
80=TL0000CPI1,MC)CPI1,MC)/2.4
IF (80) 30,70,20,10,5
710 CMM=2.0CPI1,MC)CPI1,MC)/XLE(1,2,000))
720 CMM=2.0CPI1,MC)CPI1,MC)/XLE(1,2,000))
730 CMM=2.0CPI1,MC)CPI1,MC)/XLE(1,2,000))
740 CALL TART (5,5,5,5,5,5)
900 CONTINUE
P15,13=0.0
P6=0.0
ALL CONTINUE
% FURN DIAGONAL STIFFNESSES
IF (INDIC.EQ.0) GO TO 615
DO 760 M=1,MOIC
MOI=M-LOIC(1,M)+1
IF (M-WE.MOI) GO TO 560
OT=5TL
18=LOIC(2,M)
17=LOIC(3,M)

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TABS80 1481
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TABS80 1483
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TABS80 1499
TABS80 1500
TABS80 1501
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TABS80 1529
TABS80 1530

MC=POIC(M)
IF (MC.EQ.0) GO TO 560
D=0.0
DO 510 I=2,IT
DO 510 DX=DX+BI(I-1)
521 IF (18-EQ.1) GO TO 541
DO 531 I=2,18
531 DX=DX+BI(I-1)
541 XL=5071DX+2-DY+92)
P16,MAL)=XL+CPI2,MC)CPI2,MC)
VERTINAL)=VERTINAL+P16,MAL)
IF (IPLT.EQ.0) GO TO 990
YI=Y(N+1)
IF (M-WE.MS) YI=Y(N+1)
J015)=MC
CALL ELPLDT (XEM1,XEJ1,LI,YI,VINI,DD,CPI5,MC),JD,IPLT,4)
990 CONTINUE
IF (INDPT.EQ.1) GO TO 560
M=0.0
IF (CPI1,MC) 580,580,585
595 BM=15.40CPI1,MC)/XLE(2,CPI4,MC)
580 CMM=2.0CPI1,MC)CPI1,MC)/XLE(1,2,000))
5A=CMM+12.000)
5B=CMM+12.000)
5C=CPI2,MC)CPI2,MC)/XL
CALL TART (1,5A,5B,5C,XL,0,55,CA)
LM(1)=2010,M)
LM(2)=MN+1
LM(3)=2017
LM(4)=LM(2)-1
LM(5)=LM(1)-1
LM(6)=LM(3)-1
CALL TART(5,5,5,5,5,5)
580 CONTINUE
P16,MAL)=0.
REDUCE STIFFNESS MATRIX FOR LEVEL M

TABS80 1531
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TABS80 1579
TABS80 1580

A18

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9000 FORMAT (11A15)
9000 CONTINUE
END

