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SOIL-STRUCTURE INTERACTION EFFECTS ON TALL BUILDINGS WITH MAT FOUNDATION

BY

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DECLARATION

I hereby declare that the research work reported in this thesis has been performed by me and that this work has not been submitted elsewhere for any other purpose, except for publications.

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ABSTRACT

Finite element analysis is performed to study soil-structure interaction (SSI) effects on static and seismic response of tall buildings on mat foundations. Simplified numerical models consisting of beam-column elements and lumped masses are used to model building superstructure including mat. For the static analysis the soil is represented by equivalent springs lumped at the foundation nodes. Results from the static analysis show that the mat can be modelled realistically with beam-column elements to represent the thick slabs. It is also found that the superstructure rigidity has some effects on the overall settlements and bending moments in the mat.

For seismic analysis, the soil is modelled by frequency independent springs with viscous damping. Two and three-dimensional models for different building layouts are used. Building height is varied from six to ten stories. The influence of soil flexibility on the response of the mat and the superstructure is studied. Parametric studies are performed to evaluate the effects of different parameters on the behaviour of the building. It is observed that soil-structure interaction, in general, results in reduced base shear under seismic loading. SSI effects not only depend on building configuration, but also are very much influenced by ground motion characteristics. SSI effects are also found to be beneficial for building with mass eccentricity where torsional response is important.

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NOTATION

β	Newmark's operator
q	Vectors of increments of nodal displacement
$\dot{q}$	Vectors of increments of nodal velocity
$\ddot{q}$	Vectors of increments of nodal acceleration
g	Acceleration due to gravity
${}^{1}q$	Vectors of velocity increment
${}^{1}q$	Vectors of acceleration increment
${}^{1}C$	Damping matrix
M	Consistent mass matrix
${}^{2}p$	Nodal load due to applied forces
${}^{1}R$	State of stress
${}^{1}k_{-E}$	Linear stiffness matrix
${}^{1}k_{-E}$	Geometric stiffness matrix
ω	Natural frequency
γ	Newmark's operator
δ	Newmark's operator

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Buildings come in different shapes, forms and sizes, but all of them have at least one feature in common: they all have a foundation. A foundation is the means by which the superstructure interfaces with the underlying soil or rock. Under static conditions, generally only the vertical loads of a structure need to be transferred to the supporting soil or rock. In a seismic environment, the structure is subjected to dynamic horizontal forces; as a result, there will be horizontal forces and moments at the foundation level.

Foundation engineering is inevitably concerned with the interaction of the structure with the underlying soil or rock. The foundation and the geologic formations supporting it are as much a part of the load-carrying system as is the superstructure of the building; this is true whether the foundation is deep or shallow. If a structure is supported by deep foundations, such as piles or caissons, the load is usually transferred to stiff formations. Thus, the deformations are small and their effects on the structure frequently may be disregarded.

However, many structures are supported by soil-bearing foundations that rest on materials which deform significantly under the weight of the structure. Also, the founding soil may deform elastically and inelastically due to reduction in stresses during excavation and then settle to or below its original position as the weight of the structure is imposed upon it. In these events, there is interaction between soil and the structure affecting the deformations and stresses in both.

The soil deformations modify and may control the structural deformations. Redistribution of stresses in the structure affects the loading delivered to the soil and, thus, the deformations of the soil. These, in turn, further modify stress distribution in the structure.

Because the behavior can be so complex, most practical cases have involved discrete analyses of portions of the behaviour, rather than comprehensive treatments of the entire problem. The development of computerized techniques, in particular the finite element method, has made it possible to consider many more aspects of the problem in one analysis. Thus, one can expect that more detailed and sophisticated treatments will be used in the future, but the engineer must resist the temptation to increase the complexity of the analysis at the expense of insight into the mechanics of the problem.

The dynamic interaction of structure and soil can be initiated by a number of phenomena, including vibrations from equipment, blast loading, wind effects and earthquakes. Earthquake effects include most of the problems associated with the other dynamic phenomena and have recently been the most extensively studied.

Building damage resulting from earthquakes may be influenced in a number of ways by the characteristics of the soils in the affected area. In some cases damage is caused by instability of the soil, resulting in large permanent movements of the ground surface, and associated distortion of a structure. For example, deposits of loose granular soils may be compacted by the ground vibrations induced by the earthquake, resulting in large settlements and differential settlements of the ground surface. An island near Valdivia, Chile was partially submerged as a result of the combined effect of tectonic land movements and ground settlement due to compaction in the Chilean earthquake of 1960, while parts of Niigata, Japan were inundated when settlement of ground adjacent a river occurred in the Niigata earthquake of 1964 (Council of Tall Buildings and Urban Habitat, 1980).

In cases where the soil consists of loose saturated granular materials, the tendency to compact may result in the development of excess hydrostatic pressures of sufficient magnitude to cause liquefaction of the soil, resulting in settlements and tilting in structures. Liquefaction of loose saturated sand deposits resulted in major damage to hundreds of buildings in Niigata, Japan during the earthquake of 1964 (Council of Tall Buildings and Urban Habitat, 1980).

The combination of dynamic stresses and induced pore water pressures in deposits of soft clay and sands may result in major landslides such as what developed in the Turnagain Heights area of Anchorage, Alaska, in the earthquake of March 27, 1964 (Council of Tall Buildings and Urban Habitat, 1980). The coastline in this area was marked by bluffs some 21 m (70 ft) high sloping at about 1 on 1-1/2 down to the bay. The slide induced by the earthquake extended about 3 km (almost 2 miles) along the coast and extended inland an average distance of about 270 m (900 ft). The total area within the slide zone was thus about 526000 m^2 (130 acres). Within the slide area the original ground surface was completely devastated by displacements which broke up the ground into a complex system of ridges and depression. In the depressed areas the ground dropped an average of 10.6 m (35 ft) during the sliding. Houses in the area, some of which moved laterally as much as 150 m to 180 m (500 ft to 600ft) as the slide progressed, were completely destroyed. Major landslides of this type have been responsible for much damage and loss of life during earthquakes.

While these types of soil instability may cause catastrophic damage to buildings, they can be avoided or prevented by appropriate foundation investigations and design. On the other hand, the dynamic response of structures to ground vibrations, which also depends to a large extent on the soil conditions at the site, cannot be avoided. This study encompasses building and ground responses during earthquakes under conditions where no soil instability or permanent deformations are involved.

The finite element method of dynamic soil-structure interaction analyses has been developed largely in conjunction with the design of nuclear power plants and gravity dams. It has now achieved substantial sophistication, and there are many papers describing both the details of the procedure and the numerous subtleties necessary in its use. In general, the use of dynamic finite element methods follows lines similar to the static finite element methods.

Tall buildings, because of their height, stiffness and usually urban location, deserve special consideration. Most tall buildings have basements with heavy column loads. Tall buildings may be subjected to large lateral loads from wind or earthquake. Great attention must be paid to the deformation which will occur under these lateral loads to assume compatible performance of various elements of the structure and to limit deformation to acceptable amounts.

The fundamental principle of soil-structure interaction requires interactions between the foundation engineer and the structural engineer. When each is aware that the properties of his portion of the building can affect the loads and deformations of in the other, complicated problems of foundations on soft soil can be approached rationally and efficiently. Problems and distress arise when the foundation engineer simply takes his loads as given to him by the structural engineer or the structural engineer, in his analyses, assumes that all the loads and structural deformations are to be carried by and infinitely stiff material in the foundation.

As described by Lambe and Whitman (1969), the role of the geotechnical engineers is that of a moderator or optimizer in between two mutually conflicting interests. One is imposed by the structural engineer who will be happy to have zero settlement so that the structural elements are not additionally stressed. The other is from the owner who wants satisfactory structural performance at the lowest cost. In this light, the necessity to provide safety and economy at the same time is evident, which can be done only with clear understanding of the problem and knowledge about the true behavior of the system under the loading situations considered. Soil-structure interaction is one of the aspects of considerations for obtaining more realistic solutions of the problem.

This aspect is more important for engineers designing tall structures in Bangladesh. Although mat foundations are used for most structures, they are traditionally designed as rigid slabs without considering the possible soil-structure interaction effects. With proper considerations these building may be designed more economically.

Bangladesh is geographically located close to the active seismic region of the Himalayan range. Within the last 150 years, several major earthquakes with magnitudes of 7.0 or higher in the Richter scale have occurred in the region (Ali and Choudhury, 1994). Table 1.1 provides a list of these earthquakes along with the locations of their epicenters. There are active faults within and in the neighboring regions that may be sources of major earthquakes any time in the future (Fig 1.1). Table 1.2 shows the probable magnitudes of operational basis earthquakes and maximum credible earthquakes, along with the depths of foci in these fault zones.

In the Bangladesh National Building Code (HBRI, 1993), the country is divided into three seismic zones (Fig 1.2) for design and damage considerations, The zones have been identified from earthquake magnitudes, return periods and acceleration attenuation relationships (Ali and Choudhury, 1994). Zone 3 is the region of most severe earthquakes and zone 1 is that of the least severe one. The figure clearly shows that half of the country is under the possible threat from earthquakes of moderate to high magnitude.

For economically designing tall structures that will remain safe and serviceable after a earthquake, it is essential to have a comprehensive understanding of the response of such structures under earthquake loads, considering the soil-structure interaction effects. With more and more computational power and facilities available at moderate costs, the structures of the future should be analyzed and designed in a more accurate and efficient manner.

Table 1.1 List of Major Earthquakes Affecting Bangladesh (After Ali and Choudhury, 1992)

Date	Name of Earthquake	Magnitude (Richter)	Epicentral Distance form Dhaka (km)
10 January, 1869	Cachar Earthquake	7.5	250
14 July, 1885	Bengal Earthquake	7.0	170
12 June, 1897	Great Indian Earthquake	8.7	230
8 July, 1918	Srimongal Earthquake	7.6	150
3 July, 1930	Dhubri Earthquake	7.1	250
15 January, 1934	Bihar-Nepal Earthquake	8.3	510
15 August, 1950	Assam Earthquake	8.5	780

Table 1.2 Tectonic Provinces and their Earthquake Potential (After Ali and Choudhury, 1992)

Location	Operating Basis Magnitude (Richter)	Maximum Credible Magnitude (Richter)	Depth of focus (km)
Assam fault zone	8.0	8.7	0-70
Tripura fault zone	7.0	8.0	0-70
Sub-Dauki fault zone	7.3	7.5	0-70
Bogra fault zone	7.0	7.5	0-70

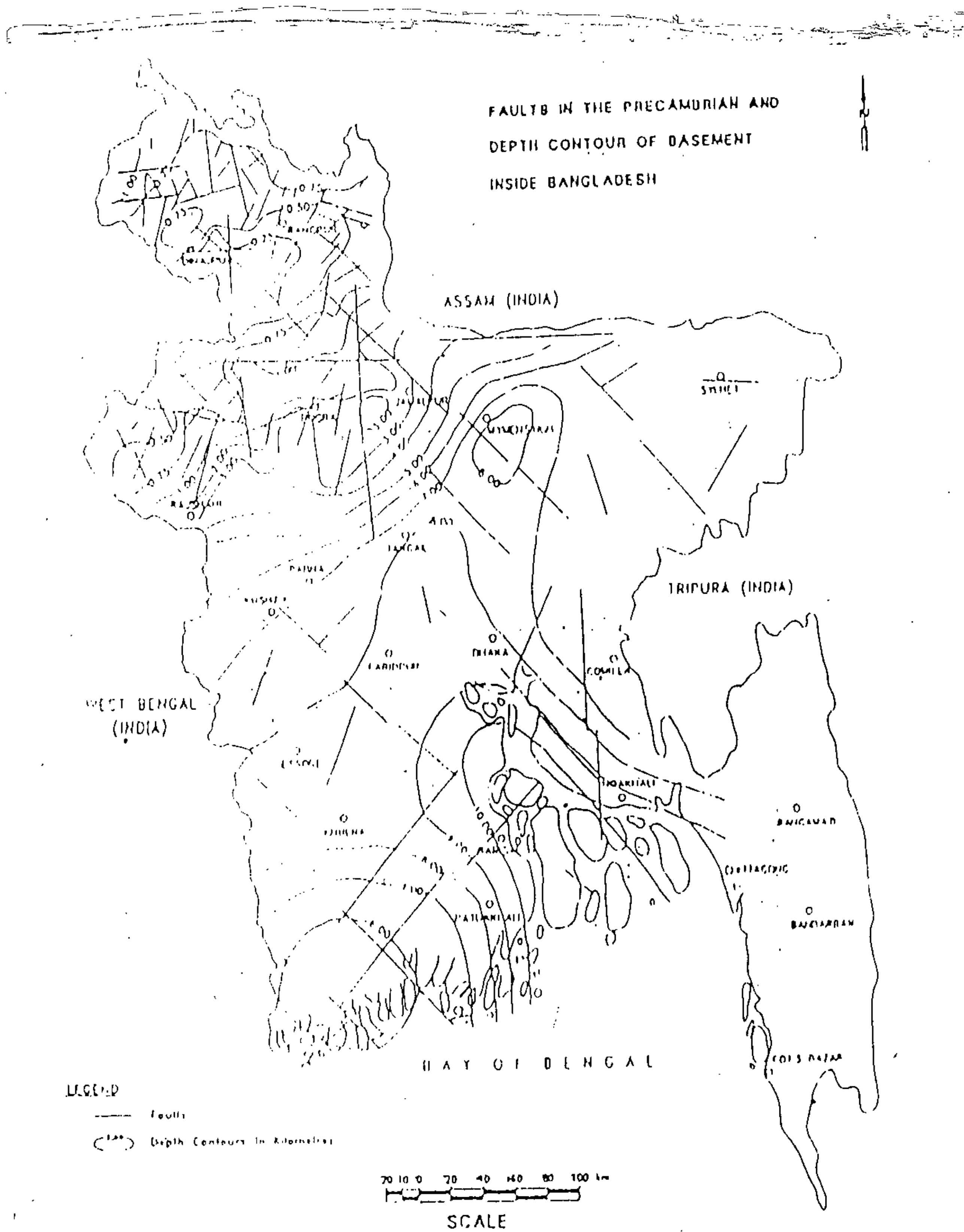


Fig 1.1 Faults in the Precambrian and Depth Contour of Basement Inside Bangladesh

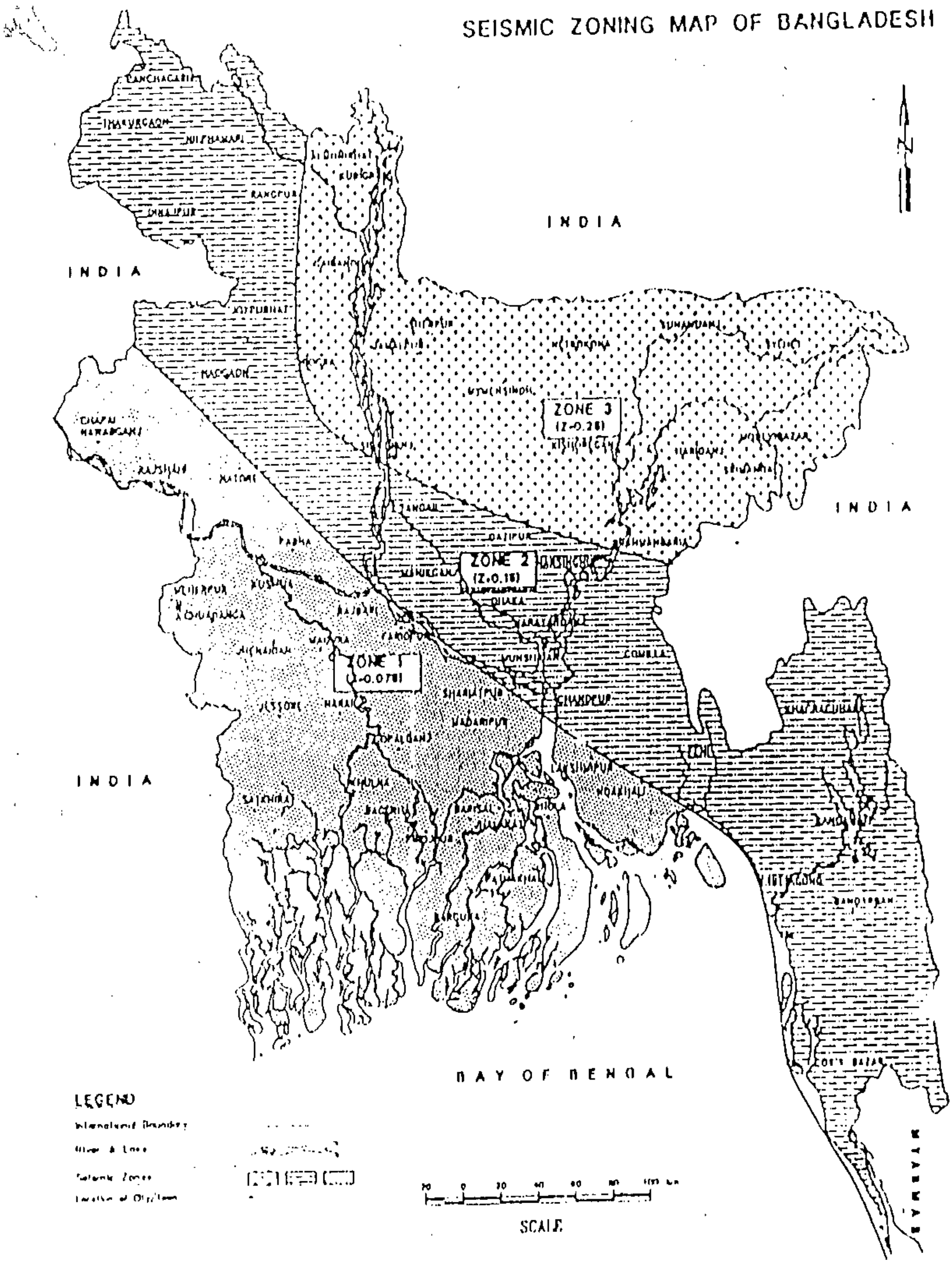


Fig 1.2 Seismic Zoning Map of Bangladesh

1.2 OBJECTIVES OF THE RESEARCH

The principle objectives of the present research are:

- To conduct a literature survey on available information on static and dynamic soil-structure interaction effects with particular reference to tall buildings and mat foundation.
- To examine the effects of soil-structure interaction on the stresses and deflections of the mat foundation for static loading.
- To perform dynamic analysis of buildings subjected to a number of earthquakes of different magnitude and frequency content.
- To investigate the effects of soil-structure interaction on the seismic response of tall buildings on mat foundations through time-history analysis.
- To obtain the natural frequencies and mode shapes of buildings through modal analysis.
- To examine the effects of different building parameters on the seismic behavior of the building.

1.3 SCOPE OF WORK

This research work involves numerical study of tall buildings with mat foundations. Finite element analysis is performed to study the soil-structure interaction (SSI) effects on the response of the frame buildings on mat foundations. A simplified model consisting of beam-column elements and lumped masses is used to model the building superstructure including mat. Linearly elastic behavior of the beam-column elements is assumed. Both static and dynamic analyses are perform to obtain the building response in the two conditions. The static analysis is performed with the model of mat foundation with and without the superstructure. The soil is represented by truss elements used as springs. The soil springs are considered to be frequency independent and with viscous damping.

The dynamic analysis is performed for both two and three-dimensional models with two different layouts. Building heights are varied from six to ten stories. Results are also presented for an eight-story building having torsional response due to mass eccentricity.

Both time-history and modal analysis are preferred. The building models are subjected to five different earthquakes for the time history analysis.

1.4 ORGANIZATION OF THE THESIS

The results of the research are presented here under different topics and are divided into seven chapters.

A brief introduction to the problem of designing buildings and foundations with consideration of possible earthquake effects is given in chapter 1. Special reference is drawn to the situation in Bangladesh considering the earthquake risks involved. The objectives and scope of the present study are also outlined.

Chapter 2 illustrates the concept of soil-structure interaction and the typical design philosophy of the building-soil systems. The recommendations of ACI committee 336 are discussed for possible incorporation of the SSI effects in the design process.

The third chapter discusses the dynamic soil-structure interaction phenomena. Theories and approaches behind the analysis of seismic response of multistory buildings with appropriate considerations are described.

The methodology for the numerical analysis is given in Chapter 4. The details of the model, loading criteria and the analysis schemes for both the static and dynamic analyses are presented.

In Chapter 5 the results of the static analysis of the foundation-mat-superstructure system is presented. The building model and effects of several parameters are described.

Soil-structure interaction effects on the seismic response of tall buildings are discussed in detail in Chapter 6. The numerical modeling, analysis options and results of the parametric studies are presented. Verification of the results is done through comparison with a more rigorous finite-element analysis.

The conclusions of the entire study and some recommendations for future research are given in Chapter 7.

CHAPTER 2

STATIC SOIL-STRUCTURE INTERACTION ANALYSIS

2.1 GENERAL

The vast majority of soil-structure interaction problems involve behavior under static loads. Therefore, most of the work that has been done over the years has concerned static problems. The calculation of rate of occurrence of time-dependent effects such as consolidation and creep is not always very accurate. Regardless of how time-dependent effects are treated, the engineer must represent the compliance of the soil in some way. The different approaches to soil-structure interaction can be distinguished by the ways they represent the behaviour of the soil and the ways they solve the resulting mathematical problem.