SUBROUTINE TART (MSC,LBC,SD,MS,MR,ML,MPROP,NST,IF)
C
C PRINT MEMBER LOCATIONS AS GENERATED
C
C DIMENSION LBCIN,MR,ML,SD(NST,15),MSC(NST,1)
C
COMMON /TARPE/ RIMP,KIMP,KOUT,KSTR,MTAPE
COMMON /TMEOR/ TMEC(20),TDATE(2),MPAGE(2),MAXLIN,LINE
COMMON /JUNK/ MLC(LCCLIS)
C
R=NST-N
DO 300 J=1,MR,15
IF (J).GT. NR J=NR
WRITE (KOUT,2000) (K-K-J,J)
WRITE (KOUT,2001)
C
DO 200 I=1,MS
DO 100 K=J,JJ
K1=K-J+1
LOC(K1)=LBC(I,K1)
IF (LOC(K1).EQ.NPROP) LOC(K1)=0
100 CONTINUE
C
K=J-J+1
ML=MSC(I,M+1)
CALL MUPAJ (1,1,1,TEST)
IF (TEST.EQ.0) GO TO 200
WRITE (KOUT,2000) (K-K-J,J)
WRITE (KOUT,2001)
LINE=LME(J)
200 WRITE (KOUT,2001) SDML,1,LOC(K1),K=L,KR)
C
300 CONTINUE
C
RETURN
C
2000 FORMAT (6HOLEVEL,2X,1514)
2001 FORMAT (13X,45,2X,1514)
2002 FORMAT (11X)

```

TAB580 1681
TAB580 1682
TAB580 1683

/(1A1511
/11A1511

9000 FORMAT (11A15)
9000 CONTINUE
END

END

SUBROUTINE TART (SA,SB,SC,XL,A,B)
COMMON /JUNK/ MLC(MD,MP,ML,MLB),SS16,61,PL6,41,T12,4)

T12,11=0/XL

T11,11=1.0/T12,11

T11,21=1.0/XL

T12,21=1.0/T11,21

T12,31=1.0/T11,31

T11,41=1.0/T12,41

T12,41=1.0/T11,41

DO 100 I=1,4

01=SA+T12,11+SB+T12,11

02=SB+T11,11+SA+T12,11

DO 100 J=1,4

100 SS(J,1)=01+T11,11+02+T12,21

161SC1 120,120,110

120 SS15,51=SC

SS16,61=SC

SS16,61=SC

DO 100 K=J,JJ

K1=K-J+1

LOC(K1)=LBC(I,K1)

IF (LOC(K1).EQ.NPROP) LOC(K1)=0

100 CONTINUE

END

SUBROUTINE TART (10,SA,SB,SC,XL,D,SS,CA)

COLUMN PANEL STIFFNESS AND FORCE MATRICES

DIMENSION SS(6,6),CALLO(6),T13,6)

DO 10 I=1,3

DO 10 J=1,6

10 T11,J1=0.0

T11,11=1.0/XL

T12,11=1.0/T11,11

T11,21=1.0/T12,21

T12,21=1.0/T11,21

T12,31=1.0/T11,31

T11,41=1.0/T12,41

T12,41=1.0/T11,41

TAB580 1727

TAB580 1728

TAB580 1729

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TAB580 1759

TAB580 1760

TAB580 1761

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TAB580 1763

TAB580 1764

TAB580 1765

TAB580 1766

TAB580 1767

TAB580 1768

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C      T(1,4)=1.0/D
      T(2,4)=T(1,4)
      T(1,3)=--T(1,4)
      T(2,3)=--T(1,4)
C
      T(3,3)= 0.5
      T(4,3)= 0.5
      T(3,4)= 0.5
      T(4,4)= 0.5
C      IF (10.E0.2) GO TO 200
C      COLUMN PANEL STIFFNESS
C
      DO 100 J=1,6
        U=SBOT(1,J)+SBOT(2,J)
        V=SBOT(1,J)+SBOT(2,J)
        W=SCOT(3,J)
        DO 120 I=1,6
          120 SS(I,J)=UOT(1,J)+VOT(2,J)+WOT(3,J)
        RETURN
C      COLUMN PANEL FORCE DISPLACEMENT TRANSFORMATION
C
      DO 200 J=1,6
        C(1,J)=SBOT(1,J)+SBOT(2,J)
        C(2,J)=SBOT(1,J)+SBOT(2,J)
        C(3,J)=SCOT(3,J)
        270 C(4,J)=--C(1,J)+C(2,J)/XL
        RETURN
      END
C
      SUBROUTINE TABV (10,SA,SB,SC,D,L,M,SS,CA)
C      DIAGONAL STIFFNESS AND FORCE MATRICES
C      DIMENSION SS(6,6),C(10,6),T(3,6)
      REAL L
      DO 10 I=1,3
        DO 10 J=1,6
          10 T(I,J)=0.0
      DO=0*0
C
      TABS00 1769 C      T(1,1)=1.
      TABS00 1770 T(1,2)=H/D
      TABS00 1771 T(1,4)=--T(1,2)
      TABS00 1772 T(1,5)=L/D
      TABS00 1773 T(1,6)=--T(1,5)
      TABS00 1774 C
      TABS00 1775 T(2,2)=T(1,2)
      TABS00 1776 T(2,3)=T(1,3)
      TABS00 1777 T(2,4)=--T(1,2)
      TABS00 1778 T(2,5)=T(1,5)
      TABS00 1779 T(2,6)=--T(1,5)
      TABS00 1780 C
      TABS00 1781 T(3,2)=L/D
      TABS00 1782 T(3,4)=--T(3,2)
      TABS00 1783 T(3,5)=H/D
      TABS00 1784 T(3,6)=--T(3,5)
      TABS00 1785 C
      TABS00 1786 IF (10.E0.2) GO TO 200
      TABS00 1787 C      DIAGONAL STIFFNESS
      TABS00 1788 C
      TABS00 1789 DO 100 J=1,6
      TABS00 1790 U=SBOT(1,J)+SBOT(2,J)
      TABS00 1791 V=SBOT(1,J)+SBOT(2,J)
      TABS00 1792 W=SCOT(3,J)
      TABS00 1793 DO 120 I=1,6
      TABS00 1794 120 SS(I,J)=UOT(1,J)+VOT(2,J)+WOT(3,J)
      TABS00 1795 RETURN
      TABS00 1796 C      DIAGONAL FORCE DISPLACEMENT TRANSFORMATION
      TABS00 1797 C
      TABS00 1798 DO 220 I=1,6
      TABS00 1799 C(1,I)=SBOT(1,I)+SBOT(2,I)
      TABS00 1800 C(2,I)=SBOT(1,I)+SBOT(2,I)
      TABS00 1801 C(3,I)=SCOT(3,I)
      TABS00 1802 220 C(4,I)=--C(1,I)+C(2,I)/D
      TABS00 1803 RETURN
      TABS00 1804 C      END
      TABS00 1805
      TABS00 1806 SUBROUTINE TABR (10,SR,MN)
      TABS00 1807 DIMENSION SINM(MN),RENM(1)
      TABS00 1808 COMMON /JUNK/ MLD,M,ND,MM,MN,LM(6),SS(6,6),P(6,4)
      TABS00 1809 GO TO (1,2,3,4,5),10
      TABS00 1810 C      10=1 ZERO MATRICES
      TABS00 1811 C
      TABS00 1812 C
      TABS00 1813 C
      TABS00 1814 C
      TABS00 1815
      TABS00 1816
      TABS00 1817
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      TABS00 1819
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      TABS00 1860

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AD-A107 635

COMPUTERS/STRUCTURES INTERNATIONAL OAKLAND CA
THEORETICAL BASIS FOR CTABS80: A COMPUTER PROGRAM FOR THREE-DIM--ETC(U)
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F/6 13/13

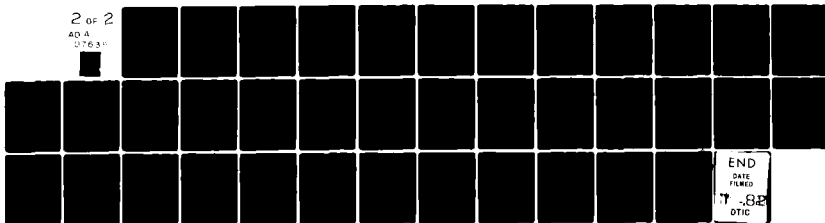
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C      1  DO 200 I=1,MN
C      DO 190 L=1,NLO
190  R(I,L)=0.0