2.2 ANALYTICAL METHODS

One major category of methods is based on the assumption that the foundation soils can be represented adequately by a simple stress-strain law and theories from continuum mechanics. One of the earliest and simplest of such methods assumes the soil can be replaced by a set of discrete springs, each acting independently of the other. This is attributed to Winkler (1867). The vertical stress and vertical motion are linearly related

$$\delta_v = \sigma_v / k_s \tag{2.1}$$

In which k_s = modulus of subgrade reaction

Extensive literature has grown up around this approach, in part because it is easy to modify existing procedures for structural analysis to incorporate the Winkler springs.

Hetenyi (1946) in particular has presented useful and comprehensive results. He presented some closed form solutions for problems related to beam on elastic foundation. Through his solutions, moments, shears and deflections can be obtained for particular cases of loading. Terzaghi (1955) described how the values of k_s are affected by the soil properties and the geometry of the loading. He also provides empirical values for various materials and conditions. It should be noted that his values are conservative in the sense that they tend to give low values of modulus of subgrade reaction and high values of settlement. If the problem requires computing loads that would cause prescribed settlements, his values may lead to an underestimate of the loads.

An alternative, in many ways the preferred method, is based on the assumption that the soil is a semi-infinite, linearly elastic half space. Using an assumed or estimated distribution of structural loads to the soil, deformations of the soil at founding level can be computed. Both elastic and time-dependent deformations can be considered by using appropriate modulus of deformation. From the deformations so determined and the structural stiffness, redistribution of stresses in the structure and revised load distributions to the soil can be computed. Then, further revised deformations of the soil at founding level can be determined.

Graphs and charts of influence factors for determining added stresses in the soil mass for a wide variety of loading have been developed by Newmark (1947) and Fadum (1948). Using these, soil stresses may be readily determined. The procedures are based on integration of the Boussinesq solution for stresses in semi-infinite elastic solid.

The solution for vertical stresses is not sensitive to lateral variations in soil properties. The procedure may be used for stratified soil by considering appropriate modulus of deformation for each stratum. Thus, for many structures the effects of soil-structure interaction on both structure and soil may be developed using simple calculations amenable to solution with hand calculators.

The approach is quite useful for manual checking of results of more sophisticated analyses. Also, in many problems, precision in computation is not warranted, as, for example, when structures are relatively flexible and stress redistribution is not sensitive to settlement; where structural stiffness, including effects of walls and partitions, can only be approximated; or where soil properties are not known in detail.

Either the Winkler spring approach or the semi-infinite elastic half-space approach (also called the Boussinesq approach) can be used in analysis of soil-structure interaction. Both can be and have been incorporated in computer programs for structural analysis. The Winkler spring approach is easier to program because the behaviour can be represented by a simple spring of elastic reaction at each nodal point of the foundation, but the Boussinesq requires coupling between the response at one point and another. On the other hand, the Boussinesq approach is clearly superior theoretically. It should be used when the foundation soils are relatively stiff, because such soils tend to behave as a semi-infinite half-space rather than as a set of independent springs. Studies by Gibson (1967) have shown that for certain nonhomogeneous distribution of modulus, the Winkler spring method may be superior to the Boussinesq method.

Some other researchers have also worked on the problem. Cheng and Nag (1968) and Bowles (1974) performed finite-element analysis of beams and plates on elastic foundation while Desai, et al (1982) worked on the mechanics of 3D soil-structure interaction. Shukla (1984) also proposed a simplified design method of mats on elastic foundations.

The application of soil-structure interaction in multistory buildings has also been investigated by for quite some time. Goschy (1978) studied the soil-structure interaction in multistoried buildings with an intention to increase structural and economic efficiency in the design of basements. In recent years there have been several more successful attempts to use foundation compliance functions to model complicated foundation behaviour. In particular Focht et al. (1971) have described the procedures used for predicting the behaviour of a foundation for a high-rise building located on soft soils. They used a modified version of the Boussinesq approach, developing the "elastic" properties of the soil by means of empirical data and consolidation tests on the material. Thus, the analysis does not so much represent an elastic analysis as it does the engineer's best estimate of the modulus of deformation that are appropriate for this case. This case illustrates that the intelligent cooperation between the foundation engineer and the structural engineer can result in a substantial improvement in the design of a building. It also shows that the values of modulus that are used in such analysis need not be truly elastic properties but can be modulus of deformation.

With the widespread applications of more sophisticated analysis and design tools, the design process gets increasingly extensive. More rigorous analysis of buildings and foundations with all possible practical considerations become feasible and often desirable (Ball and Notch, 1984). Reports and guidelines for analysis and design procedures for footings and mats have also suggested considering soil-structure interaction in the analysis and design process. ACI committee 336 (1988) recommends to include such considerations in the design of combined footings and mats.

2.3 NUMERICAL METHODS

When the properties of the soil or the geometry of the problem becomes too complicated to be handled by analytical approaches, numerical methods are necessary. It is now easily possible to represent the behaviour of the soil by numerical methods, in particular the finite element method.

The finite element method allows the engineer to divide the foundation soil into a large number of discrete elements and to describe the properties of the material in each one of these elements by different stress-strain relations. Thus, the complete, nonlinear time-dependent problem could, in theory, be analysed. In practice, such analyses are not carried out for several reasons. In the first place, the analysis would be expensive, especially because most buildings involve a truly three-dimensional geometry, which cannot be adequately represented in plain strain. In the second place, it is very difficult to determine material properties that would go into such analyses, and one doubts whether the results would be any more accurate that those that could be obtained by simpler methods.

Thus, the usefulness of finite element methods is restricted to two classes of problems:

1. Those problems in which the geometry of the soil and its material properties are reasonably well known and where the geometry of the nonlinear nature of these properties is considered to have a potentially significant effect on the behaviour of the structure

2. Those problems in which a particular aspect of the foundation performance must be studied in great detail.

2.4 DESIGN PHILOSOPHY FOR MATS

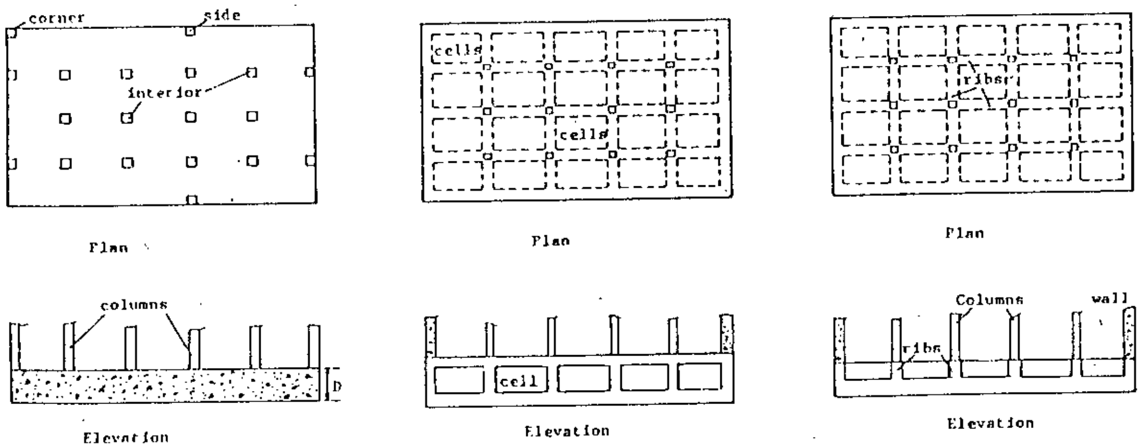
Mat foundations are commonly used on erratic or relatively weak subsurfaces where a large number of spread footings would be required and a well-defined bearing stratum for deep foundations is not near the foundation base. Often, a mat foundation is used when spread footings cover more than one-half the foundation area. A common mat foundation configuration is shown in Fig. 2.1(a).

The flexural rigidity EI of the mat may be of considerable aid in the horizontal transfer of column loads to the soil and may aid in limiting differential settlements between adjacent columns. Structure tilt may be more pronounced if the mat is very rigid. Load concentrations and weak subsurface conditions can offset the benefits of mat flexural stiffness.

Mats for buildings are usually beneath a basement that extends at least one-half story below the surrounding grade. Additionally, the top mat surface may function as a basement floor. Depending on the structure geometry and weight, a mat foundation may "float" the structure in the soil so that settlement is controlled. In general, the pressure causing settlement in a mat analysis may be computed as

Net pressure = {[Total structure weight (including mat)] - Weight of excavated soil} / Mat area

Part of the total structure weight may be controlled by using cellular mat construction, as illustrated in Fig. 2.1(b). Another means of increasing mat stiffness while limiting mat weight is to use inverted ribs between columns in the basement area as in Fig. 2.1(c). The cells in a cellular mat may be used for liquid storage of to alter the weight by filling or pumping with water. This may be of some use in controlling differential settlement or tilt.



(a) Solid mat of reinforced concrete; most common configuration. D = depth for shear, moment or stability and ranges from about 1.5 to 6^+ ft (0.5 to 2^+ m).

(b) Mat using cell construction. Cells may be filled with water or sand to control settlements or for stability.

(c) Ribbed mat used to control bending with minimum concrete. Ribs may be either one or two-way.

Fig. 2.1—Mat configurations for various applications

2.4.1 ANALYSIS AND DESIGN PRINCIPLES

Mats may be analyzed and designed as either rigid bodies or as flexible plates supported by an elastic foundation (the soil). An exact theoretical design of a mat as a plate on an elastic foundation can be made; however, a number of factors rapidly reduce the exactness to a combination of approximations. These include:

1. Great difficulty in predicting subgrade responses and assigning even approximate elastic parameters to the soil.
2. Finite soil-strata thickness and variations in soil properties both horizontally and vertically.
3. Mat shape.
4. Variety of superstructure loads and assumptions in their development.
5. Effect of superstructure stiffness on mat (and vice versa).

With these factors in mind, it is necessary to design conservatively to maintain an adequate factor of safety. The designer should work closely with the geotechnical engineer to form realistic subgrade predictions, and not to rely on values from textbooks.

Many structural engineers analyze and design mat foundations by computer using the finite element method. Soil response can be estimated by modeling with "soil springs". The spring properties are usually calculated using a modulus of subgrade reaction, adjusted for mat size, tributary area to the node, effective depth, and change of modulus with depth. Caution should be exercised when using finite element analysis for soils. Without good empirical results, soil springs derived from values of subgrade reaction may only be a rough approximation of the actual response of soils. Some designers perform several finite element analyses with soil springs calculated from a range of subgrade modulus to obtain an adequate design.

A large number of commercially available computer programs are available that can be used for mat analysis. But the program user remains responsible for the design. A program should be used that the designer is most familiar with or has investigated sufficiently to be certain that the analyses and output are correct.

A mat may be designed using either the Strength Design Method (SDM) or working stress design according to the Alternate Design Method (ADM) of ACI 318-83, Appendix B. The ADM is an earlier method, and most designers prefer to use the SDM.

Computer analysis of mat foundations is usually based on a discrete element formulation of the mat. There are three discrete element formulations, which may be used, namely Finite Difference Method (FDM), Finite Grid Method (FGM) and Finite Element Method (FEM). Computers and available software make the use of any of the discrete element methods economical and rapid.

2.4.2 ACCEPTABLE DESIGN CRITERIA

A number of criteria have been proposed to restrict total and differential settlement and angular distortions within tolerable limits. The definitions of these values are presented in Fig. 2.2. Tolerable values of total and differential settlement and angular distortions for mat foundations recommended by different codes are presented in Table 2.1 (Kabir, et. al, 1993).

ACI Committee 336 (1988) has recommended the acceptable values of differential settlement as a function of relative stiffness K_r, which is represented by the following equation.

$$K_r = \frac{EI_b}{E_s B^3} \tag{2.2}$$

Where EI_b may be taken as

$$EI_b = EI_f + \sum Eb_i + \sum \frac{Eah^3}{12} \tag{2.3}$$

where

EI_b = Flexural rigidity of the superstructure and footing

E_s = Soil modulus

B = Base width perpendicular to the direction of interest

EI_f = Flexural rigidity of the footing

Eb_i = Rigidity of members making up the frame resistance perpendicular to B.

$Eah^3/12$ = Effective rigidity of shear walls perpendicular to B.

The committee recommendation of expected values of differential settlement as a function of K_r is presented in Table 2.2.

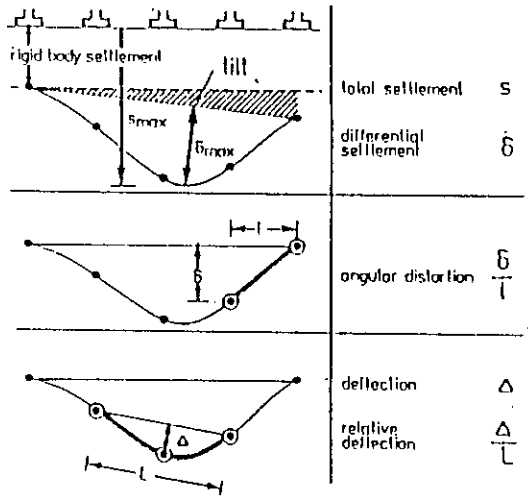


Fig. 2.1 : Definition of Settlement and Distortion

Table 2.1 Tolerable Settlements and Distortions for Mat Foundations in Sands (After kabir, et al, 1993)

Value	Range	Average
Angular Distortion	1/150 to 1/500	1/300
Differential settlement (mm)	18 to 31	25
Total settlement (mm)	50 to 100	75

Table-2.2 ACI committee 336 (1989) Recommendation on differential settlement

K_r (stiffness ratio)	Differential settlement/ Total settlement
0	0.5 for Long base 0.35 for square base
0.5	0.1
>0.5	Rigid mat: no differential settlement

2.4.3 EFFECT OF SUPERSTRUCTURE RIGIDITY

This factor tends to restrict the free response of the mat to the soil deformation. Redistribution of reactions occurs within the superstructure frame as a result of its stiffness, which reduces the effects of differential settlements. This must be considered together with the mat stiffness to evaluate the validity of stresses computed on the basis of foundation modulus theories. Also, such redistribution may increase the stresses in elements of the superstructure.

G. G. Mayerhof (1947) was the first to recognize the importance of superstructure rigidity in the foundation design and its influence on contact pressure distribution and indicated a simple but very approximate method for including its effect in the design of foundations. Sommer (1965) thoroughly studied of the general problem of superstructure foundation interaction and to suggest a method to include its effects in foundation design. Later Lee and Harrison (1970) and Lee (1975) suggested different methods for solving this problem. The ACI committee 336 (1988) has given considerable emphasis on inclusion of soil structure interaction in proper designing of mat foundations. This has also been emphasized by a number of researchers like Focht, et al. (1978).

CHAPTER 3

DYNAMIC SOIL-STRUCTURE INTERACTION AND SEISMIC ANALYSIS

3.1 GENERAL

In the normal dynamic analysis of a building, the usual method is to determine the response of the building for the free-field ground motion applied at the base of the building, assuming the base to be fixed. In reality, the presence of the structure will modify the free-field motion as the soil and the structure interact to create a dynamic system which will result in a structural response that may be quite different from the response computed from a fixed-base building subjected to a free-field ground motion. Almost all earthquake engineers agree that soil conditions have a great deal to do with damage to structures during earthquakes. The large magnitude of the potential effect of soil conditions on building response merit its careful consideration in evaluating the response of tall buildings during earthquakes.

3.2 DYNAMIC CHARACTERSTICS OF SOIL-STRUCTURE SYSTEMS

The effect of soil on dynamic response of buildings can be divided into two areas: amplification and interaction. Amplification refers to the effects of the layers of soil on the earthquake signal before it reaches the building. Amplification effects would occur regardless of whether the building were present. Interaction property refers to the effect of presence of the building on the motions. In other words, the building and soil together are a more complicated system than the building alone on a rigid foundation or the foundation soil alone without a structure on it. The combination of soil and structure may have a different dynamic behavior from that of the structure alone.

Studies on past earthquakes have shown that the greatest damage occurred where the fundamental period of the structures coincided with the fundamental period of the soil beneath the building. Therefore, an early question to be asked by the designer is whether, considering both structure and soil, the structure has a vibratory frequency that corresponds to one of the characteristic frequencies of the foundation strata. Removing the structure from this condition of resonance will decrease the major effect of the foundation on the structural response. But it should be also noted that calculations of amplification effects and of the corresponding resonant periods involve a considerable range of uncertainty and careful judgement is required from the engineer's part in making any prediction of the behavior of such systems.

The interaction effects between a tall building and the foundation soil on which it rests may be divided into two parts:

1. Physical interaction effects which involve the effects of stresses and deformations at the contact boundaries between structure and soil. Potential consequences of such effects include a change in ground response adjacent to the building, changes in period of the building or in deformations of the upper floors of the building resulting from rocking deformations of the underlying soil, and changes in response of the building due to soil deformations.

2. Response interaction, involving changes in response of a given type of structure as a result of changes in the response of different soil deposits to earthquake-induced motions in the underlying rock.

3.3 DETERMINISTIC ANALYSIS OF EARTHQUAKE RESPONSE

The only special feature of the earthquake problem, compared with any other form of dynamic loading, is that the excitation is applied in the form of support motions rather than by external loads. Thus the essential subject of seismic analysis is the methods of defining the effective external-load history resulting from a given form of support motion.

Earthquake ground motions usually are expressed in terms of three components of translational accelerations. The response of any linear system to these three components of input can be computed, of course, by superposing the responses calculated separately for each component. Thus the standard analytical problem is reduced to the evaluation of the structural response to a single component of support translation.

In a more general case, the support point will be subjected to rotations in addition to the translational motions, as the earthquake waves propagate through the foundation soils. Thus a complete description should, in principle, include three components of support rotations as well as translations.

Another assumption inherent in the usual treatment of earthquake excitations is that the same motion acts simultaneously at all parts of the structure's foundation. If rotational motions are neglected, this assumption is equivalent to considering the foundation soil or rock to be rigid. Such a hypothesis is not consistent with the concept of earthquake waves propagating through the earth's crust from the point of fault rupture. However, if the base dimensions of the structure are small relative to the vibration wavelengths, the hypothesis is acceptable. But large structures like suspension bridges or dams would be subjected to drastically differing motions along their lengths which can contribute significantly to the dynamic response stresses. Therefore, for such structures, it is important to develop analysis procedures capable of dealing with multiple support excitation, that is, with different earthquake inputs applied at separate points of support.

One final factor that should be considered in defining the effective forces developed in a structure by an earthquake is that the ground motions at the base of the structure may be influenced by the motions of the structure itself. In other words, the motion introduced at the base of the structure may be different from the free-field motions that would have been observed without the structure. This effect will be of no importance if the foundation rock is firm and the building is relatively flexible. In this case, the structure can transmit little energy into the soil, and free-field motion is an adequate measure of the foundation displacements. On the other hand, if a heavy, stiff structure (such as a nuclear-reactor power station) is supported on a deep, soft soil layer, considerable energy will be transferred from the structure to the soil and the base motions may differ drastically from the free-field conditions. This soil-structure interaction effect is independent of and in addition to the effect that the soil layer might have on the characteristics of the free-field motions. In general, both the ground-motion modification and the interaction effects of a soft surficial soil layer can be important and must be accounted for in an earthquake-response analysis.

The deterministic earthquake-response analysis of various types of structural systems, considering successively each of the above different input conditions, include: (1) simple single-component translation of the base, (2) rigid-base rotations, (3) relative movements of different support points, and (4) the case of soil-structure interaction where the motion of the base of the structure does not directly follow the specified free-field motion.

The dynamic soil-structure interaction phenomena has been studied by a good number of researchers over the past few decades. Parmalee (1967) was the first to look into the building-foundation interaction effects for dynamic and seismic loading. Seed, et al (1975) investigated the SSI effects on seismic response of nuclear power plants. The DSSI effects have also been examined for other types of loading other than earthquakes. Dasgupta and Rao (1978), for example, performed finite-element analysis to get further insight into the dynamics of machine foundations.

There have also been some attempts to make some seismic design provision for dynamic soil-structure interaction. Veletsos (1983) made the first known attempt to incorporate the effects of soil-structure interaction in design codes. He also presented simplified analysis procedures to account for soil structure interaction (Veletsos, et al, 1988).

The dynamic soil-structure interaction effects on tall buildings have been investigated by Agarwal (1983), Ellis (1986) and others. In more recent times, these effects have been studied from recorded motions of instrumented buildings during earthquakes (Celebi and Safak, 1990; Safak, 1990).

3.4 MODELS AND ANALYSIS METHODS FOR DYNAMIC SOIL-STRUCTURE INTERACTION

Both the soil deposit and the structure at a given site respond to the motions in the underlying rock, and analytical models have been developed for evaluating these responses. The most sophisticated of these is the finite-element representation shown in Fig. 3.1 (a), which permits a 2- or 3- dimensional analysis of the complete soil-structure system. This involves dividing the soil into discrete elements and solving for the dynamic behavior of the resulting multiple degree-of-freedom system.

A somewhat simplified model is the lumped mass representation as shown in Fig. 3.1 (b). The lumped springs technique involves replacing the soil and foundation by one or more springs, masses, and

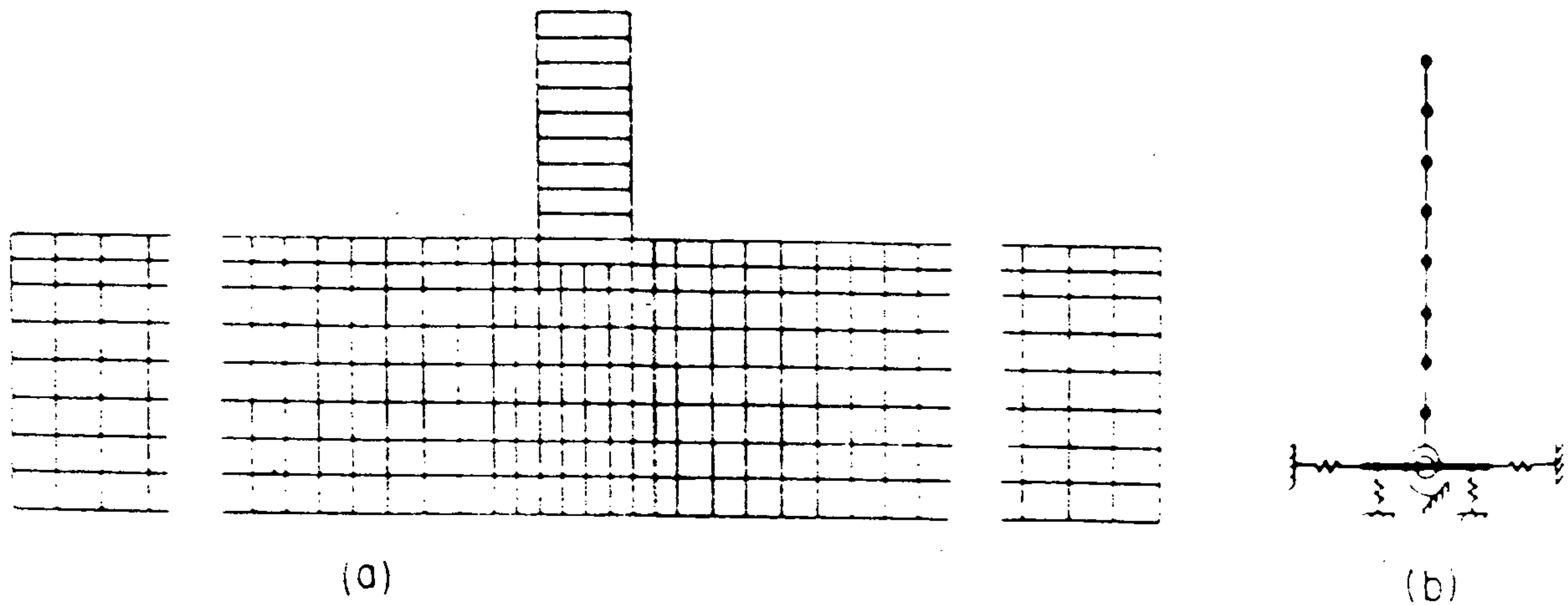


Fig. 3.1 Analytical models for soil-structure systems: (a) Finite element model; (b) Lumped mass-spring model

dashpots, whose properties are selected to represent the soil and the frequency range of interest. Relatively simple "equivalent springs" may be used, or complex, frequency dependent compliance functions can be developed. But the behavior of the system is very sensitive to the component characteristics in the lumped mass representation for a building and the foundation soil.

An important limitation of the discretized model of the foundation medium is that such models necessarily have finite boundaries. Where natural physical boundaries exist, such as hard rock underlying soft surficial soil deposit, the extent of the model will be obvious and its boundary behavior can be expected to simulate the prototype system adequately. On the other hand, if the structure is founded on a broad, deep, and uniform soil mass, the model boundaries tend to retain the vibration energy within the system, and unless they are at a great distance, they will inhibit the radiation energy loss from the structure. Under these circumstances, a very large number of degrees of freedom may be required to simulate the behavior of the soil system adequately. Frequently the number of degrees of freedom in the foundation far exceeds the number representing the structure, which is the real subject of the investigation. Thus the overall efficiency of this type of model may be very poor.

Foundation "springs" for dynamic analyses are intended to represent the behavior of the foundation as a whole and are not just discrete representations of pieces of the foundation as those used in static analyses. Therefore, the selection of the foundation springs requires judgement. Most selection starts from the theory of behavior of a footing dynamically excited on the surface of a half-space. Spring constants are selected from the static spring for an equivalent static condition. When the foundation is not at the surface or when the soil is not very deep, these models must be modified, but the theoretical difficulties increase enormously. Another complication is introduced when the soil underlying the foundation is layered or when there is stiff bedrock reasonably close to the foundation.

In cases where the foundation medium is rather uniform over a broad, deep zone and where the contact surface between the supported structure and the soil deposit may be considered as a rigid plate, this limitation of the discretized model can be avoided by treating the soil as an elastic half space. The function of the foundation medium is to resist the forces applied to it by the base of the building. During an earthquake, a rigid base slab may be subjected to displacements in six degrees of freedom, and the resistance of the soil may be expressed by the six corresponding resultant-force components. Hence the structural behavior of the elastic half space is represented completely by a set of force-displacement relationships defined for these degrees of freedom.

To simulate the static behavior of the soil-structure system, it is evident that the foundation medium could be modeled by six linear springs acting in the rigid-base degrees of freedom. Appropriate static spring constants can be evaluated for the elastic half space by the methods of continuum mechanics. However, the dynamic resistance of soil mobilized during an earthquake includes inertial and damping effects in addition to the static spring stiffness. The appropriate dynamic-force-displacement relationships of an elastic half space subjected to a harmonic excitation of the rigid building can also be evaluated by methods of continuum mechanics, and the results show that both the relative response amplitude and the phase of the resisting forces are functions of the frequency of applied displacements. Consequently, the dynamic behavior of the half space should be modeled in each degree of freedom by a spring-dashpot device having frequency-dependent properties.

In principle, such a frequency-dependent foundation model could be used directly in a frequency-domain analysis of the soil-structure interaction problem. However, the analysis of a complicated soil-structure idealization involving a large number of degrees of freedom can be greatly simplified if the foundation component of the model is assumed to be frequency-independent. Moreover, if the analysis is to be extended into the nonlinear range, the frequency-domain approach is not applicable.

For these reasons, it is advantageous to represent the elastic half-space foundation medium by frequency-independent components, with their properties selected to reproduce the true frequency-dependent behavior as well as possible.

In cases where the foundation slab is to support a mechanical system operating at relatively high frequencies, the inertial resistance of the soil is small and the half-space can be modeled by a simple spring-dashpot device in each degree of freedom. The spring constant should be selected to give the correct static displacements, and then the dashpot coefficient is chosen to provide the best possible agreement with the theoretical resultant base-force amplitude and phase. In modeling the foundation system for an earthquake-response analysis, however, where the highest significant response frequencies are no more than a few times the fundamental soil-structure frequency, better results can be obtained by introducing the virtual mass of the soil as an additional model parameter. The model for each degree of freedom then consists of the spring constant defined by the static-load displacement plus an appropriate virtual mass together with a dashpot coefficient reduced somewhat from that required in the massless model. With the proper selection of these dynamic properties, the response of the lumped parameter foundation model can be controlled to within a few percent of the theoretical half-space performance over the frequency range of interest.

The material is assumed to be linearly elastic in most analysis. Actually, modulus of soils, both granular and cohesive, are strain-dependent and soften with increasing strain. The effects of this nonlinear stress-strain softening are simulated by an iterative procedure in which successive solutions are obtained for different approximated values of modulus and damping, until the strains computed in the problem are compatible with the values of damping. Analyses employing such iterations can be complicated and time-consuming. Generally, iterations beyond those necessary to satisfy the strain compatibility for the amplification problem are unnecessary.

The earthquake response of any soil-structure system can be evaluated by numerical integration of the general equations of motion. However, the mathematical models of such systems usually include a large number of degrees of freedom, and the analysis may be both tedious and expensive if it is done directly in these coordinates. It generally is desirable to transform the equations of motion to the normal (free-vibration mode) coordinates before performing an earthquake-response analysis because the earthquake tends to excite only relatively few modes. It should be emphasized that no approximation is inherent in the use of normal coordinates, if all N coordinates are included in the analysis, the computed response will be the same whether it is evaluated in normal coordinates or in the original nodal coordinates of the discretized system. However, the normal coordinates are much more efficient in describing the displaced shape of the structure, and are generally a very good approximation of the response is given by the first few modal shapes. In this case, the results are approximate only to the extent that significant motions are contained in the model components which have been truncated. Such modal analysis is valid for linear systems only.

CHAPTER 4

METHODOLOGY FOR NUMERICAL ANALYSIS

4.1 GENERAL

Time-history analysis of tall building structures is performed to obtain the dynamic response of such structures during earthquakes. Two finite element programs are used to perform the analyses. The first one is ANSR, a finite element program for nonlinear analysis of structures. The other is ANSYS, a general-purpose finite element software.

The main features of the two programs are described in this chapter briefly, alongwith the different elements used in the two programs to model the structures. Also the earthquake records used in the analyses are presented.

4.2 INTRODUCTION TO FINITE ELEMENT ANALYSIS OF STRUCTURES

The finite element concept has been accepted as a very useful means of solving scientific problems. That is true for problems related to structural analysis too. The development of the method as an analysis tool was essentially initiated with the advent of computers. In the numerical solution of a continuum problem it is basically necessary to establish and solve a system of algebraic equations. Using the finite element method it is possible to establish and solve governing equations for complex systems in a very efficient way with the help of computers. It is mainly for the generality of the structure or continuum that can be analyzed, for the relative ease of establishing the governing equations, and for the good numerical properties of the system matrices involved that the finite element method has found wide appeal.

4.3 FINITE ELEMENT ANALYSIS USING ANSR

4.3.1 The program ANSR

ANSR is a general purpose computer program for the static and dynamic analysis of nonlinear structures. Several versions of this program have been developed over the years by Professor Graham H. Powell and others at the University of California, Berkeley. The first one, called ANSR-I, was developed in 1975 (Mondkar and Powell, 1975). The second and third versions of the program (ANSR-I and ANSR-II) were realeased in 1979 and 1982 , respectively (Mondkar and Powell, 1979; Oughourlian and Powell, 1982). The personal computer version of the ANSR-I, named PC-ANSR, has been used for this research.

4.3.2 Idealization of the Structure

1. The structure is idealized as an assemblage of discrete finite elements connected at the nodes. The theory and solution procedures are based on the finite element formulation of the displacement method, with nodal displacement as the field variable.

2. Each node may posses upto six degrees of freedom in displacement.

3. Provision is made for degrees of freedom to be deleted or combined.

4. The masses of different structural elements are assumed to be lumped at the nodes.

5. There are provision to include viscous damping in the material properties of the elements. Damping effects proportional to mass, initial stiffness and tangent stiffness can be incorporated.

4.3.3 Static and Dynamic Loading Options

1. Loads are applied only at the nodes. Both static and dynamic loads can be specified with options for analyses for both or any one of them at any particular attempt. But the static loads, if any, must be applied prior to the dynamic analysis.

2. Several static force patterns may be specified at the same time in case of static analysis. The loads are then applied in a series of load increments, each being specified as a linear combination of the static force pattern.

3. The dynamic loads may consist of earthquake ground accelerations, time dependent nodal loads and prescribed initial values of the nodal velocities and acceleration.

4. Earthquake excitations are defined by time histories of ground acceleration. Three different time histories may be specified, one for each of X, Y and Z axes of the structure.

5. Any number of time histories of dynamic forces may be specified. As with the earthquake records, these time histories may be specified to be at equal or unequal time intervals.

6. Values of initial translation and/or rotational velocity and acceleration may be specified at each node. The structure subjected to impulsive loads can be analyzed by prescribing appropriate initial velocities.

4.3.4 Finite Elements

Three Dimensional Truss Element

Truss elements can transmit axial load only, but may be oriented arbitrarily in space (Fig. 4.1a). Large displacement effects may or may not be included. When this effect is included, it is considered for both static and dynamic analysis.

Two alternative modes of inelastic behavior may be specified, namely (1) yielding in both tension and compression (Fig. 4.1 b) and (2) yielding in tension and buckling in compression (Fig. 4.1 c). Strain hardening effects may be considered.

Initial axial forces in the truss elements can be specified. These initial forces will typically be the forces in the elements under static loading, as calculated by a separate analysis. For consistency, these forces should be in equilibrium with the static load producing them but this is not essential as the computer program makes corrections for any equilibrium unbalance resulting from the initial forces. Thus it is possible to compute the displacements of a truss-bar structure with specified initial forces.

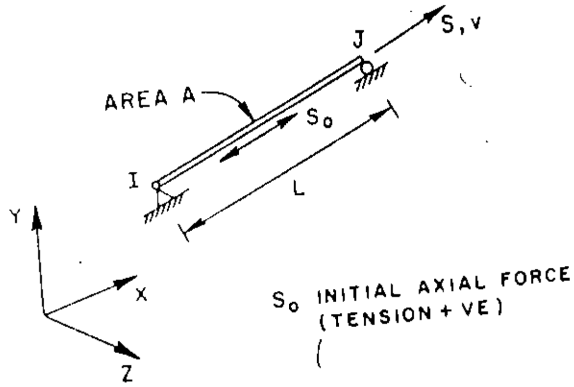
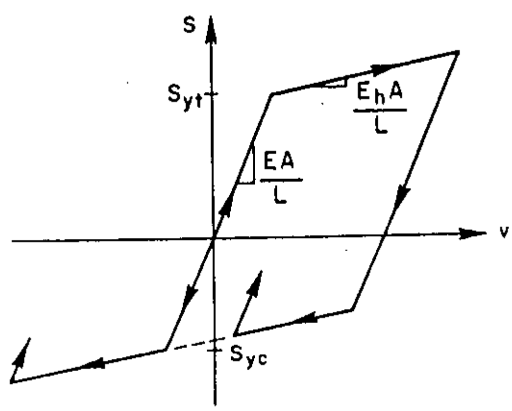


Fig 4.1 (a) Truss Element



(a) YIELD IN TENSION AND COMPRESSION

Fig 4.1 (b) Yield in Tension and Compression (Inelastic Truss Element)

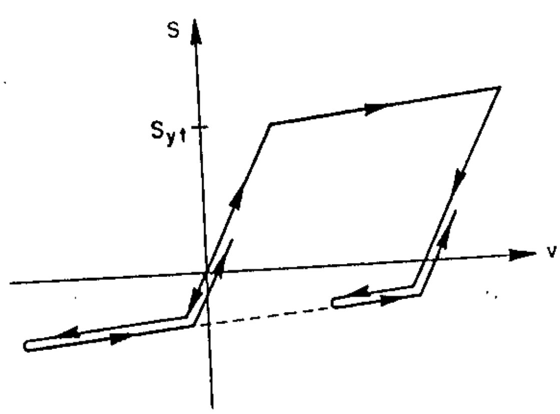


Fig 4.1 (c) Yield in Tension Buckling in Compression (Inelastic Truss Element)

Two Dimensional Beam-Column Element

Beam-Column elements may be arbitrarily oriented in the global XYZ plane (Riahi et al., 1978). Each element must be assigned an axial stiffness plus a major axis of flexural stiffness. Torsional and minor axis flexural stiffnesses may also be specified if necessary. Elements of variable cross section can be considered by specifying appropriate flexural stiffness coefficients. Flexural shear deformations and the effects of eccentric end connections can be taken into account.

Yielding may take place in concentrated plastic hinges at the element ends. Hinge formation is affected by the axial force and major axis bending moment only. That is, an element may be placed in a three-dimensional frame, but its yield mechanism is only two-dimensional, in the plane of major axis bending. The yield moments may be specified to be different at the two element ends, and for positive and negative bending. The interaction between axial force and moment in producing yield is taken into account approximately.

Strain hardening is approximated by assuming that the element consists of elastic and elasto-plastic components in parallel. With this type of strain hardening idealization, if the bending moment in the element is constant, and if the element is of uniform strength, then the moment-rotation relationship for the element will have the same shape as its moment-curvature relationship (Fig.4.2a). This follows because curvature and rotation in this case are directly proportional. If, however, the bending moment or strength vary, then the curvatures and rotations are no longer proportional, and the moment-rotation and moment-curvature variations may be quite different (Fig.4.2b).

The beam-column has three primary modes of deformation, namely (a) axial extension, (b) flexural rotations in the major plane at ends i and j and (c) deformation due to nodal displacement. These three modes of deformation are shown in Fig. 4.3.

Rigid Floor Diaphragms

A frequently made assumption in the analysis of tall buildings is that each floor diaphragms is rigid in its own plane. To introduce this assumption, a "master" node at the center of mass of each floor may be specified, as shown in Fig. 4.4. Each master node has only three degrees of freedom as shown, which are displacements of the diaphragm horizontally as a rigid body. If any beam-column is connected to a diaphragm, its stiffness may be formulated partly in terms of these "master" displacements and partly in terms of displacements which are not affected by the rigid diaphragm assumption.