C      DO 200 J=1,MN
200  S(I,J)=0.0
C      GO TO 800
C
C      10=2 SHIFT STORAGE
C
C      2  DO 220 I=1,MN
C      IF=MP+I
C      DO 210 J=1,MN
C      JJ=MP+J
C      S(I,J)=S(I,I,JJ)
C      S(I,I,JJ)=0.0
210  S(I,JJ)=0.0
C      DO 215 L=1,NLO
215  R(I,L)=R(I,L)+S(I,I,JJ)
C      JJ=MP+1
C      DO 220 J=JL,MN
C      S(I,J)=S(I,I,J)
220  S(I,I)=0.0
C      GO TO 800
C
C      10=3 REDUCE MATRIX AND WRITE B.S. EQUATIONS ON TAPE 3
C
C      3  MN=PM-MN-1
C      DO 305 I=1,MN
C      IF (S(I,I)) 305,305,305
305  DO 320 L=1,NLO
320  R(I,L)=R(I,L)/S(I,I)
C      JJ=I+1
C      DO 310 J=JJ,MN
C      S(I,J)=S(I,I,J)
310  S(I,I)=S(I,I)/S(I,I)
C
C      DO 340 K=JJ,MN
C      DO 330 J=K,MN
C      S(I,J)=S(I,J)-S(I,K)*R(K,J)/R(K,I)
340  R(I,J)=R(I,J)-S(I,K)*R(K,I)/R(K,I)
345  CONTINUE
C      WRITE (3) ((S(I,I,J),J=1,MN),I=1,MN),((R(I,L),L=1,NLO),I=1,MN)
C      GO TO 800
C
C      10=4 WRITE LATERAL STIFFNESS MATRIX ON TAPE 2
C      4  IL=MM+1
C      1L=MM-1
C      MS=MM-MM-1
C      LD=((MS+1)+MS)/2
C      WRITE (2) LD,M,((S(I,J),J=1,MN),I=1,IL),((R(I,L),L=1,NLO),I=1,1L)
C      GO TO 800
C
C      10=5 ADD ELEMENT STIFFNESS TO TOTAL STIFFNESS
C      5  DO 550 I=1,ND
C      IL=LR(I)
C      DO 510 L=1,NLO
510  R(II,L)=R(II,L)+P(II,L)
C      DO 550 J=1,ND
C      JJ=LR(J)
550  S(II,J)=S(II,J)+S(II,J)
C      GO TO 800
C      800 RETURN
C      END
C
C      SUBROUTINE TABO (A,M,B,NL)
C      SOLUTION OF SYMMETRICAL LINEAR EQUATIONS- E L WILSON
C      DIMENSION AIM(N),BEM(NL)
C      M=0
C      REDUCTION OF M TH EQUATION
C      50  M=M+1
C      DO 60 L=1,NL
C      60  BEM(L)=R(L)/AEM(P)
C      IF (P-M) 70,130,70
C      DO 80 J=MP,M
C      80  AIM(J)=AIM(J)+BEM(L)*AEM(M)
C      SUMMATION INTO REMAINING EQUATIONS
C      DO 120 I=MP,M
C      IF (A(I,M)) 90,120,90
C      120  A(I,I)=A(I,I)-BEM(L)*BEM(L)

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TAB580 1911
 TAB580 1912
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 TAB580 1954
 TAB580 1955

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IF (MOUT.EQ.0) GO TO 815
CALL MUPAJ (1,1,1,1,1,TEST)
WRITE (MOUT,3001)
LINE=LINE+1
815 DO 850 J=1,NFO
DO 851 K=1,13
851 PFKI=0.0
DO 750 J=1,NST
JJ=301J-13
751 PFKI=PFKI+J*J*N, (J*J*J*J)
750 CONTINUE
C
IF (MURC.EQ.0) GO TO 550
DO 540 K=1,13
WH=PF(K)*PF(K)
IF (WH-LT-TP(K)) GO TO 540
TP(K)=WH
TP(K)=1
540 CONTINUE
C
550 IF (MOUT.EQ.0) GO TO 850
CALL MUPAJ (1,1,1,1,1,TEST)
WRITE (MOUT,3001)
LINE=LINE+1
852 WRITE (MOUT,3002) 1, (LRLAB(K),15), PF(K), K=1,13
WRITE (MOUT,2002)