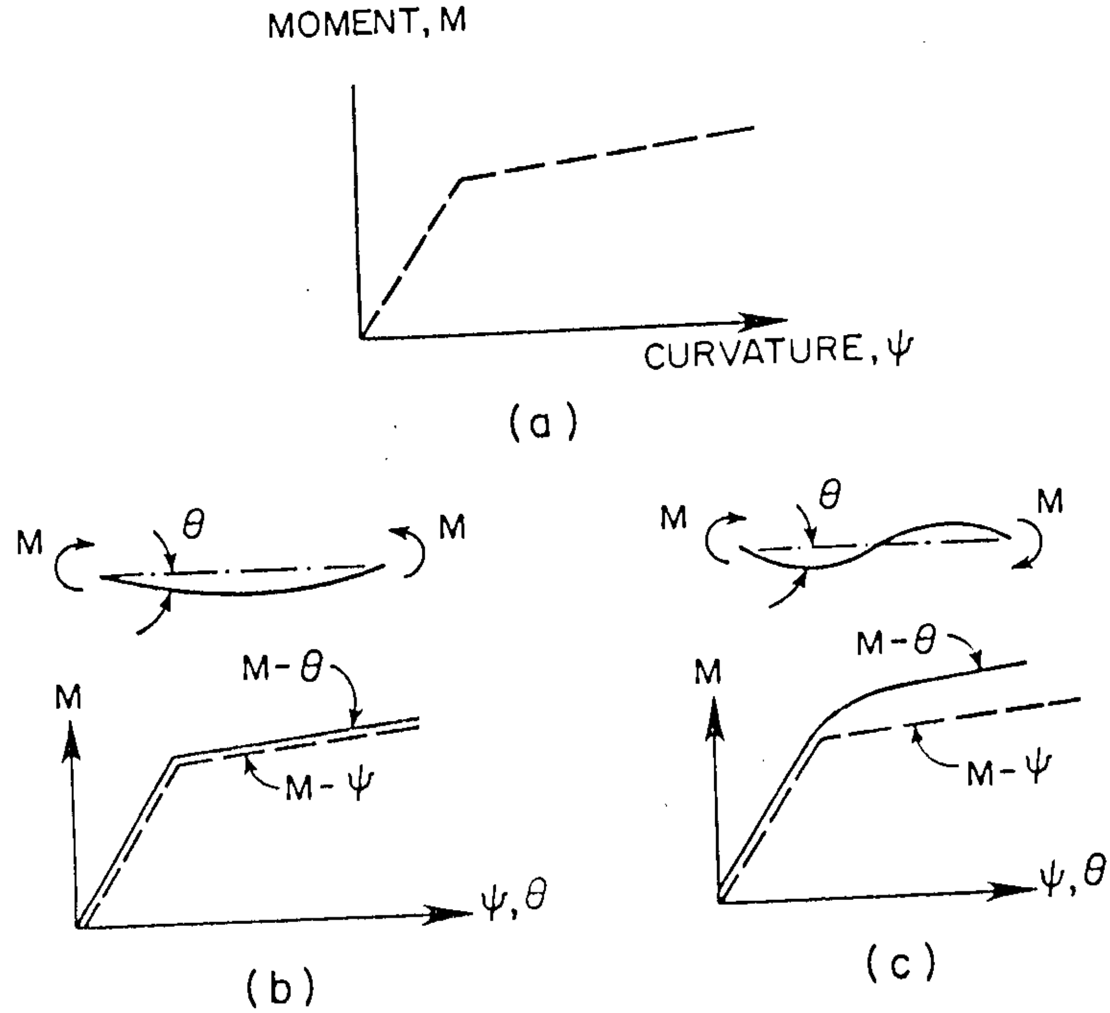


Fig. 4.2 Moment-curvature and moment-rotation relationship

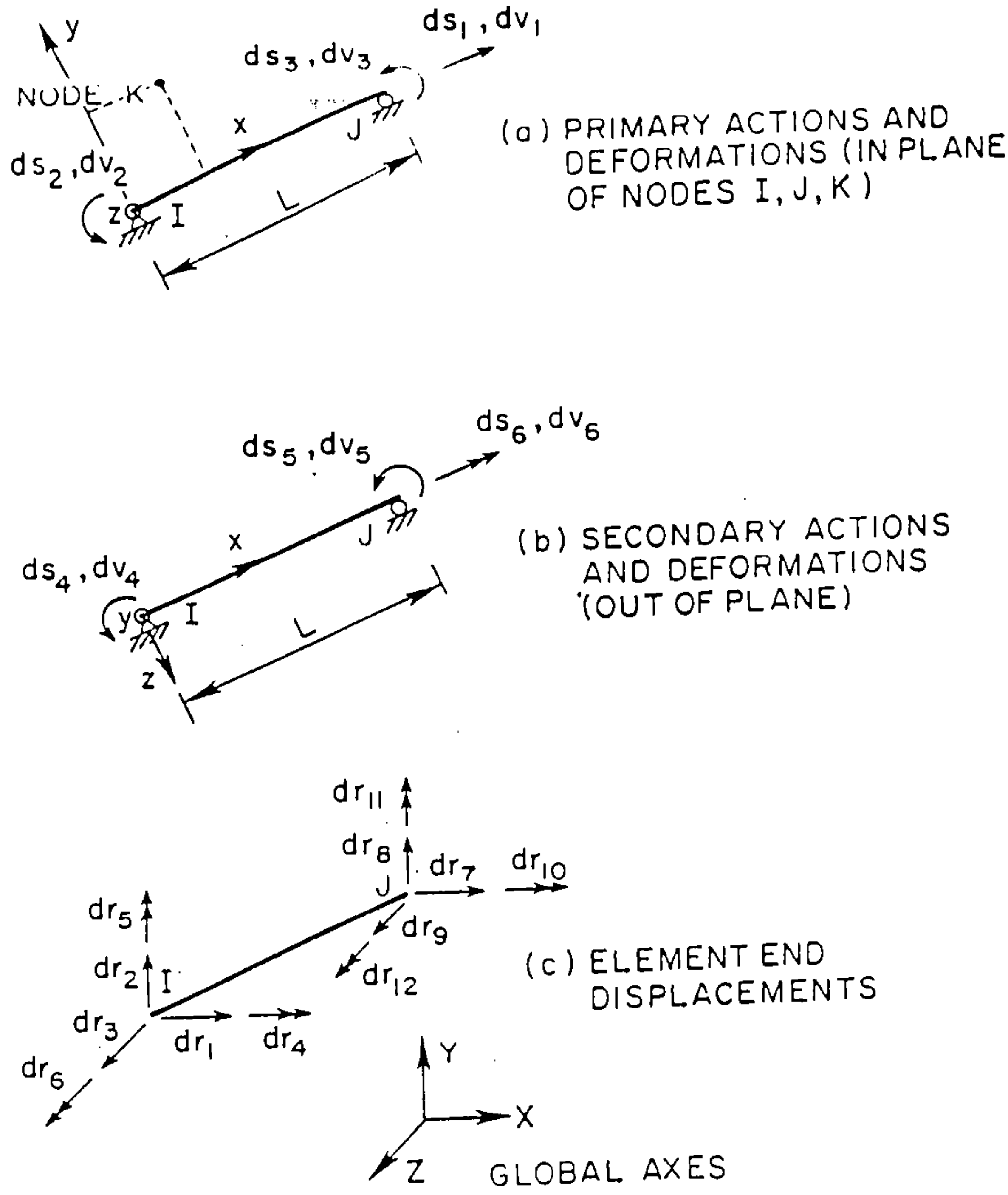


Fig. 4.3 Deformations and displacements

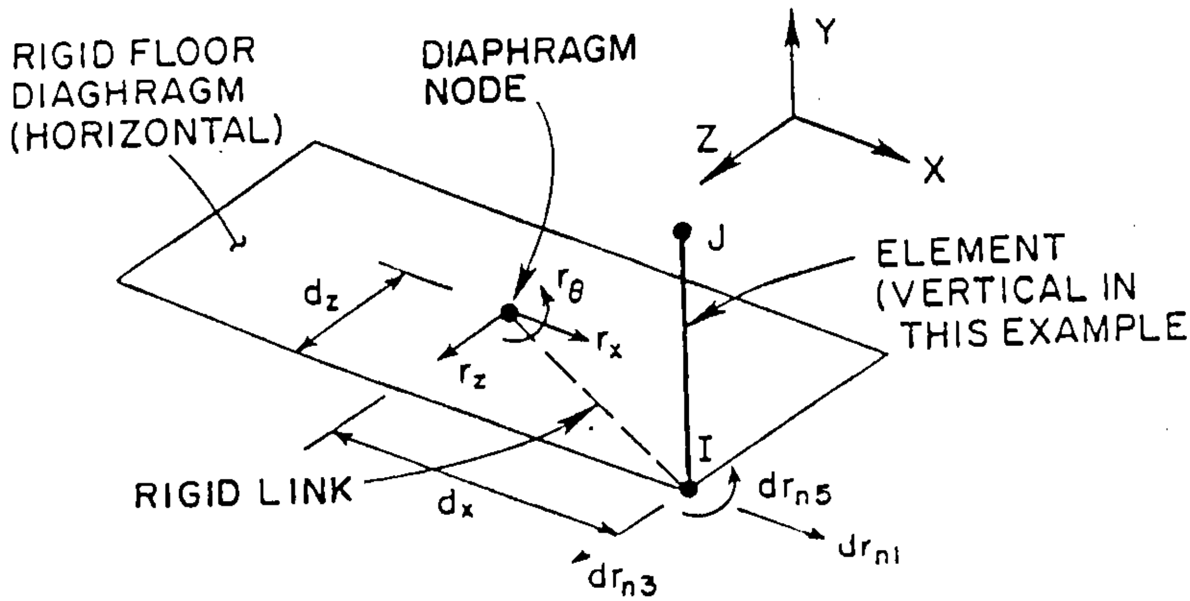


Fig. 4.4 Rigid floor diaphragm modelling

4.3.5 SOLUTION PROCEDURES

1. The program incorporates a solution strategy defined in terms of a number of control parameters. By assuming appropriate values to these parameters, a wide variety of solution schemes, including step-by-step, iterative and mixed schemes, may be constructed. This permits the program user considerable flexibility in selecting optimal schemes for particular types of nonlinear behavior.

2. For static analysis, a different solution scheme may be employed for each load increment.

3. The dynamic response is computed by step-wise time integration of the incremental equations of motion using Newmark's β-γ-δ operator. A variety of integration operators may be obtained by assigning appropriate values to the parameters β and γ operator. The most commonly used scheme is the 'constant average acceleration' scheme with β=0.25, γ=0.5 and δ=0.0.. Viscous damping effects may be introduced by specifying a positive value to the parameter δ. In most cases, damping effects will be introduced more explicitly, in mass dependent or stiffness dependent form.

4.3.5.1 Equations of Motion

The discrete incremental equations of motion for an undamped system are:

$$M \bullet \ddot{q} + [{}^1k_{-E} + {}^1k_{-G}] \bullet q = {}^2p - (M \bullet {}^1\ddot{q} + {}^1R) \tag{4.1}$$

Where q and $\ddot{q}$ are the vectors of increments of nodal displacement and acceleration, respectively; and ${}^1\ddot{q}$ is the vector of nodal acceleration. M, ${}^1k_{-E}$, ${}^1k_{-G}$ are consistent mass matrix, linear stiffness matrix, geometric stiffness matrix respectively and are obtained from the element matrices using well known assembly procedure (Zienkiewich, O. C., 1971).

The equation of equilibrium for static analysis can be obtained from equation (4.1) by omitting the terms containing accelerations. Viscous effects may be included by modifying equation (4.1) as follows:

$$M \bullet \ddot{q} + {}^1C \bullet \dot{q} + [{}^1k_{-E} + {}^1k_{-G}] \bullet q = {}^2p - (M \bullet {}^1\ddot{q} + {}^1C \bullet {}^1\dot{q} + {}^1R) \tag{4.2}$$

In which $\dot{q}$ and ${}^1\dot{q}$ are the vectors of velocity increment and velocity respectively; and ${}^1C$ is a damping matrix.

In general, the structural response will be computed by applying the load in small steps, and in some cases equilibrium iterations may have to be carried out to obtain results with a sufficient degree of accuracy.

4.4.5.2 Solution Technique

Most solution procedures for nonlinear analysis can be classified as either step-by-step or iterative. Both procedures have been widely used in static nonlinear analysis, and both applicable to dynamic nonlinear analysis in which the response is computed by step-wise marching in time.

In step-by-step solution procedure the load is applied in several small steps and the structure is assumed to respond linearly within each step, the response being obtained without iteration.

Two types of iterative procedure are commonly used, namely Newton-Raphson iteration and Constant Stiffness Iteration. In Newton-Raphson iteration the structure tangent stiffness matrix is reformed at every iteration, and a disadvantage of this procedure is that a large amount of computational effort may be required to form and decompose the stiffness matrix. In constant stiffness iteration, the stiffness matrix is formed only once. Although this procedure has the advantage that the tangent stiffness matrix is not formed and inverted at every iteration, its disadvantage is that iteration will typically converge more slowly than Newton-Raphson iteration, and schemes to accelerate convergence may be desirable.

The computer program includes "alpha constant" acceleration scheme (Nayak and Zienkiewicz, 1971). In this scheme the displacement increments during any iteration are scaled in an attempt to obtain the same result as if Newton-Raphson iteration were employed. For each iteration the scheme requires two steps of displacement computation, and two steps of residual load computation.

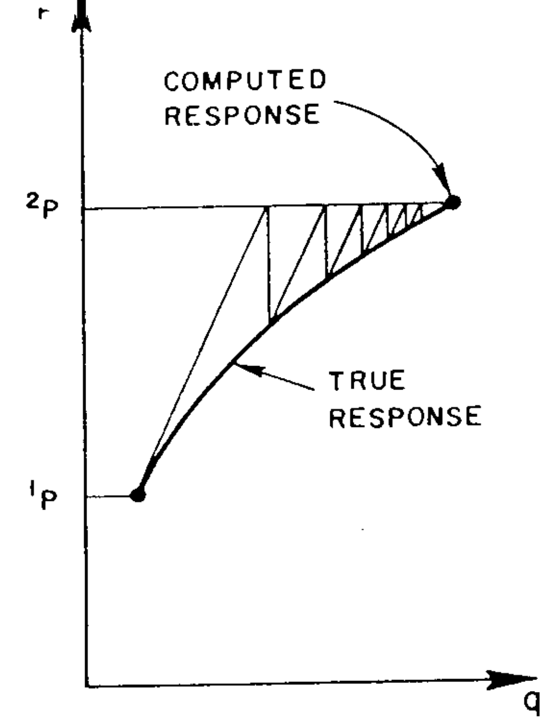


Fig 4.5 Constant Stiffness Iteration

4.3.5.3 Integration of Equation

For integration of equation of motion (4.2), the time domain is divided into a number of time steps, and it is required to compute the displacements, velocities and accelerations in configuration at time $\tau = t + \Delta t$ with the knowledge of the previous deformation history from time 0 to time t. An implicit, single step, two parameter (β,γ) family of integration operators has been described by Newmark (Newmark, 1959), in which it is assumed that the increments in velocity and acceleration are related to increment in displacement and the state of motion at time t. A number of operators can be obtained by specifying various values of the parameters β and γ. The "constant average acceleration" operator with β=1/4, γ=1/2 has been shown to be unconditionally stable for linear analysis. Accuracy and stability for nonlinear analysis can be studied only by numerical experimentation. It is possible to introduce artificial viscous affects by specifying a damping parameter δ. With Newmark's operator, an integration algorithm is been designed in which iterations are performed within a time step to satisfy equilibrium subject to a specified tolerance. In this study, β=1/4, γ=1/2, δ=0 have been used.

4.4 FINITE ELEMENT ANALYSIS USING ANSYS

ANSYS (SAS IP Inc., 1996) is a general-purpose finite element analysis software from ANSYS Inc. of Canonsburg, PA, USA. It was introduced in 1970 and several revisions have been published since then. It is also available for different types of hardware from PCs to mainframes. The PC version of Revision 5.3 was used for the present research. The PC version includes almost all the main features of the software, but with limited capabilities in some cases.

4.4.1 FINITE ELEMENT IDEALIZATION

ANSYS has a library with large number of elements to represent different types of structural components. For the present study three dimensional beam elements and elastic shell elements are used to model beams, columns, and floor slabs and mat foundation. Two special types of elements are used to incorporate the true nature of the structure. One is the structural mass element to lump the structural masses at nodal points and the other is the spring element used to replace the soil beneath the structure. The elements are described in details below.

Beam Element

An elastic, uniaxial, three dimensional beam element is used which can be subjected to tension, compression, torsion and bending. The element has two nodes and six degrees of freedom at each node: translations in the nodal X, Y and Z directions and rotations about the X, Y and Z axes. The geometry, nodal locations and coordinate systems for this element are shown in Fig. 4.6. The beams and columns of the superstructure and the beam-column elements in the mat are represented by this element while formulating the model of the frame structure.

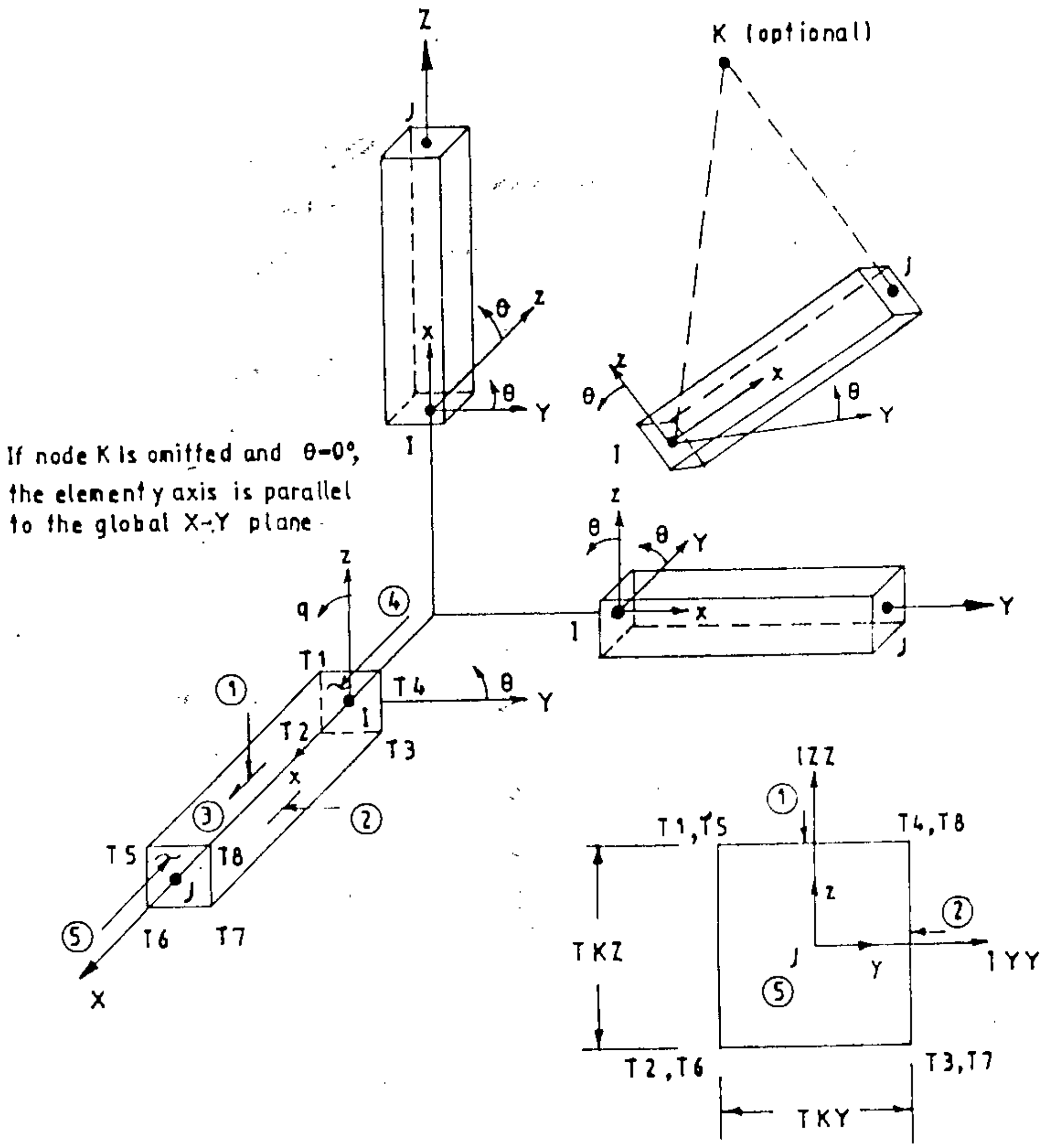


Fig.4.6 3-Dimensional elastic beam element

Shell Element

Elastic shell element has been used for the floor slabs. Both bending and membrane stresses can develop in these elements. It has six degrees of freedom at each node. Fig. 4.7 shows the geometry, node locations and the coordinate system for this element. The element is defined by four nodes, the thickness and orthotropic material properties. In the building model, the shell elements are used with zero curvature so that it acts in the same way as the flat floor slab.

Mass Element

This is a point element used as a structural mass at any particular nodal point. The mass element can correspond to any of the six degrees of freedom, as shown in Fig. 4.8. The masses of the structural elements and the dead loads are modeled with this element.

Spring Element

The elastic soil below the mat foundation is represented by this elements. Elastic soil properties lumped at different nodes are incorporated in these elements. Damping can also be included in this element. But for the present research it is done by specifying that with the analysis options.

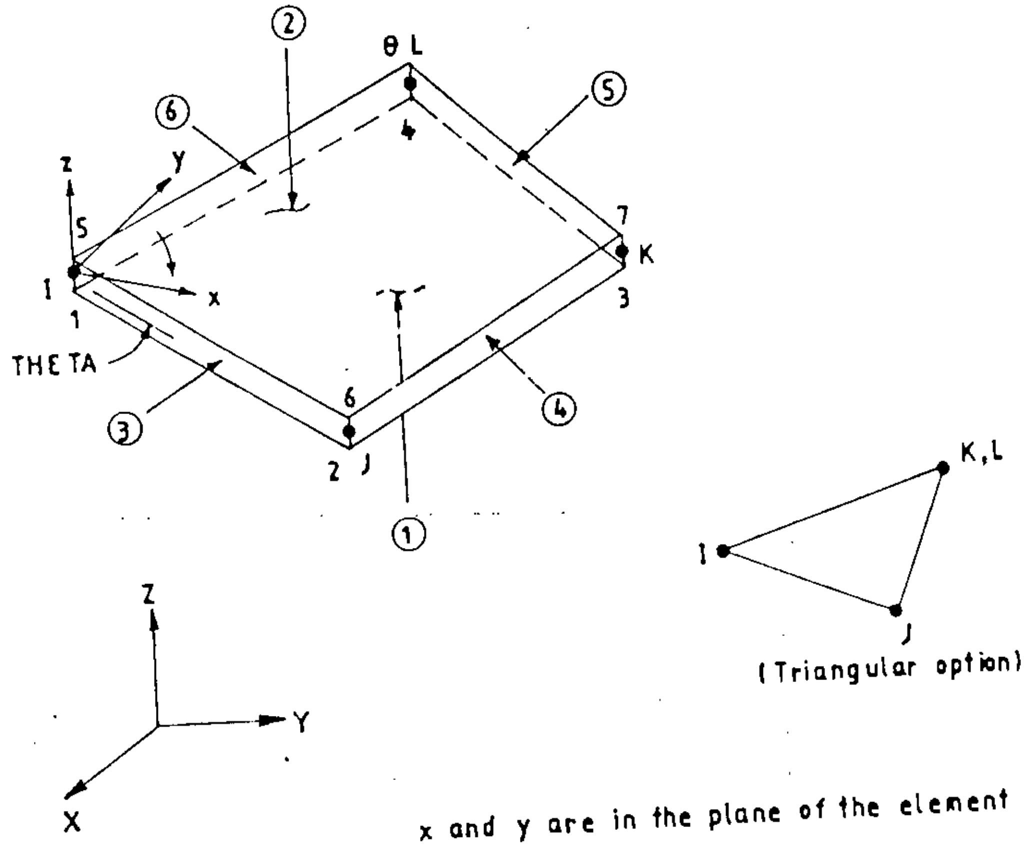


Fig. 4.7 Elastic shell element

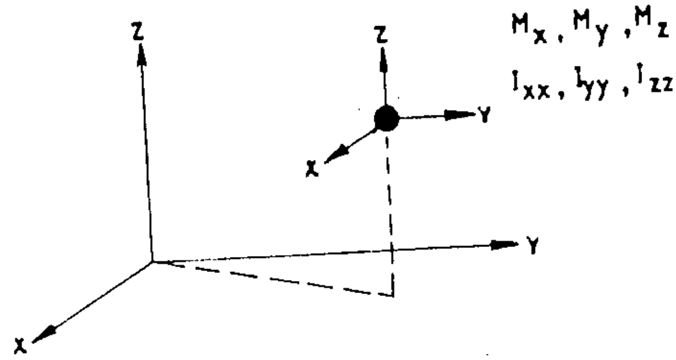


Fig. 4.8 Structural mass element

4.4.2 ANALYSIS OPTIONS

ANSYS has been used to verify the results of the static analysis by ANSR. Several options are available for the type of dynamic analysis done in the present study. These include Modal Analysis, Transient Dynamic Analysis and Spectrum Analysis. Transient Dynamic Analysis is performed for the time-history analysis. The frequencies and periods of different mode shapes are obtained through modal analysis.

Transient Dynamic Analysis

The basic equation of motion solved by a transient dynamic analysis is

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{u\} = \{F(t)\} \tag{4.3}$$

where $[M]$ = mass matrix

$[C]$ = damping matrix

$[K]$ = stiffness matrix

$\{\ddot{u}\}$ = nodal acceleration vector

$\{\dot{u}\}$ = nodal velocity vector

$\{u\}$ = nodal displacement vector

$\{F(t)\}$ = load vector

At any given time, t, these equations can be thought of as a set of "static" equilibrium equations that also take into account inertia forces ($[M]\{u\}$) and damping forces ($[C]\{u\}$). The ANSYS program uses the Newmark time integration method to solve these equations at discrete timepoints. The time increment between successive timepoints is called the integration time step.

Modal Analysis

Linear analysis in performed in ANSYS for obtaining the frequencies and mode shapes. The system is considered undamped unless the damped eigensolver is selected. The governing equation in matrix notation is

$$[M]\{\ddot{u}\} + [K]\{u\} = \{0\} \tag{4.4}$$

where $[M] =$ structural mass matrix

$[K] =$ structural stiffness matrix

$\{\ddot{u}\} =$ nodal acceleration vector

$\{u\} =$ nodal displacement vector

4.5 MODELING OF FOUNDATION-MAT-SUPERSTRUCTURE SYSTEM

Fig. 4.9 shows the two-dimensional model for a rectangular eight-story two-bay framed building on mat foundation. The mat foundation is modelled by beam-column elements joining the column bases. The building columns and beams are represented by beam-column elements. Floor masses are lumped at the column-beam joints. Frequency independent horizontal springs (truss element) with viscous damping are used to represent horizontal stiffness and damping for the rectangular mat foundation. In order to allow rocking vibration, vertical springs are placed, the stiffness of which are calculated based on the rocking stiffness of the foundation. Damping in the rocking mode, which is highly frequency dependent and negligible at low frequencies, is ignored. Soil stiffness and damping values for embedded rectangular foundations have been obtained using expressions given by Pais and Kausel (1988). Superstructure damping of 5% is assigned to the beams and columns at the element level.

A three-dimensional numerical model for an eight story framed building on mat foundation is shown in Fig. 4.10. Here the modeling of floor mass is different from that of the 2D model. Assuming in-plane rigidity of each floor diaphragm, the mass of each floor is lumped at the center of mass. The three degree of freedom (two horizontal translations and torsional rotation) of the center of mass represent the displacements of the floor diaphragm as a rigid body (Fig. 4.4). The other three degrees of freedom of the corresponding beam-column joints are not constrained by the rigid diaphragm assumption.

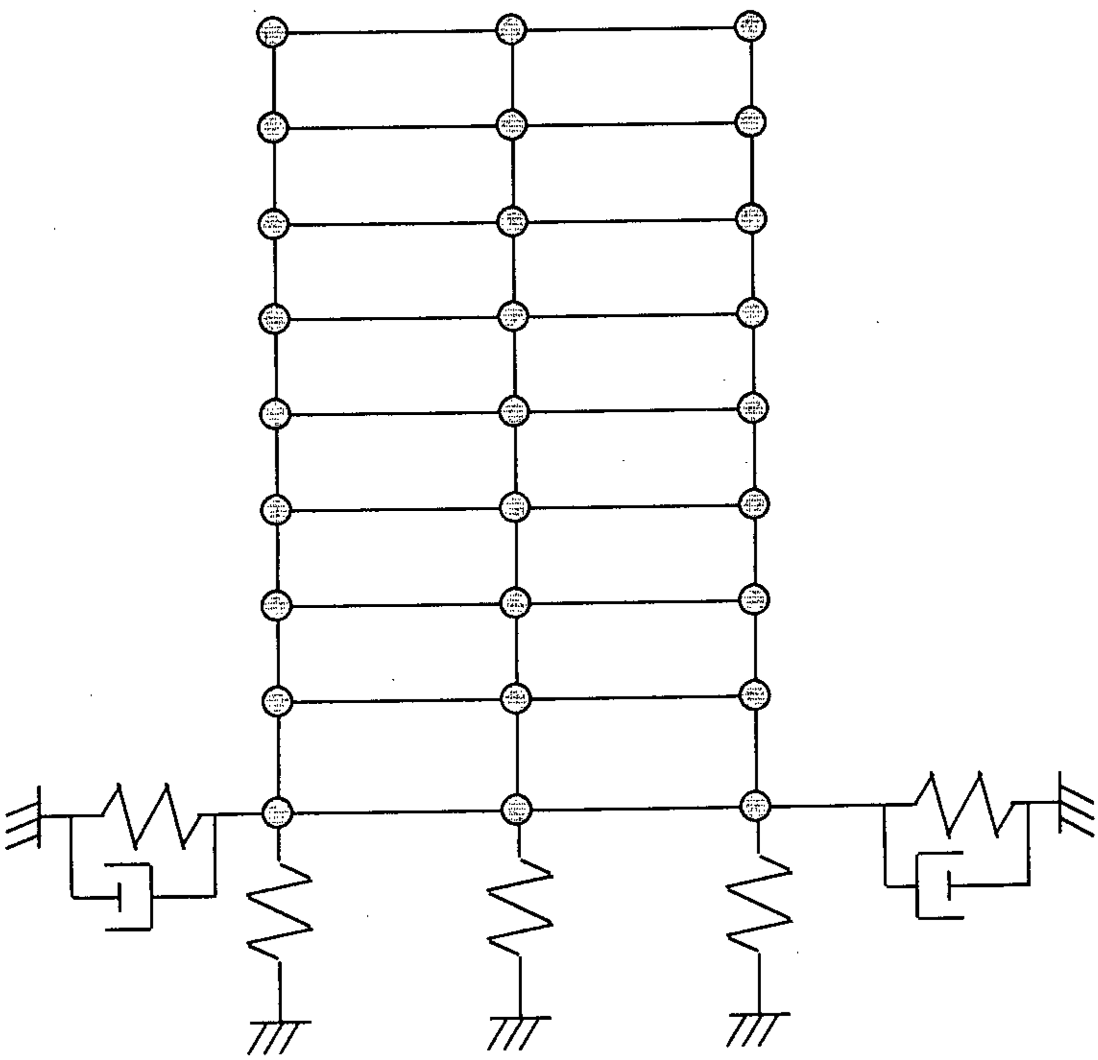


Fig. 4.9 2D analysis model

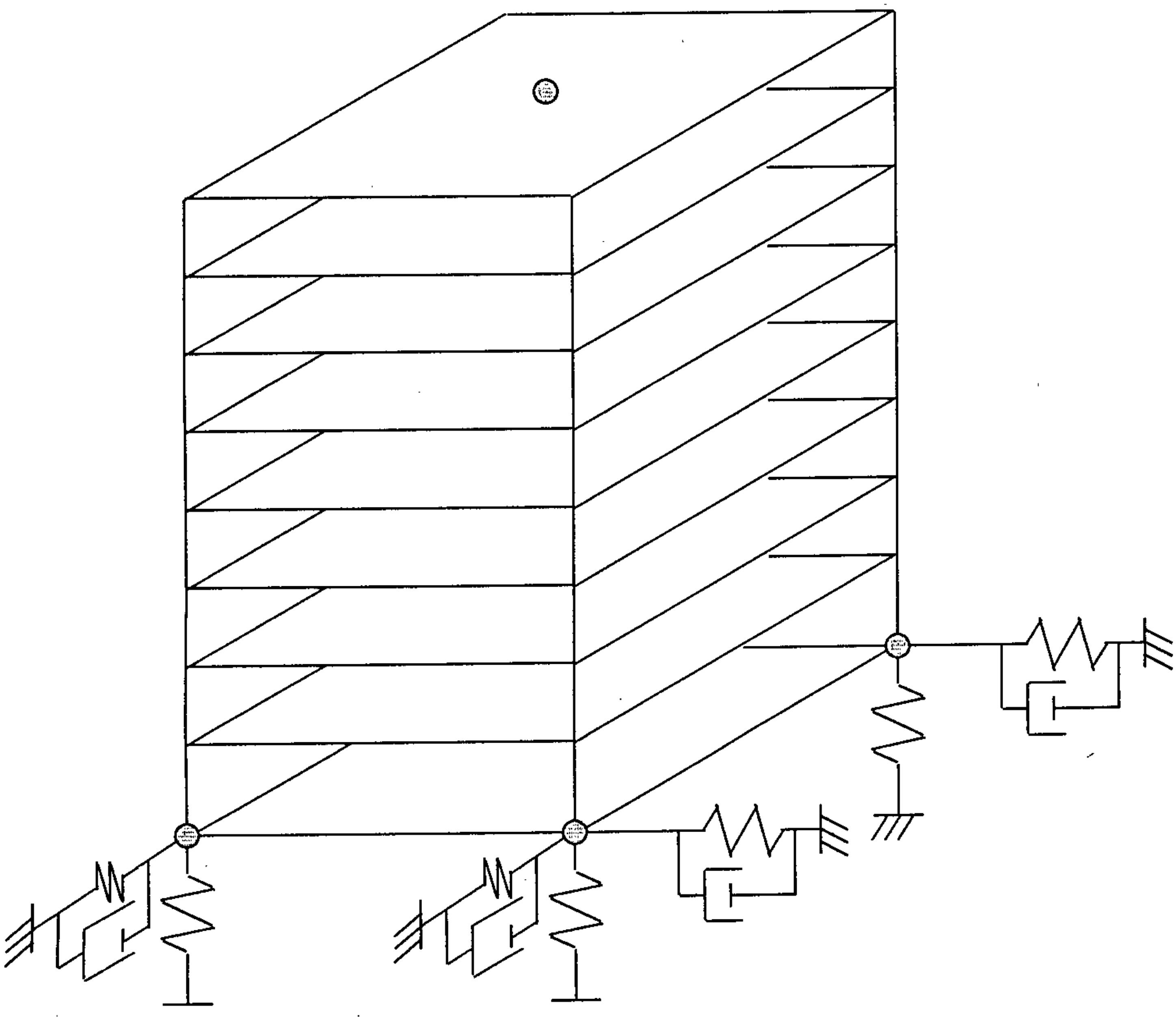


Fig. 4.10 3D analysis model

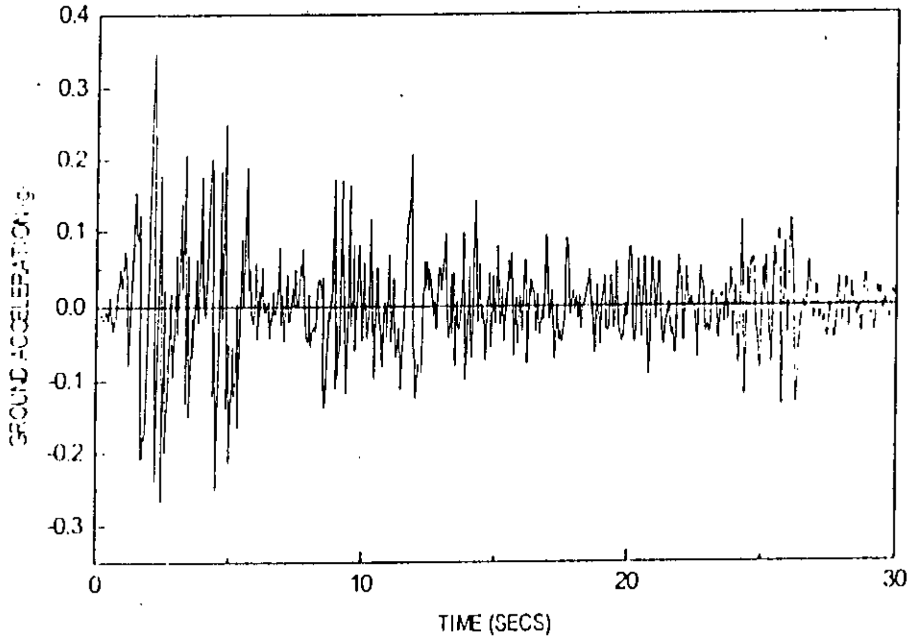
4.6 EARTHQUAKE RECORDS USED IN THE STUDY

Earthquake induced ground motions have different characteristics regarding site conditions, intensity and frequency content. Actual records of five different earthquakes are used as input time histories. The list of the earthquake records used are given in Table 4.1. A fraction of each of the earthquake records is used in this study such that the base shear coefficient (horizontal base shear divided by the weight of the superstructure remains below 0.3. It is assumed that the superstructure remains elastic for this level of forces.

The earthquake motion applied to the building structure is expressed as a percentage of the actual record. The five different records are identified as El Centro 35%, Taft 75%, Loma Prieta 40%, Northridge 25% and San Fernando (Pacoima Dam) 25%. Figs. 4.11 to 4. 15 show time histories of ground motion and response spectra for 5% damping ratio for the five earthquake records used in this study.

Table 4.1 List of Earthquake Motion Used

Earthquake	Record Description	Magnitude (Richter)	Peak Ground Acceleration
El Centro	Imperial Valley, May 1940 Component NS	7.2	0.35g
Taft	Kern County, July 1952 Component N21E	6.7	0.15g
Loma Prieta	Corralitos -Eureka Canyon, October 1989 Component 0 Deg.	7.1	0.63g
Northridge	Sylmar County Hospital, January 1994 Component 360 Deg.	6.6	0.892g
San Fernando	Pacoima Dam, Feberuary 1971 Component S74W	6.4	1.075g



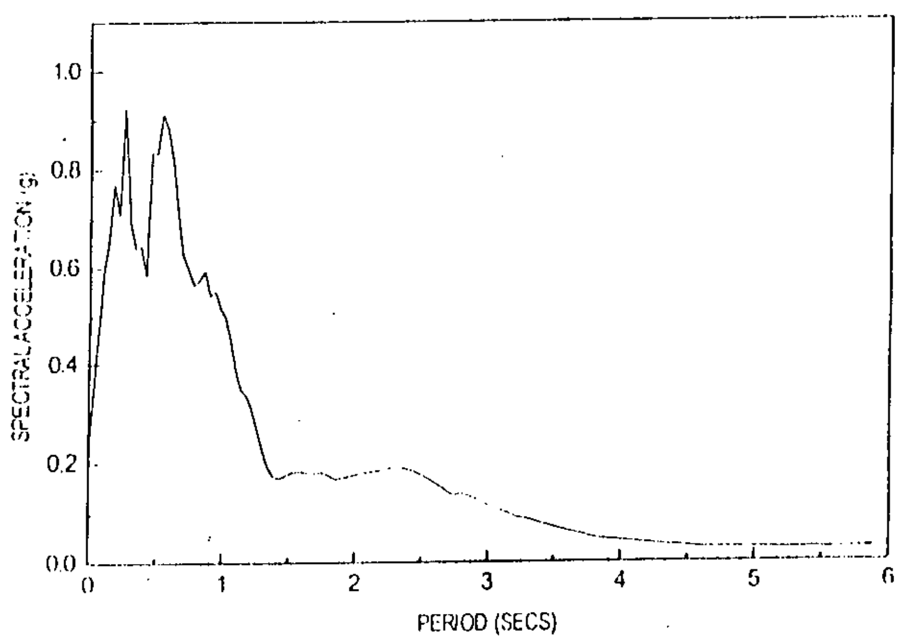


Fig. 4.11 Time History of Ground Motion and Response Spectra for 5% Damping Ratio of El Centro Earthquake

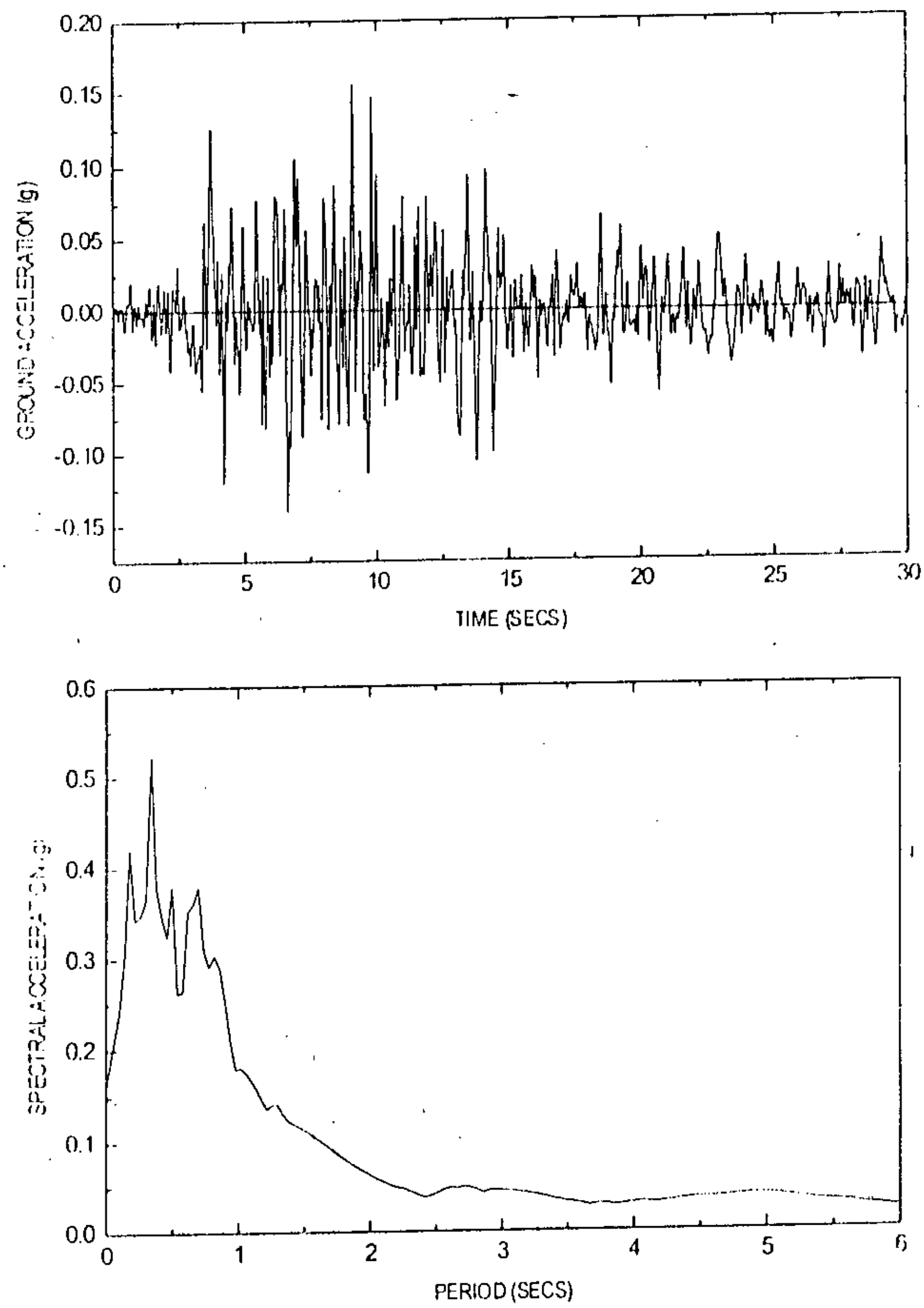


Fig. 4.12 Time History of Ground Motion and Response Spectra for 5% Damping Ratio of Taft Earthquake

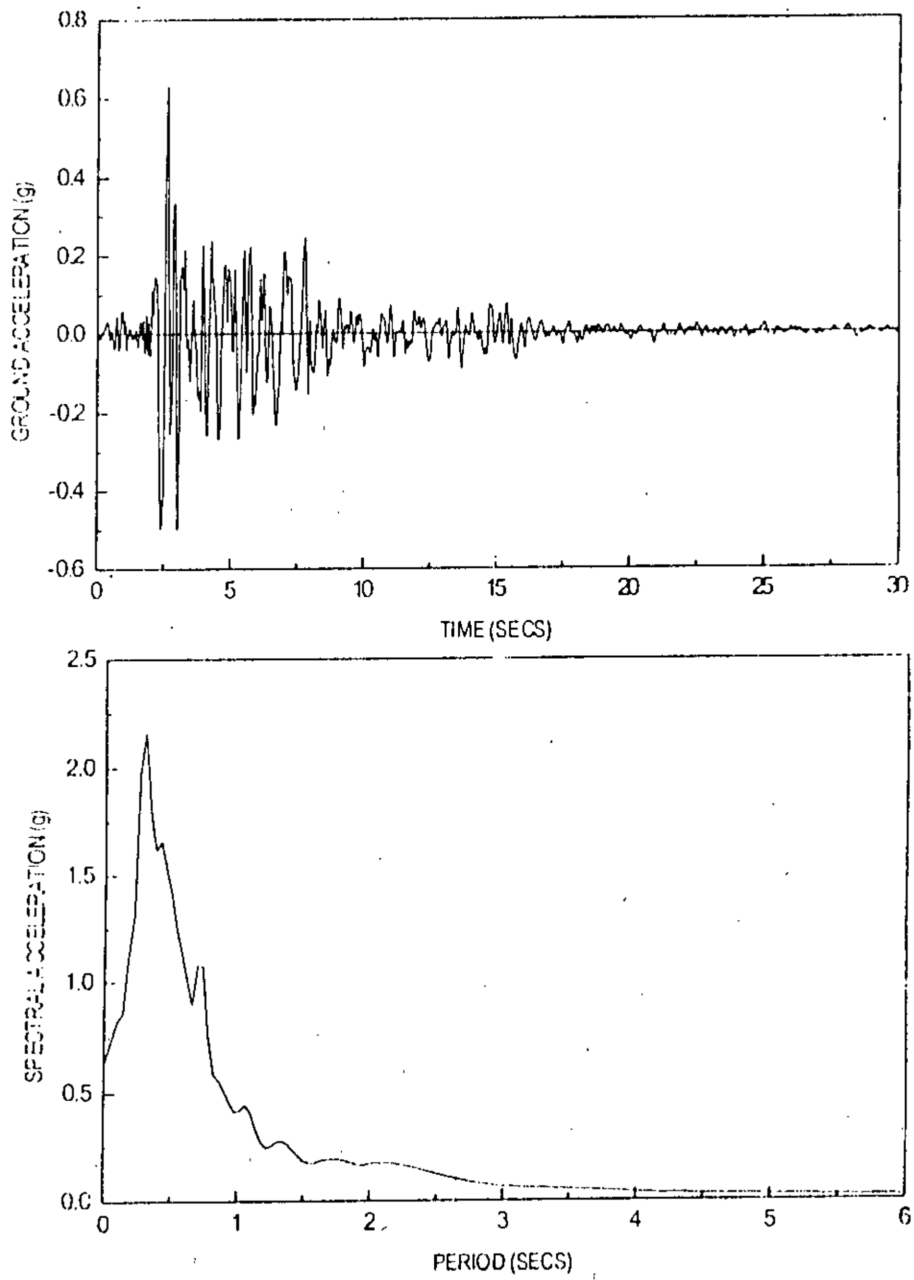
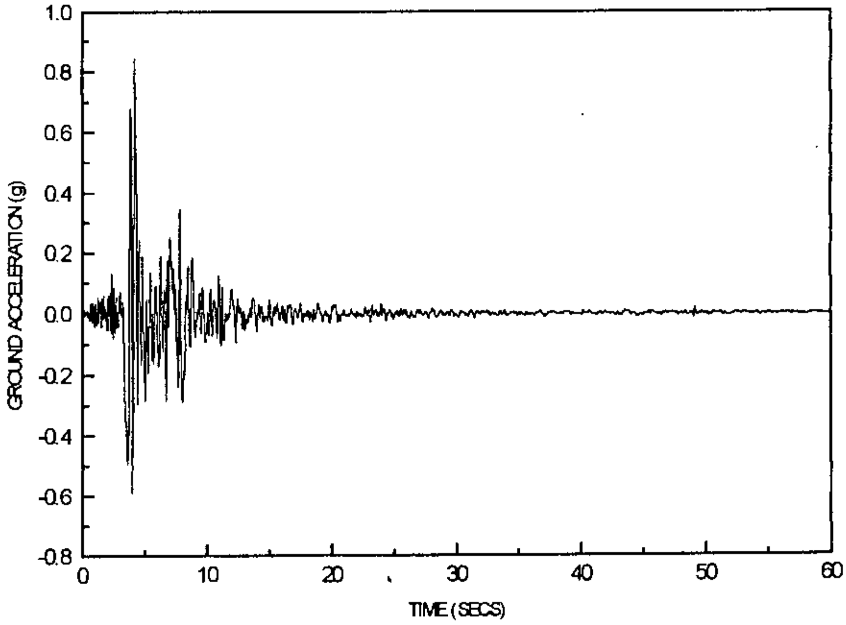