850 CONTINUE
C
IF (MURC.EQ.0) GO TO 560
JJ=13
IF (JJ.EQ.3) JJ=2
DO 555 K=1,JJ
L= (2*J*J*J*J-15)*J*J-1
L=13*J*J
PERIOD (K)=13+L
555 CONTINUE
C
556 IF (MURC.EQ.0) GO TO 560
IF (MIMP.EQ.0) GO TO 560
LINE=99
CALL MUPAJ (1,1,1,1,1,TEST)
WRITE (MOUT,3005) J, JS, UMCI, GRAY,
* (MOUT,1), MOUT(1), PERIOD(1), UMCK(1), 1-1, 2)
C
960 IF (MOUT.EQ.1) RETURN
C
C TIME HISTORY ANALYSIS
C
IF (MOUT.EQ.0) GO TO 440
READ (1,IMP,9001) SMO,SEMT,MPC,NTIME,SF,FI,DT,IMTYP,MOT
IF (IMTYP.EQ.1) IMTYP=1M0
DO 909 K=1,10
DO 909 K=1,10
IF (SEMT(K).NE.4H) I GO TO 906
909 CONTINUE
SEMT(1)=4H(2F1
SEMT(2)=4H(2F1
SEMT(3)=4H(2F1
SEMT(4)=4H(2F1
SEMT(5)=4H(2F1
SEMT(6)=4H(2F1
SEMT(7)=4H(2F1
SEMT(8)=4H(2F1
SEMT(9)=4H(2F1
SEMT(10)=4H(2F1
906 CONTINUE
C
IF (MIMP.EQ.0) GO TO 720
LINE=99
CALL MUPAJ (1,1,1,1,1,TEST)
WRITE (MOUT,9002) SMO,SEMT,MPC,NTIME,SF,FI,DT,IMTYP,MOT
GO TO 720
C
RESPONSE SPECTRUM ANALYSIS
C
440 IF (MOUT.EQ.3) GO TO 700
701 READ (1,IMP,1000) MPC,MND,SF,FI,DAMP,SHO,
* (MOUT,1), MOUT(1), PA(1), PA(2), 1-1, MPC)
IF (DAMP-LE-1) DAMP=0
IF (DAMP-LE-0) DAMP=0
IF (MND-LE-0) MND=0
IF (MND-LT-0) MND=0
IF (MIMP.EQ.0) GO TO 720
LINE=99
CALL MUPAJ (1,1,1,1,1,TEST)
WRITE (MOUT,2000) SMO,MPC,MND,SF,FI,DAMP
IF (MPC-GE-200) CALL TARR (MPC,26)
WRITE (MOUT,3003)
LINE=LINE+1
DO 710 K=1,MPC
CALL MUPAJ (1,1,1,1,1,TEST)
IF (TEST.EQ.0) GO TO 710
LINE=LINE+3
710 WRITE (MOUT,3004) PA(1), PA(2), 1)
C
720 IF (MOUT.EQ.1) GO TO 704
FI=FI*ATAN(1)/45.
SC(1)=SIMFI
SC(2)=COSFI
C
IF (MOUT.EQ.3) GO TO 730

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230 IF (IOTI-JPIV)30,240,350
240 =IOTI
250 MEPP=MI(J)
MI(J)=0.
IPI=I-I
XII=0.
C SEARCH IN DEPLETED ROW FOR NEW MAXIMUM
C
C DO 320 J=IPI,M
IF (XII-ABS(MI(J))) 300,300,320
300 REI=ABS(MI(J))
IOTI=J
320 CONTINUE
MI(J)=MEPP
350 CONTINUE
C
C XIP(J)=0.
XIP(J)=0.
C CHANGE THE OTHER ELEMENTS OF M
C
C DO 530 I=1,M
IF (I-IPI)370,530,420
370 MEPP=MI(IPIV)
MI(IPIV)=COSINE*MEPP+SI*MI(I-JPIV)
IF (I=IPI) 380,MI(IPIV) 380,390,390
380 REI=ABS(MI(IPIV))
IOTI=IPIV
390 MI(JPIV)=-SI*MEPP+COSINE*MI(I-JPIV)
IF (XII-ABS(MI(JPIV))) 400,530,530
400 REI=ABS(MI(JPIV))
IOTI=JPIV
GO TO 530
C
420 REI=JPIV,30,530,480
430 MEPP=MI(IPIV)
MI(IPIV)=COSINE*MEPP+SI*MI(I-JPIV)
IF (XIP(JPIV)-ABS(MI(IPIV))) 440,450,450
440 IOTI(JPIV)=I
450 MI(I-JPIV)=-SI*MEPP+COSINE*MI(I-JPIV)
IF (XII-ABS(MI(I-JPIV))) 460,530,530
C
480 MEPP=MI(IPIV-I)
MI(IPIV-I)=COSINE*MEPP+SI*MI(IPIV-I-I)
490 IPI(IPIV-I)=ABS(MI(IPIV-I-I))
IOTI(IPIV-I)=I

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```

500 MI(JPIV-I)=-SI*MEPP+COSINE*MI(IPIV-I-I)
IF (XIP(JPIV-I)-ABS(MI(JPIV-I))) 510,530,530
510 XIP(JPIV-I)=ABS(MI(JPIV-I-I))
530 CONTINUE
C TEST FOR COMPUTATION OF EIGENVECTORS
C
C IF (EIGEN)40,540,40
540 DO 550 J=1,M
MI(J)=MI(IPIV)
MI(IPIV)=COSINE*MI(J)+SI*MI(I-JPIV)
550 UI(I,JPIV)=SI*MEPP+COSINE*UI(I,JPIV)
GO TO 40
1000 RETURN
END
C
FUNCTION TABA (MPC,SE,I,PA)
RESPONSE SPECTRUM LINEAR INTERPOLATOR
DIMENSION PA(2,MPC)
DO 100 I=2,MPC
T2=PA(I,I-1)
T3=PA(I,I-1)
IF (T2-T3)11,11,11
CALL TABR (I,T2)
IF (T-LE-T2.AND.-T-GE-T1) GO TO 200
100 CONTINUE
IF (I-CT.PAI1.MPC) TABA=PA(2,MPC)
IF (I-CT.PAI1-1) TABA=PA(2,I)
GO TO 999
C 200 RI=I2-T1/I2-T11
R2=I3-T1/I2-T11
R3=I4-T1/I2-T11
TABA=SE*PA(2,I-1)+R1*PA(2,I)+R2*PA(2,I+1)+R3*PA(2,I+2)
999 RETURN
END
C
FUNCTION COCIV,PU,NEQ)

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C	55C CONTINUE	TAB500 3797	C	CALL TABST (F,CP12,MC),CP14,MC),CP13,MC),CP15,MC), 9,CDP,11	TAB500 3847
C	600 CONTINUE	TAB500 3798			TAB500 3848
C		TAB500 3799			TAB500 3849
C	77C CALCULATE PANEL FORCES	TAB500 3800	C	CALL TABM (R,XM,IMP,STOR,RO,FP,NH,WLD,MLO,MFO,L,3,2)	TAB500 3850
C	77C IF (NPN,EO,0) GO TO 811	TAB500 3801			TAB500 3851
C		TAB500 3802			TAB500 3852
C		TAB500 3803			TAB500 3853
C		TAB500 3804			TAB500 3854
C		TAB500 3805			TAB500 3855
C		TAB500 3806			TAB500 3856
C		TAB500 3807			TAB500 3857
C		TAB500 3808			TAB500 3858
C		TAB500 3809			TAB500 3859
C		TAB500 3810			TAB500 3860
C		TAB500 3811			TAB500 3861
C		TAB500 3812			TAB500 3862
C		TAB500 3813			TAB500 3863
C		TAB500 3814			TAB500 3864
C		TAB500 3815			TAB500 3865
C		TAB500 3816			TAB500 3866
C		TAB500 3817			TAB500 3867
C		TAB500 3818			TAB500 3868
C		TAB500 3819			TAB500 3869
C		TAB500 3820			TAB500 3870
C		TAB500 3821			TAB500 3871
C		TAB500 3822			TAB500 3872
C		TAB500 3823			TAB500 3873
C		TAB500 3824			TAB500 3874
C		TAB500 3825			TAB500 3875
C		TAB500 3826			TAB500 3876
C		TAB500 3827			TAB500 3877
C		TAB500 3828			TAB500 3878
C		TAB500 3829			TAB500 3879
C		TAB500 3830			TAB500 3880
C		TAB500 3831			TAB500 3881
C		TAB500 3832			TAB500 3882
C		TAB500 3833			TAB500 3883
C		TAB500 3834			TAB500 3884
C		TAB500 3835			TAB500 3885
C		TAB500 3836			TAB500 3886
C		TAB500 3837			TAB500 3887
C		TAB500 3838			TAB500 3888
C		TAB500 3839			TAB500 3889
C		TAB500 3840			TAB500 3890
C		TAB500 3841			TAB500 3891
C		TAB500 3842			TAB500 3892
C		TAB500 3843			TAB500 3893
C		TAB500 3844			TAB500 3894
C		TAB500 3845			TAB500 3895
C		TAB500 3846			TAB500 3896