Fig. 4.13 Time History of Ground Motion and Response Spectra for 5% Damping Ratio of Loma Prieta Earthquake



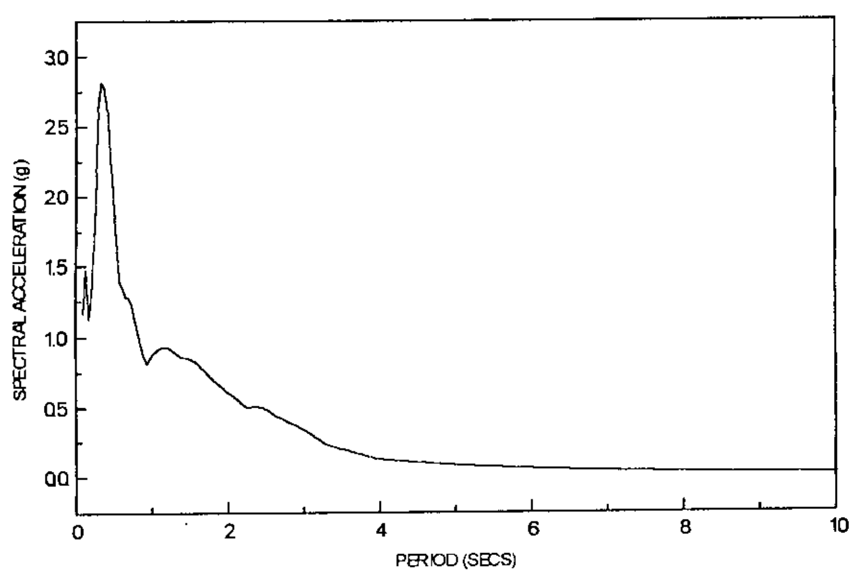
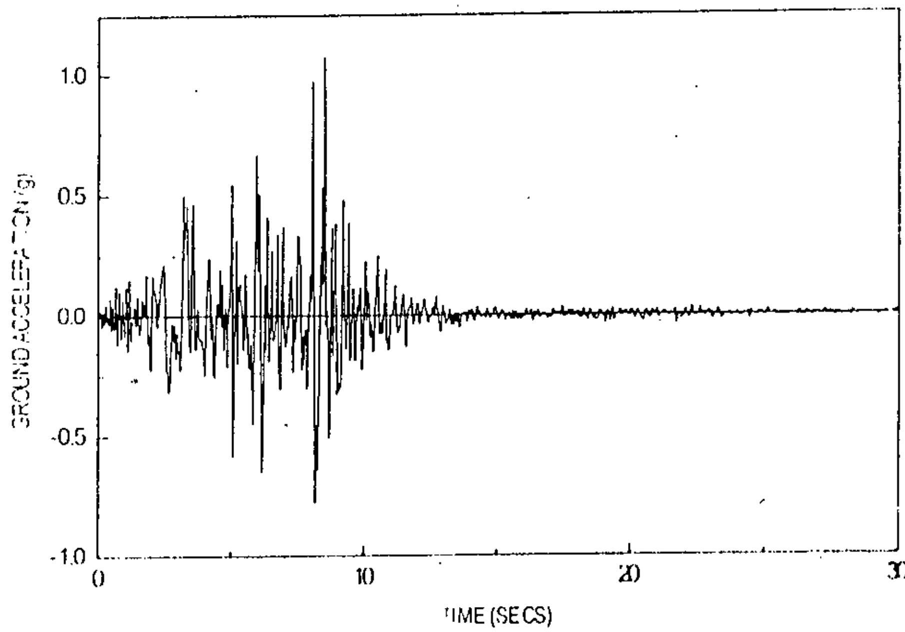


Fig. 4.14 Time History of Ground Motion and Response Spectra for 5% Damping Ratio of Northridge Earthquake



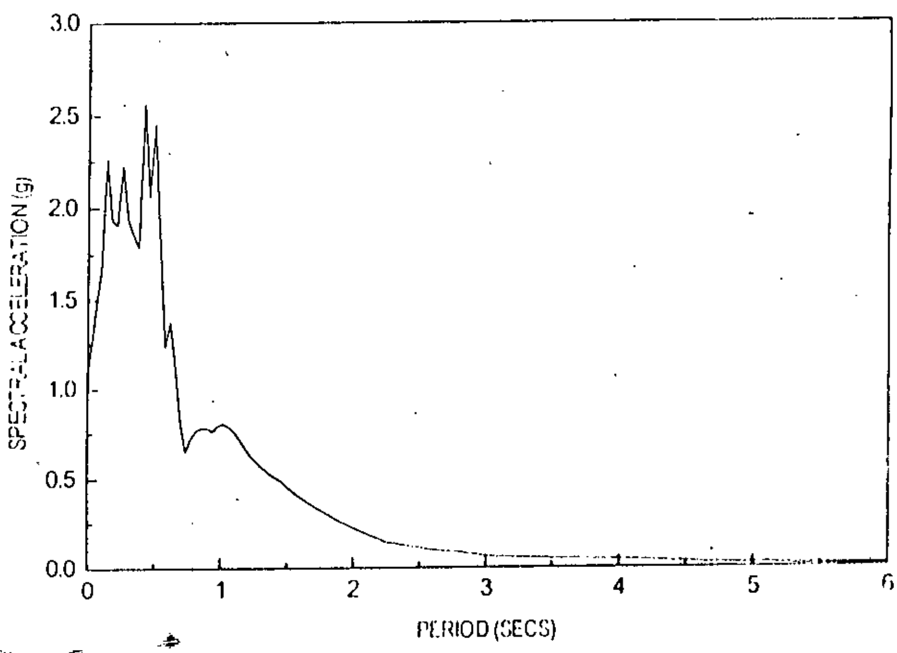


Fig. 4.15 Time History of Ground Motion and Response Spectra for 5% Damping Ratio of San Fernando Earthquake

CHAPTER 5

STATIC SSI EFFECTS ON BUILDINGS

5.1 GENERAL

Foundations receive loads from the superstructure through columns, walls, or both and act to transmit these loads into the soil. The response of a foundation is a complex interaction of the foundation itself, the superstructure above, and the soil. Moments, shears, and deflections can only be computed if these soil reactions can be determined. No analytical method has been devised that can evaluate all of the various factors involved in the problem of soil-structure interaction and allow the accurate determination of the contact pressures and associated subgrade response. Simplifying assumptions must be made for the evaluation of the effect with sufficient accuracy.

5.2 MODELLING OF MAT

As mentioned, the mat foundation is represented by beam-column elements as shown in Fig. 5.1. The soil below the mat is modeled by elastic truss elements acting as equivalent springs. An isometric view of the mat-soil system is shown in Fig. 5.2. The properties of the springs are determined considering their respective contributing areas as illustrated in Fig. 5.3.

To check the validity of the beam-column simplification of the mat, the results of this model was compared with a typical model with shell elements. Both the models were analyzed using ANSYS.

A 3.66m square mat 1.5m thick was used for the analysis. The superstructure was not considered. A typical load of 956 kg/m^2 (200 psf) for each floor was considered. The column loads were applied at the four corners. Value of modulus of subgrade reaction used was 5297 t/m^3 (300 kcf). 2400 kg/m^3 and 210970 kg/cm^2 were taken as self-weight and modulus of elasticity of concrete respectively.

Table 5.1 shows the summary of the comparative results. The deflections obtained from both cases are almost identical. The nodal moments have some differences, but not without acceptable explanation. It is obvious that the beam-column elements have little more moments than the shell elements, but the difference can be kept in mind while analyzing the mat as an assemblage of beam-column elements.

The beam-column model was used with both ANSR and ANSYS to verify the results. While the ANSYS model takes all the necessary details into account, a simplified analysis is performed in ANSR. The results show good agreement with reasonable differences. Table 5.2 gives the results of the two analyses to have a comparative view.

Table 5.1 Comparative results with beam-column and shell elements used to model the mat

Description	Maximum BM (t-m)	Deflection (mm)	
		Maximum	Minimum
With Beam-column elements	20.4	2.45	2.38
With Shell elements	18.84	2.44	2.39

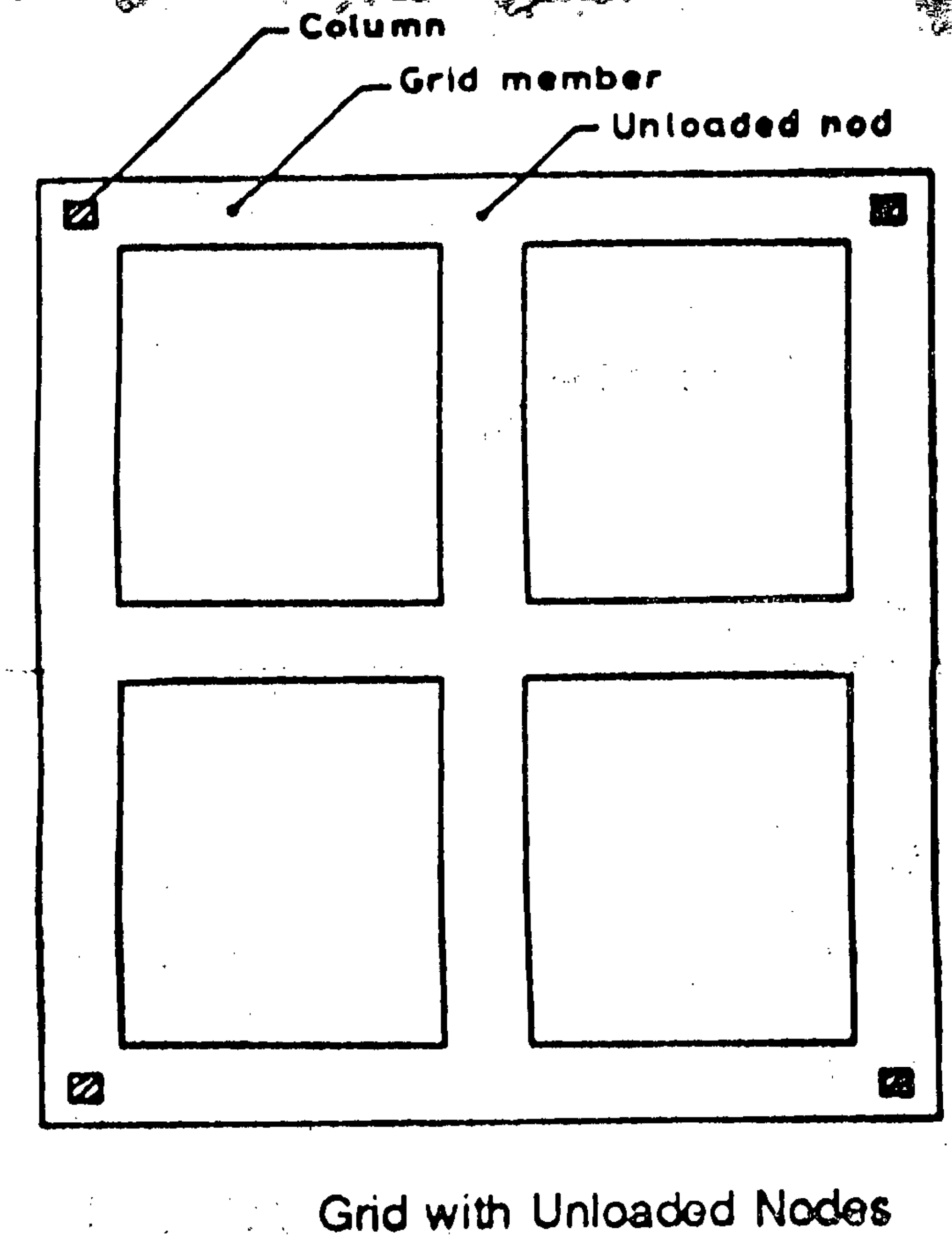


Fig. 5.1 Beam-column representation of the mat

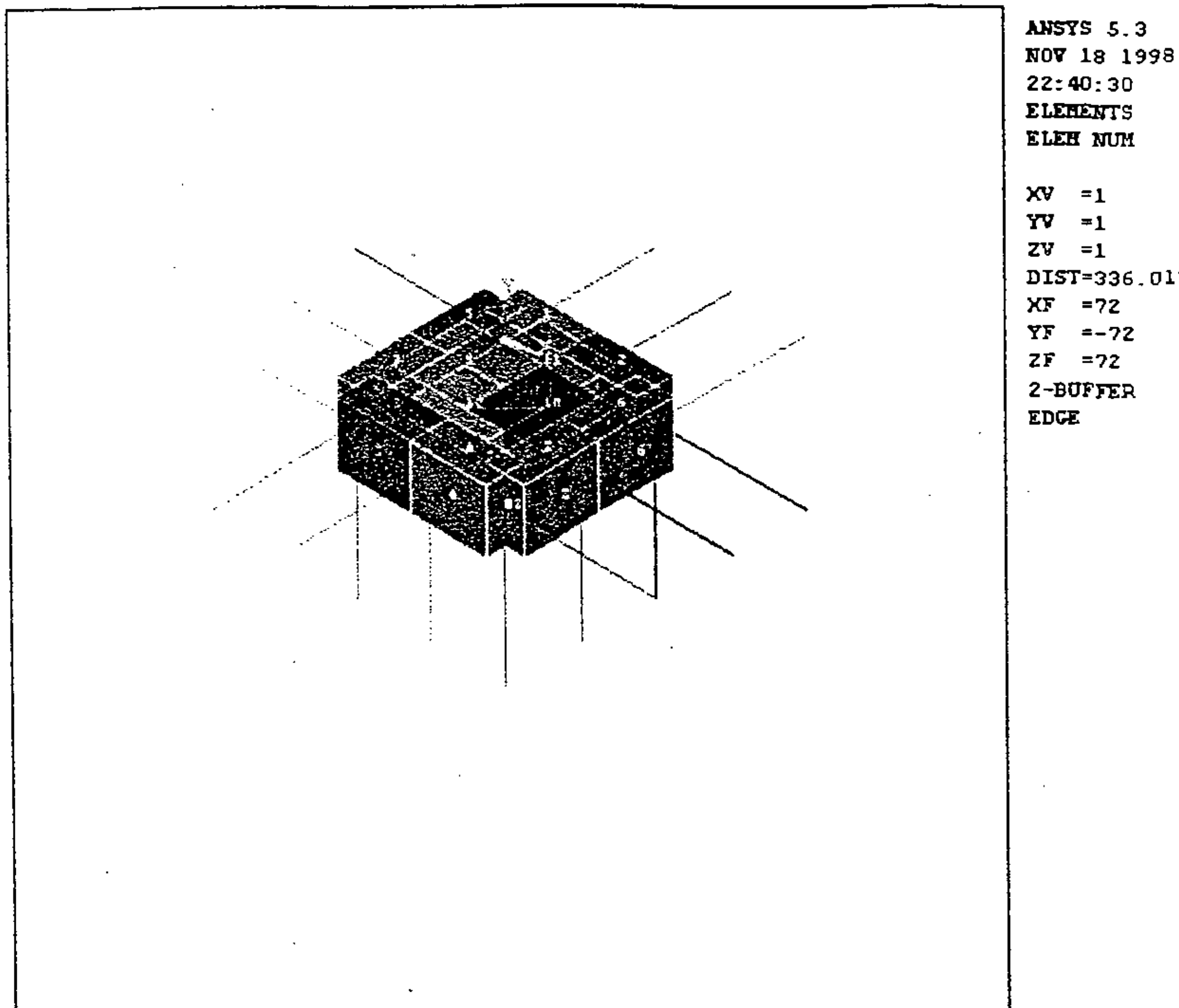


Fig. 5.2 Isometric view of the mat with the springs

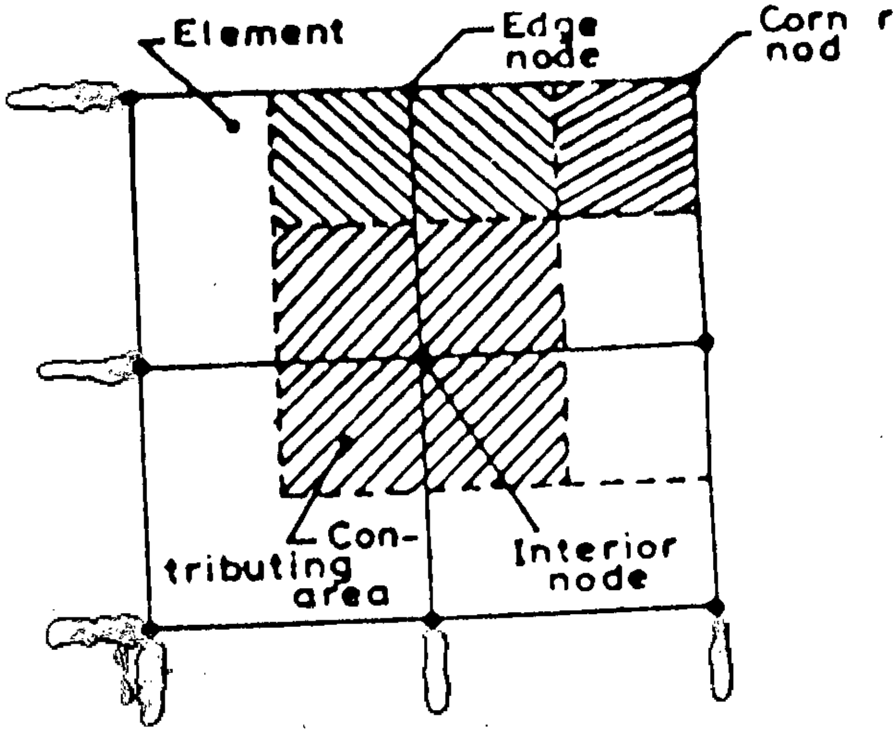


Fig. 5.3 Contributing areas for the nodes

Table 5.2 Results obtained from ANSR and ANSYS for the beam-column model

Description	Maximum BM (t-m)	Deflection (mm)	
		Maximum	Minimum
Analysis with ANSR	20.4	2.45	2.38
Analysis with ANSYS	21.9	2.42	2.36

5.3 PARAMETRIC STUDY

An 8-story reinforced concrete frame building on mat foundation is used for the parametric study. The mat dimensions and concrete properties are the same as those used for the model verification. The columns in the superstructure are 356 mm square and the beam depth is 457 mm. Soil modulus is varied from 2648 t/m^3 (150 psf) to 10594 t/m^3 (600 psf).

Settlements and Bending Moments at the nodes are the two major parameters considered being of major interest. The maximum (total) settlement among all the nodes in the mat and the maximum differential settlement within the mat are studied. Also the building model is analyzed for two different conditions: with and without the superstructure. The effect of different parameters of the foundation and the building are discussed as follows presented in the following subsections.

5.3.1 EFFECT OF SOIL PARAMETERS

Figs. 5.4 to 5.6 show the variations in total settlements, differential settlements and the maximum bending moment in the mat for different types of soils, and for both the cases of with and without considering the superstructure. The three values of the modulus of subgrade reaction correspond to soft, medium and hard clay soils. It is evident that the settlement comes down with increase in soil stiffness (Fig. 5.4). Also the differential settlements decrease as the soil becomes stiffer, but in a slightly different way than the total settlements (Fig. 5.5). But the maximum bending moments in the mat do not change significantly, although the plot shows that they have little higher values for stiffer soils (Fig. 5.6).