```

120 RETURN
END

SUBROUTINE TABST (F,AA,AV,AT,DD,MLD,COM,ITAG)
C
C CALCULATING STRESSES
C
C DIMENSION F(10,MLD)
C
C DATA TOL /1.E-8/
C
C IF (MLD.EQ.0) GO TO 999
C
C ZF=0.
C IF (DD.GT.TOL) ZF=Z+AT/DD
C
C IF (ITAG.EQ.2) GO TO 200
C
C 100 DO 10 L=1,MLD
C IF (ZF.LE.0) GO TO 15
C F(1,L)=F(1,L)+COM/ZF
C F(6,L)=F(2,L)+COM/ZF
C 15 IF (TAB.GT.0) F(7,L)=F(3,L)+COM/AA
C IF (TAB.GT.0) F(8,L)=F(4,L)+COM/AV
C GO TO 999
C
C 200 DO 20 L=1,MLD
C F(7,L)=0
C F(8,L)=F(1,L)+COM/ZF
C F(9,L)=F(2,L)+COM/ZF
C F(10,L)=F(3,L)+COM/ZF
C 25 IF (AV.LE.0) GO TO 20
C F(9,L)=F(4,L)+COM/AV
C F(10,L)=F(5,L)+COM/AV
C 20 CONTINUE
C
C 999 RETURN
C
END

SUBROUTINE TABG (L,K,C,MJ,VJ,XL,ITAG)
C
C RELEASING FIXED END FORCES FOR PINNED CONDITIONS

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TAB580 4090 C REAL K,L,MJ,MJ
TAB580 4091 C ASSUMES PRISMATIC BEAM
C
C IF (K.EQ.0) GO TO 10
C IF (L.EQ.0) GO TO 10
C IF (M.EQ.0) GO TO 10
C GO TO 999
C
C PINS AT BOTH ENDS
C
C 10 TEMP=(PI*MJ)/XL
C AT=0.
C MJ=0.
C IF (ITAG.EQ.2) GO TO 999
C
C VJ=VI-TEMP
C VJ=VJ+TEMP
C
C 999 RETURN
C
END