5.3.2 EFFECT OF SUPERSTRUCTURE RIGIDITY

The building model mentioned in the earlier section is analyzed with and without considering the superstructure above the mat to examine the effect of superstructure rigidity on the maximum and differential settlements and also on the maximum bending moments in the mat. Fig. 5.4 to 5.6 show the comparative variations in the total settlements, differential settlements and nodal moments in the mat. The above three figures clearly show that the building exhibits similar trends in the two different situations. The values of the settlements are little lower if we consider the superstructure (Figs. 5.4 and 5.5), while the maximum bending moment in the mat tend to increase slightly if the superstructure is included in the analysis.

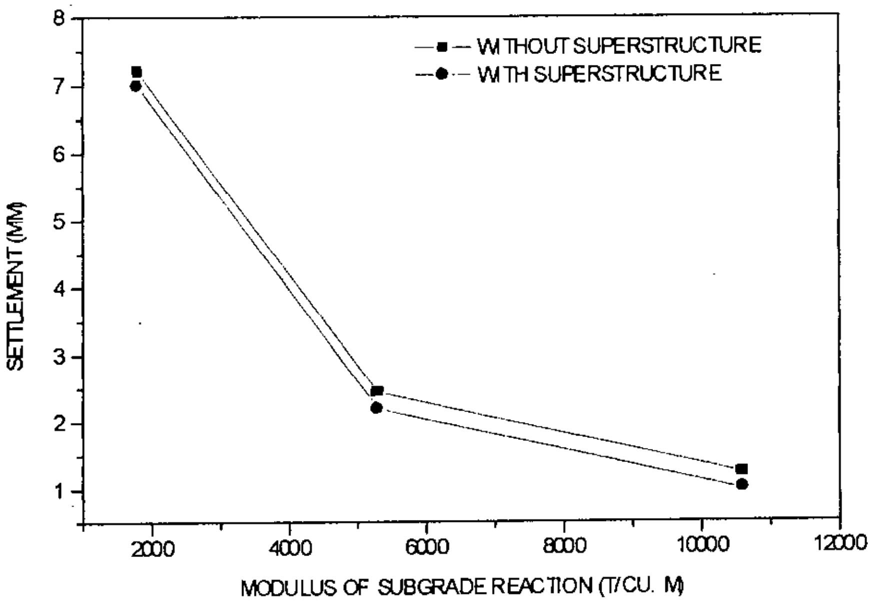


Fig. 5.4 Variations in settlements with change in soil modulus

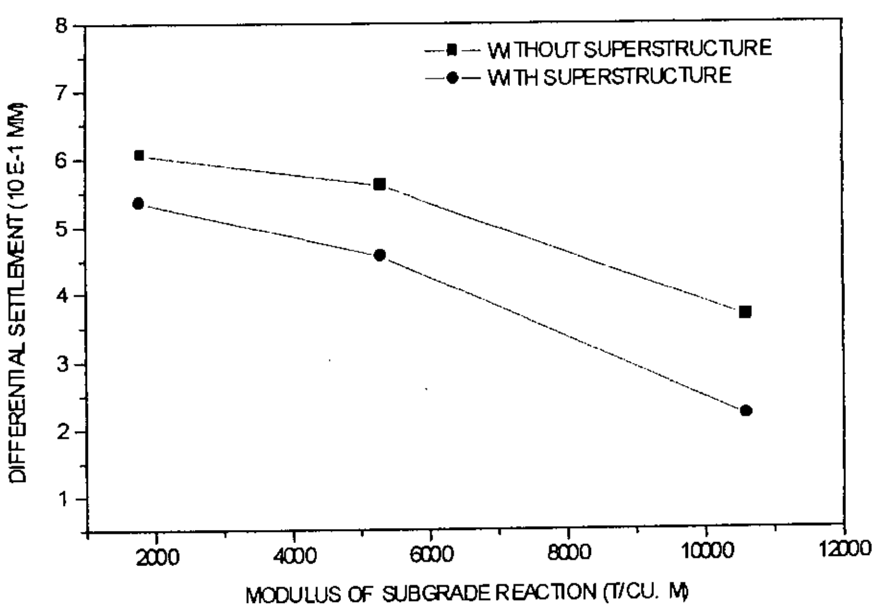


Fig. 5.5 Differential settlements for varying soil modulus

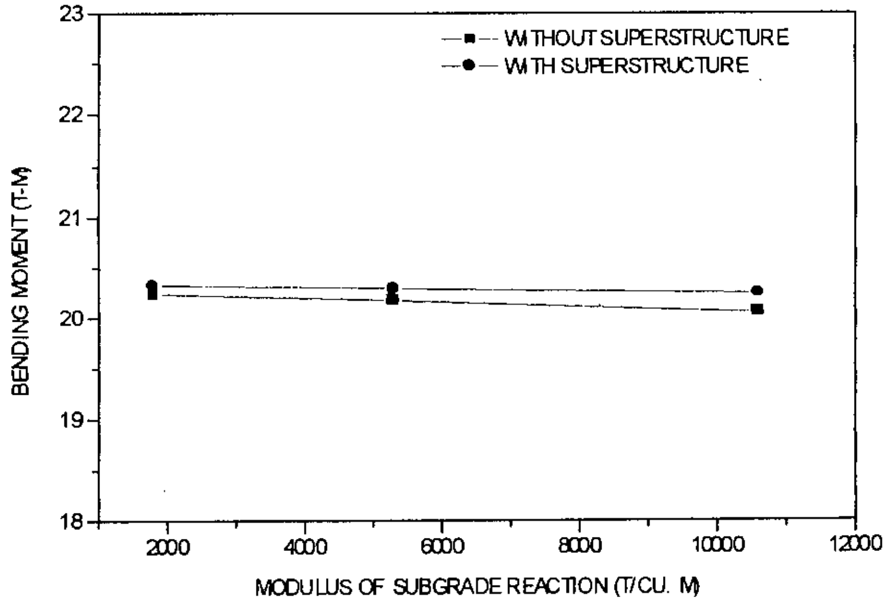


Fig. 5.6 Maximum bending moments in the mat for different soil modulus

5.3.3 EFFECT OF MAT FLEXIBILITY

The effect of mat flexibility on the response of the mat foundation is studied by varying the mat thickness. Fig. 5.7 to 5.9 show the results of the analyses when the superstructure is not considered. The findings of the same analyses considering the superstructure is plotted in Fig. 5.10 to 5.12. It is observed that the foundation flexibility does not have significant effect on the total settlement (Fig. 5.7 and 5.10). But the differential settlements are found to be significantly different with variation in the mat thickness (Fig. 5.8 and 5.11). The maximum bending moments too are different for the three cases (Fig. 5.9 and 5.12).

From the two sets of plots (with and without the superstructure) we see that the results are similar for both the situation with and without considering the superstructure with the mat. But the values of the settlements (both total and differential settlements) are little less if the superstructure is considered, while there is slightly greater moments in the mat when the superstructure is considered.

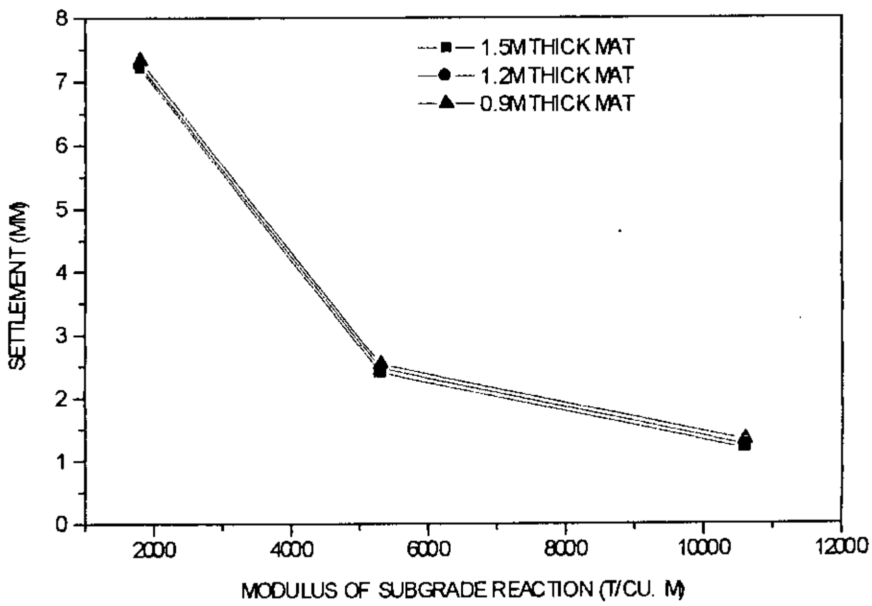


Fig. 5.7 Effect of foundation flexibility on total settlement without considering the superstructure

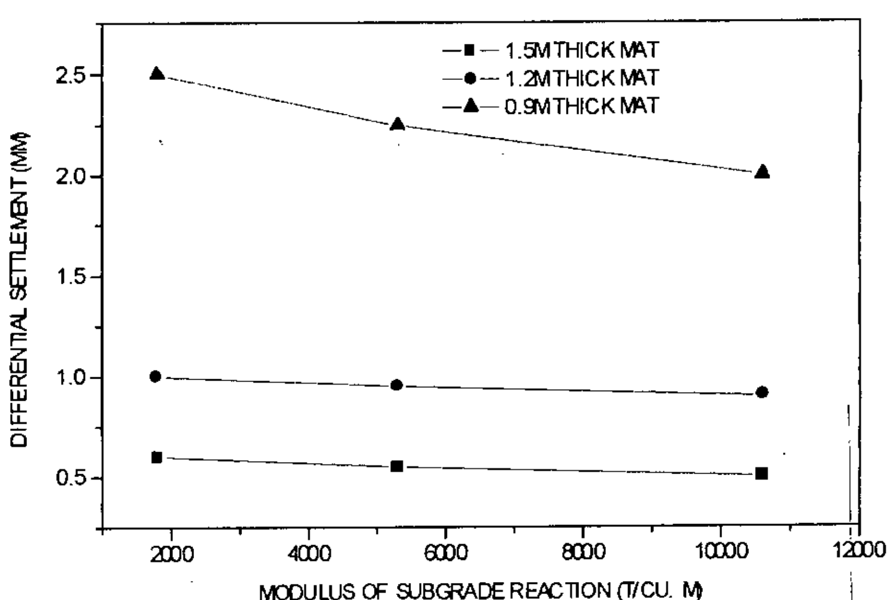


Fig. 5.8 Effect of foundation flexibility on differential settlement without considering the superstructure

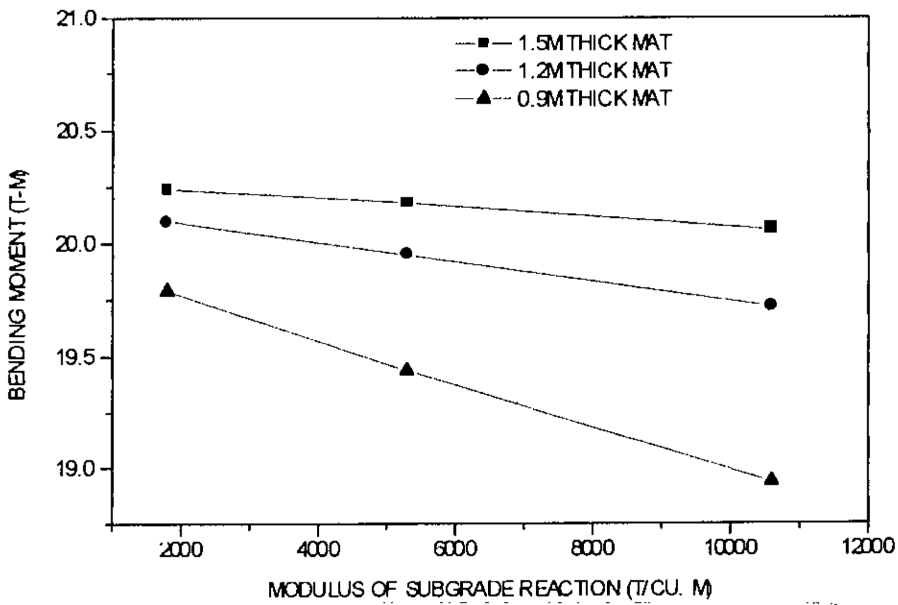


Fig. 5.9 Change in maximum bending moment in the mat when the superstructure is not considered

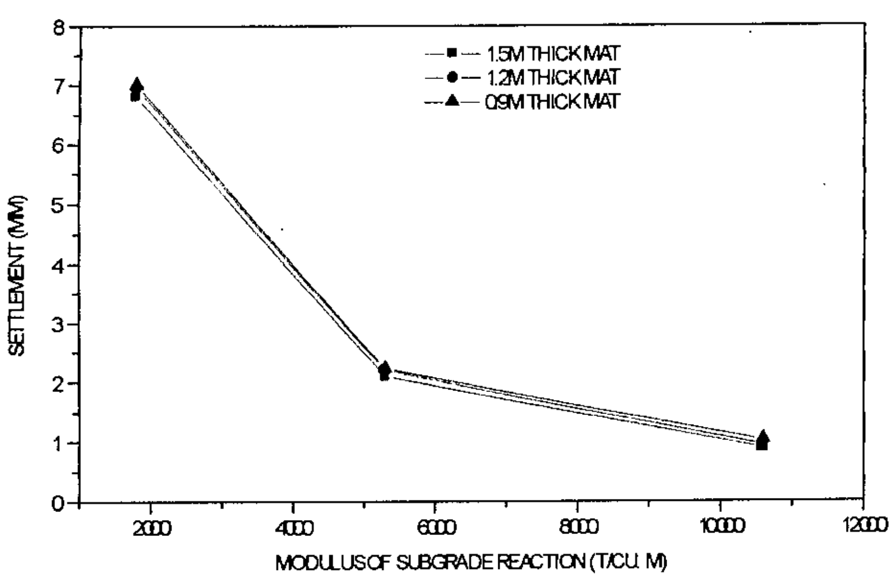


Fig. 5.10 Building settlements considering the superstructure

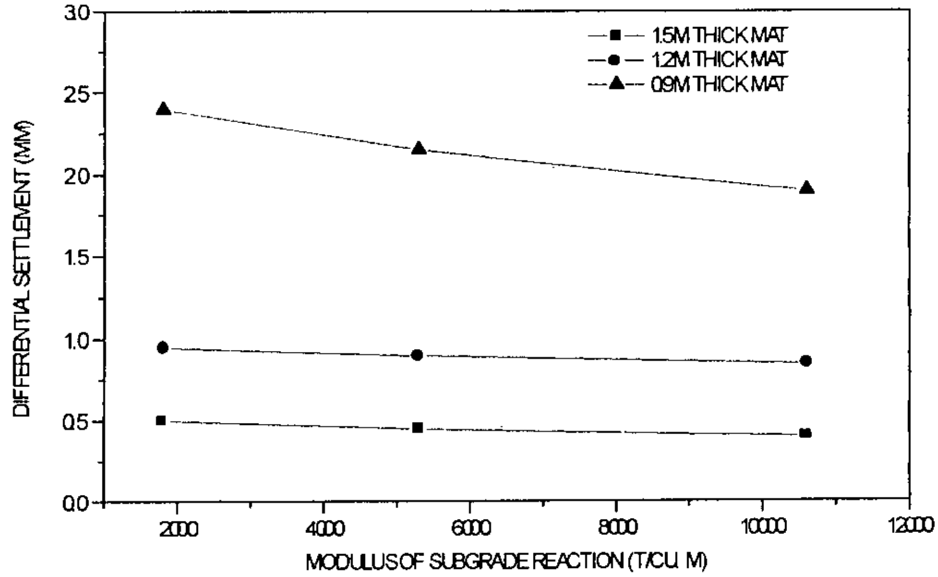


Fig. 5.11 Differential settlements for different soils considering the superstructure

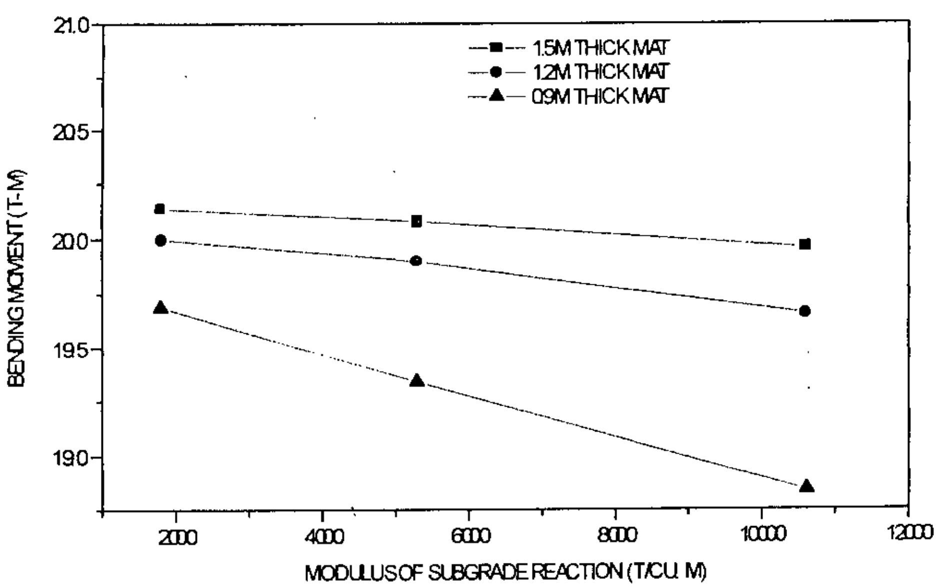


Fig. 5.12 Maximum bending moments in the mat (the superstructure is considered)

CHAPTER 6

SSI EFFECTS ON SEISMIC RESPONSE OF TALL BUILDINGS

6.1 GENERAL

Conventional seismic design of buildings normally ignores the influence of soil-structure interaction. Fixed-base assumption of buildings lead to conservative results, since soil-structure interaction generally results in reduction of structural response. The effect depends on various parameters including soil stiffness and damping, ground motion characteristics and building properties. But the effects cannot be precisely defined by any simple method. Simplified analysis procedures to account for soil-structure interaction have been presented (Veletsos et al., 1988), but their validity for varying conditions of building configuration and earthquake characteristics need to be examined. This research uses two and three-dimensional numerical models to study the effects. Buildings having six to ten story and with mat foundations are analyzed to observe the effects of different building and foundation parameters.

6.2 DYNAMIC SOIL-STRUCTURE INTERACTION PHENOMENA

Soil-structure interaction influences building response in two opposite ways. Inertia forces in a building generated during an earthquake causes deformation of the underlying soil. Horizontal base shear in the building causes horizontal soil deformation, while moment about the base due to inertia forces in a tall building causes rocking motion. Horizontal soil flexibility results in elongation of building period. Also energy is dissipated by waves propagating into the underlying soil. Both these effects are usually beneficial from structural response point of view. On the other hand, rocking motion of tall buildings due to soil flexibility tends to increase acceleration of the building masses. This results in increased shear forces.

6.3 BUILDING-SOIL MODEL

6.3.1 2D ANALYSIS MODEL

A 7.3 m (2L) by 3.66 m (2B) rectangular reinforced concrete building (Fig. 6.1) of height varying from 18.3 m (six story) to 30.5 m (ten story) is considered. The following parameters are used for the building model:

(a) Mat embedment E = 1.83 m, (b) Mat thickness = 1.5 m, (c) Load per floor = 956 kg/m^2, (d) Square column size = 305 mm, 356 mm and 432 mm for 6 story, 8 story and 10 story respectively, (e) Beam depth = 457 mm, (f) Young's modulus of concrete = 210970 kg/cm^2.

Horizontal soil stiffness and damping values are calculated using mat dimensions (L/B = 2), mat embedment (E/B = 1) and soil properties. Equivalent horizontal springs with viscous damping are assigned. The density and Poisson's ratio of soil are assumed to be 1762 kg/m^3 and 0.35 respectively, while the shear modulus of soil is varied to obtain shear wave velocities (V_s) of 98m/sec to 370 m/sec. The building model is subjected to ground motion records of five different earthquakes described in Sec. 4.6 using the nonlinear finite element program ANSR.

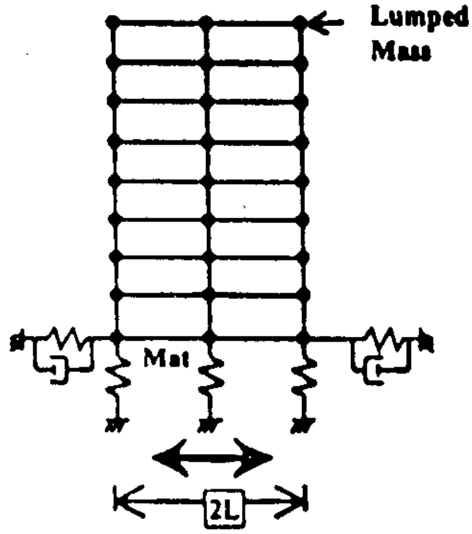


Fig.6.1 2D Analysis Model for Rectangular
8 Story Building

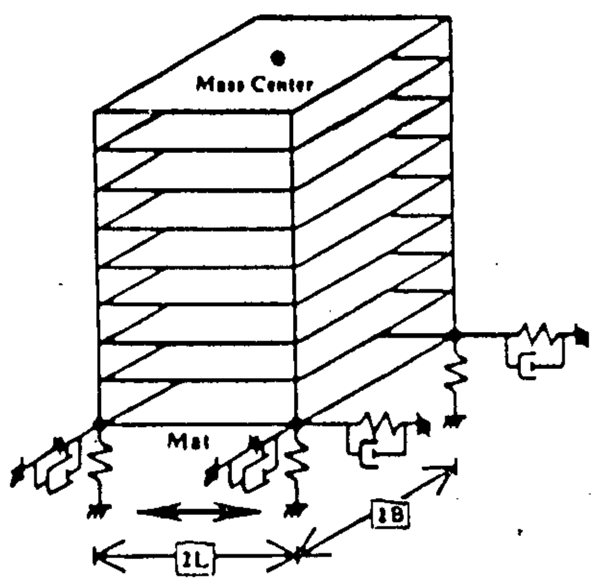


Fig.6.2 3D Analysis Model for Square
8 Story Building

6.3.2 3D ANALYSIS MODEL

A 3.66 m by 3.66 m square eight story building (Fig. 6.2) with L/B = 1, E/B = 1, and floor mass lumped at the geometric center of floor, is subjected to the above mentioned earthquakes. The column section is 356mm square and other building and soil parameters are similar to those mentioned for 2D analysis (Sec 6.3). This model is used for time-history analysis by both ANSR and ANSYS. Besides, modal analysis is performed by ANSYS to obtain the natural frequencies, and mode shapes of the building.