SUBROUTINE TABB (F,Q,LDB,RP,IFEF,FEF,COML,M,M,MS,MH,MB,MHP,MFEF,
*
* MCONL,XL,AAA,BBB,AAA,XLL,MAL)
C
C CORRECTIONS TO BEAM MOMENTS AND SHEARS DUE TO SPAN LOADS
C
C DIMENSION LDB(MS,MH,MB,MHP,MFEF)
C DIMENSION IFEF(MFEF),FEF(MFEF),COML(MCONL,MFEF)
C DIMENSION F(10,9)
C DIMENSION Q(4,4)
C
C DO 500 L=1,4
C J=LDB(M,M,L)
C IF (J.LE.0) GO TO 500
C
C F(1,L)=F(1,L)+Q(1,L)
C F(2,L)=F(2,L)+Q(2,L)
C F(3,L)=F(3,L)+Q(3,L)
C F(4,L)=F(4,L)+Q(4,L)
C
C SPAN MOMENT
C
C F(5,L)=.5*Q(1,L)-F(2,L)

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TAB580 4132
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C GO TO 110,20,30,40,50,60
 C 10 J01M=IFIXER
 CALL PLOTS (101M,J01M,RO)
 GO TO 100
 C 20 CALL PLOT (XX,YY,RO)
 GO TO 100
 C 30 CALL SYMBOL (XX,YY,SY,101M,TANT,RO)
 GO TO 100
 C 40 CALL NUMBER (XX,YY,SY,RO,TANT,RO)
 GO TO 100
 C 100 RETURN
 END

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"September 1981."

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Army Engineer Waterways Experiment Station."

"A report under the Computer-Aided Structural Engineering (CASE) Project."

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TA7.W34 no.K-81-2

WATERWAYS EXPERIMENT STATION REPORTS PUBLISHED UNDER THE COMPUTER-AIDED STRUCTURAL ENGINEERING (CASE) PROJECT

	Title	Date
Technical Report K-78-1	List of Computer Programs for Computer-Aided Structural Engineering	Feb. 1978
Instruction Report O-79-2	User's Guide: Computer Program with Interactive Graphics for Analysis of Plane Frame Structures (CFRAME)	Mar. 1979
Technical Report K-80-1	Survey of Bridge-Oriented Design Software	Jan. 1980
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Instruction Report K-80-4	A Three-Dimensional Stability Analysis Design Program (3DSAD): Report 1: General Geometry Module	Jan. 1980
Instruction Report K-80-6	Basic User's Guide: Computer Program for Design and Analysis of Inverted-T Retaining Walls and Floodwalls (TWDA)	Dec. 1980
Instruction Report K-80-7	User's Reference Manual: Computer Program for Design and Analysis of Inverted-T Retaining Walls and Floodwalls (TWDA)	Dec. 1980
Technical Report K-80-4	Documentation of Finite Element Analyses: Report 1: Longview Outlet Works Conduit Report 2: Anchored Wall Monolith Bay Springs Lock	Dec. 1980 Dec. 1980
Technical Report K-80-5	Basic Pile Group Behavior	Dec. 1980
Instruction Report K-81-2	User's Guide: Computer Program for Design and Analysis of Sheet Pile Walls by Classical Methods (CSHTWAL): Report 1: Computational Processes Report 2: Interactive Graphics Options	Feb. 1981 Mar. 1981
Instruction Report K-81-3	Validation Report: Computer Program for Design and Analysis of Inverted-T Retaining Walls and Floodwalls (TWDA)	Feb. 1981
Instruction Report K-81-4	User's Guide: Computer Program for Design and Analysis of Cast-in-Place Tunnel Linings (NEWTUN)	Mar. 1981
Instruction Report K-81-6	User's Guide: Computer Program for Optimum Nonlinear Dynamic Design of Reinforced Concrete Slabs Under Blast Loading (CBARCS)	Mar. 1981
Instruction Report K-81-7	User's Guide: Computer Program for Design or Investigation of Orthogonal Culverts (CORTCUL)	Mar. 1981
Instruction Report K-81-9	User's Guide: Computer Program for Three-Dimensional Analysis of Building Systems (CTABS80)	Aug. 1981
Technical Report K-81-2	Theoretical Basis for CTABS80: A Computer Program for Three-Dimensional Analysis of Building Systems	Aug. 1981