6.4 PARAMETRIC STUDY

Both the two-dimensional and three-dimensional models are subjected to various foundation conditions, building properties and ground motion records to have understanding about the effect of different parameters. The findings for each of them are discussed separately in the following sections.

Observations of the building responses are based mainly on two values: maximum base shear and maximum building drift. The maximum total horizontal shear forces in all the columns at the ground floor level is taken as the maximum base shear while the maximum relative displacement between the top and at the ground floor level is termed maximum building drift. These two values indicate the element stresses and deformation of the system respectively.

6.4.1 EFFECT OF SOIL STIFFNESS

The most important parameter in the whole building-soil system is the soil stiffness. It gives an idea about the characteristics of the geologic media beneath the building. In the present study, different types of soils are identified based on the values of shear wave velocities in that particular soil.

Let us first consider the 2D model of the two-bay structure (Fig 6.1). Fig 6.2, 6.3 and 6.4 present the base shear coefficient for 6 story, 8 story and 10 story buildings respectively as a function of the soil shear wave velocity V_s. The same analysis with the 8 story 3D model shows trend similar to the 2D analysis (Fig. 6.6). The general trend found from these figures is that the base shear coefficient increases with increased soil stiffness. In other words, base shear coefficient decreases with increased soil flexibility. So we can say that considering soil-structure interaction reduces the structural response. This is more true for soft soils which have lower shear wave velocities. Soil-structure interaction can result in reduction of base shear by as much as 40%. But the effect may not be that much significant for stiffer soils. In fact, the base shear at V_s = 370 m/sec is found close to the corresponding value for fixed-base condition. However, in some cases, base shear increases with reduction of soil stiffness, which we can see here for the Taft earthquake.

Another important observation is that the building drift generally decreases with increased soil stiffness (Fig. 6.7), indicating that rocking is a dominant mode of vibration.

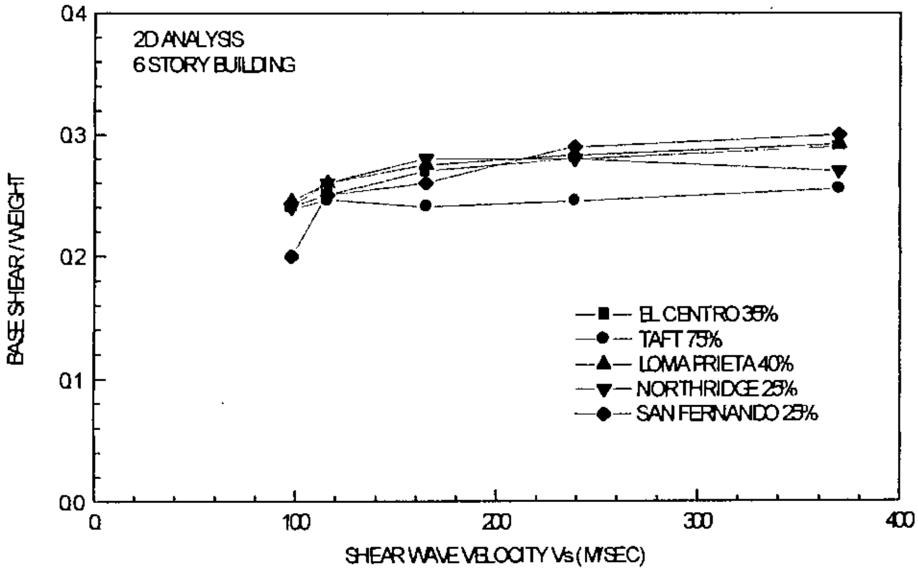


Fig. 6.3 2D analysis of 6-story building

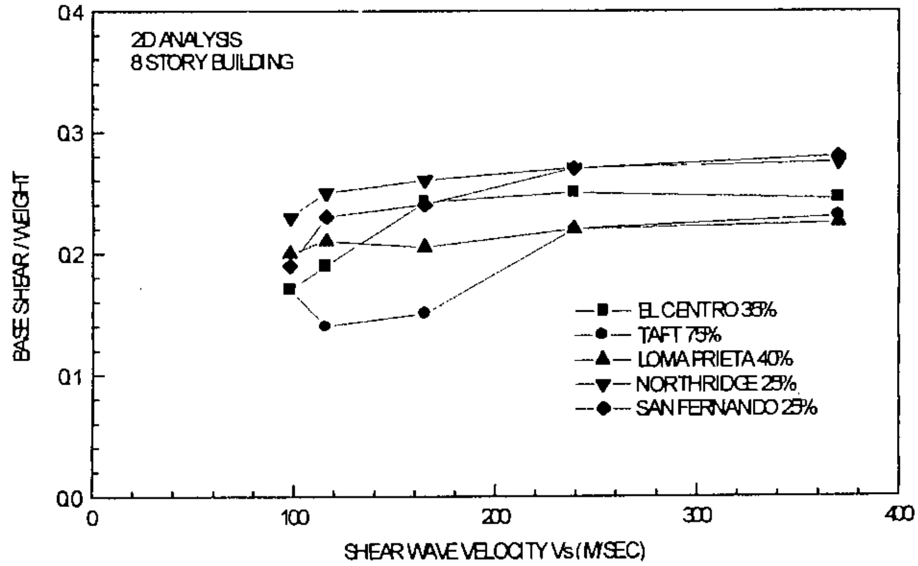


Fig. 6.4 2D analysis of 8-story building

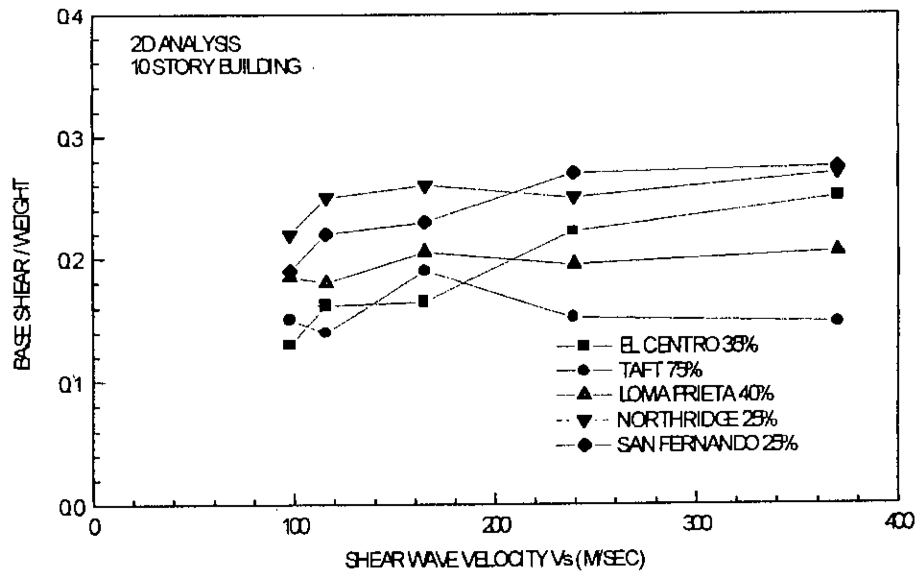


Fig. 6.5 2D analysis of 10-story building

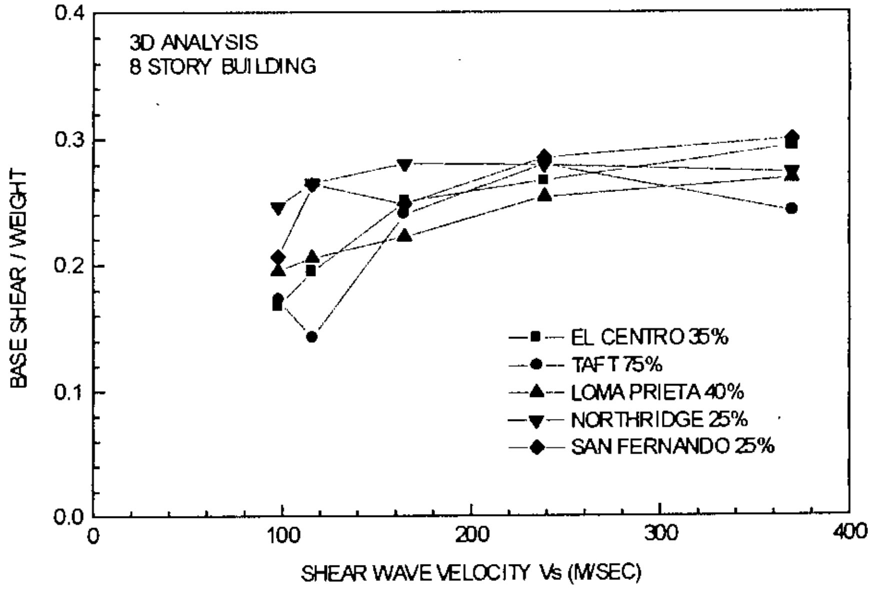


Fig. 6.6 3D analysis of 8-story building

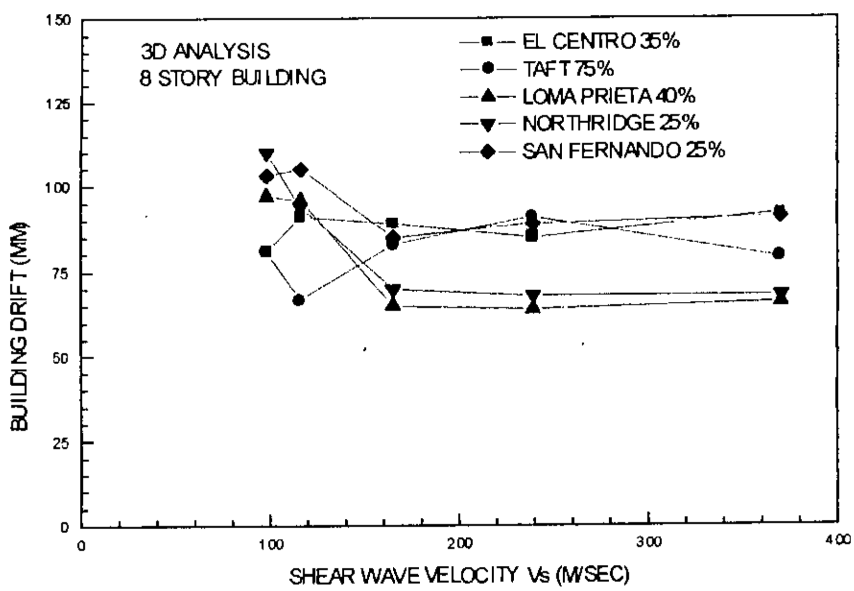


Fig. 6.7 Building drifts for different soil conditions

6.4.2 EFFECT OF BUILDING HEIGHT

The height of the building is an important factor in determining the flexibility of the building superstructure. Building from 6 to 10 story high are considered in this research. From Figs. 6.2 to 6.4, we can see that the range of base shear ratios get lower as the building height is increased. That means the building becomes more flexible as more stories are added. To have a better idea, the results of 6, 8 and 10 story building are plotted separately for 35% El Centro earthquake in particular (Fig. 6.8).

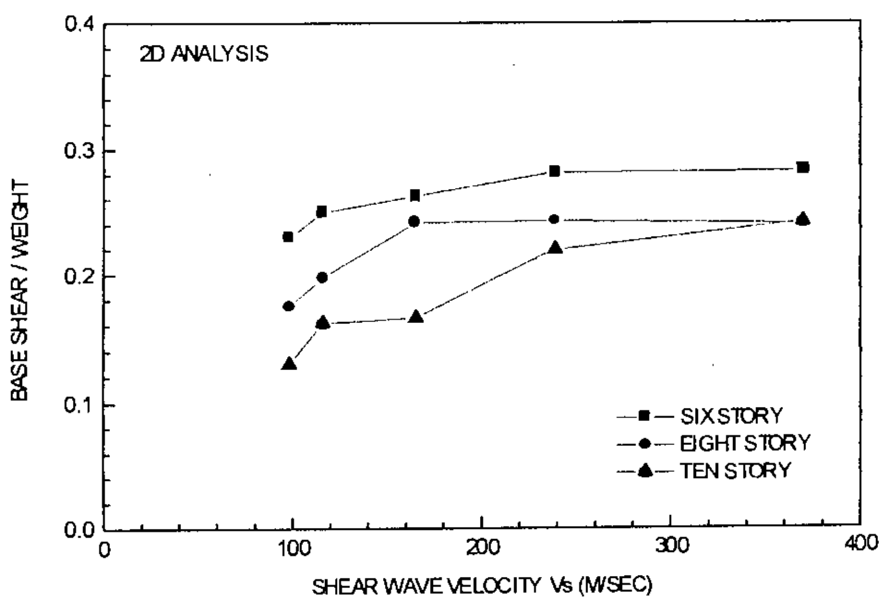


Fig. 6.8 Effect of building height on base shear coefficient

6.4.3 TORSIONAL EFFECTS

To investigate the torsional effect on the structure, the mass of the symmetric-plan building is positioned 305 mm eccentrically in one direction at all floor levels. Applying earthquake motion to this model for varying soil conditions we see that the base shears increase with increased soil flexibility, a trend similar to the symmetric mass building (Fig.6.9). But the values of the base shears are little higher than the symmetric mass condition. This is understandable due to the addition of torsional shear to the structure. Fig. 6.10 shows the comparative results of the two cases for the 35% El Centro earthquake.

Due to the torsional effects, there is some shear in the transverse direction of the building. These are much lower than the respective values in the major direction. But it is noted that they also follow the same trend of increasing for stiffer soils, as those in the other direction. The base shear coefficient in this direction for El Centro earthquake is always less than that (0.039) for fixed base case.

The rotations at the top story due to the torsional motion are also observed. Fig. 6.11 shows that the value of the rotation increases with increased soil stiffness. For the El Centro earthquake, the torsional rotation of the topmost floor is about 0.2° for fixed base condition while that for the flexible soil condition is always less.

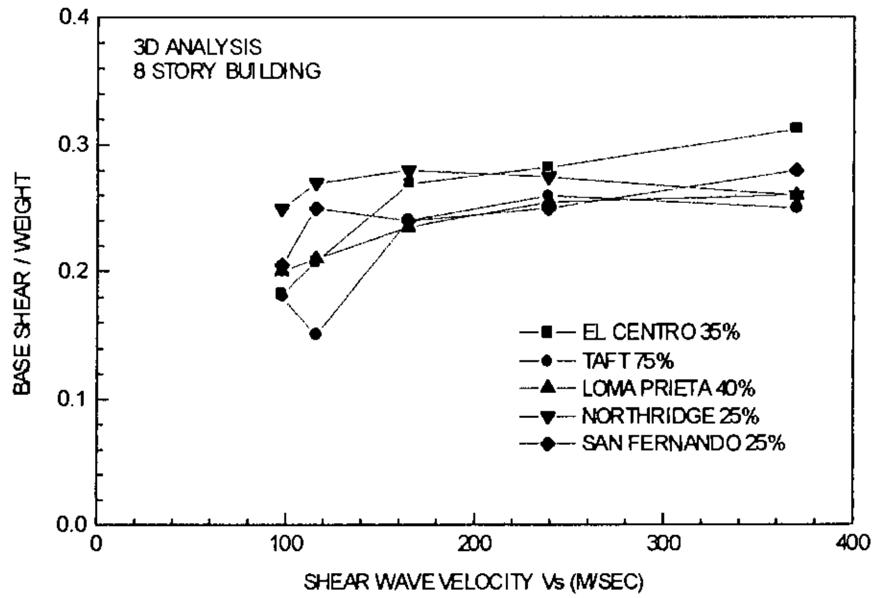


Fig. 6.9 3D analysis of 8-story building with mass eccentricity

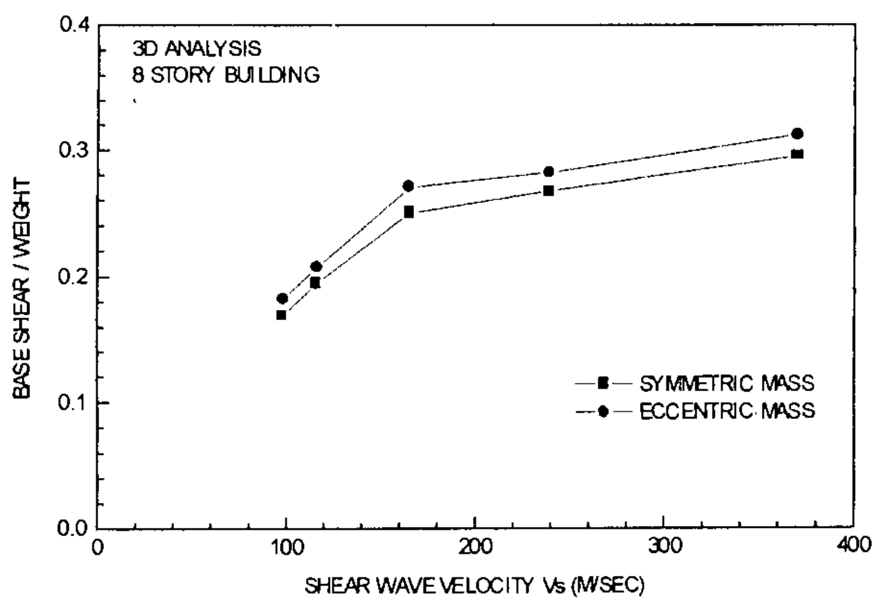


Fig. 6.10 Comparison between the results with and without mass eccentricity

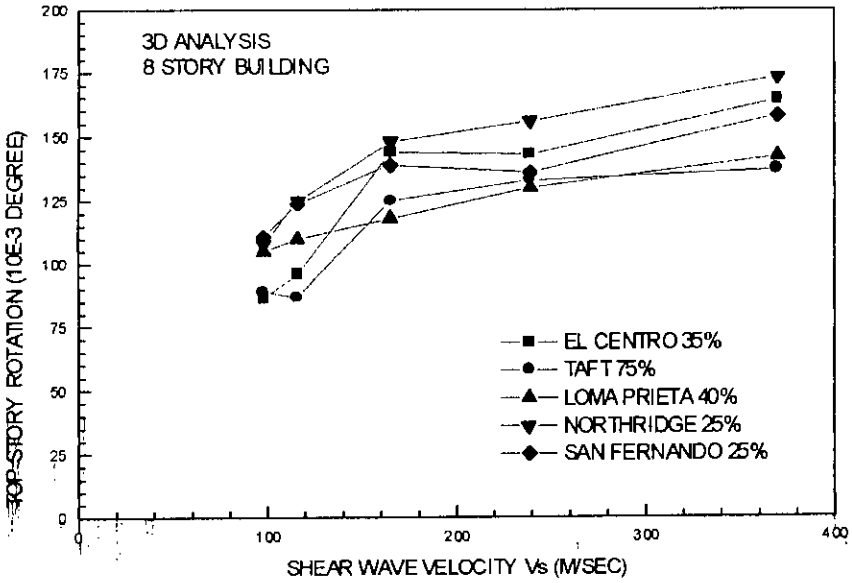


Fig. 6.11 Top story rotation for the building with mass eccentricity

6.4.4 EFFECT OF GROUND MOTION CHARACTERSTICS

As mentioned earlier, the response of the structure can be influenced by the characteristics of the ground motion. This is particularly evident for the Taft earthquake in our study, which may be considered to have a wider range of dominant frequencies. Also strong motions like the one of San Fernando earthquake show that results may be a little less predictable for such cases. So we need to consider a wider range of variations in results from analyses with different earthquake ground motions in order to avoid potential mistake through over-simplification of the phenomena.

6.5 VERIFICATION OF RESULTS

Transient dynamic analysis was performed using the full method which uses the full system matrices to calculate the transient response (no matrix reduction). The results were compared with those of the time-history analysis by ANSR for verification.

The 3D model of the 8 story building (Fig. 6.12) is analyzed to compare the results with those of similar analysis with ANSR. The geometry and the material properties are the same for the two models. The only difference is that the realistic thickness (20 cm) is used for the floor slabs while they are considered as rigid diaphragms in ANSR. Besides, global damping value of 2% of critical is used for simplification in both the cases. In the other cases, separate values of damping are used for different sets of materials. The two models are analyzed for the N-S component of 35% El Centro Earthquake.

The analyses were performed for two different cases, one with the fixed-base condition and the other for the models with springs at the base. The top story deflections (obtained from both ANSR and ANSYS) in the two cases are plotted in Figs. 6.13 and 6.14 respectively. It is very clear that the results from the two different analyses are in excellent match. Same degree of similarity is found between the two sets of results if other parameters like base shears are considered.

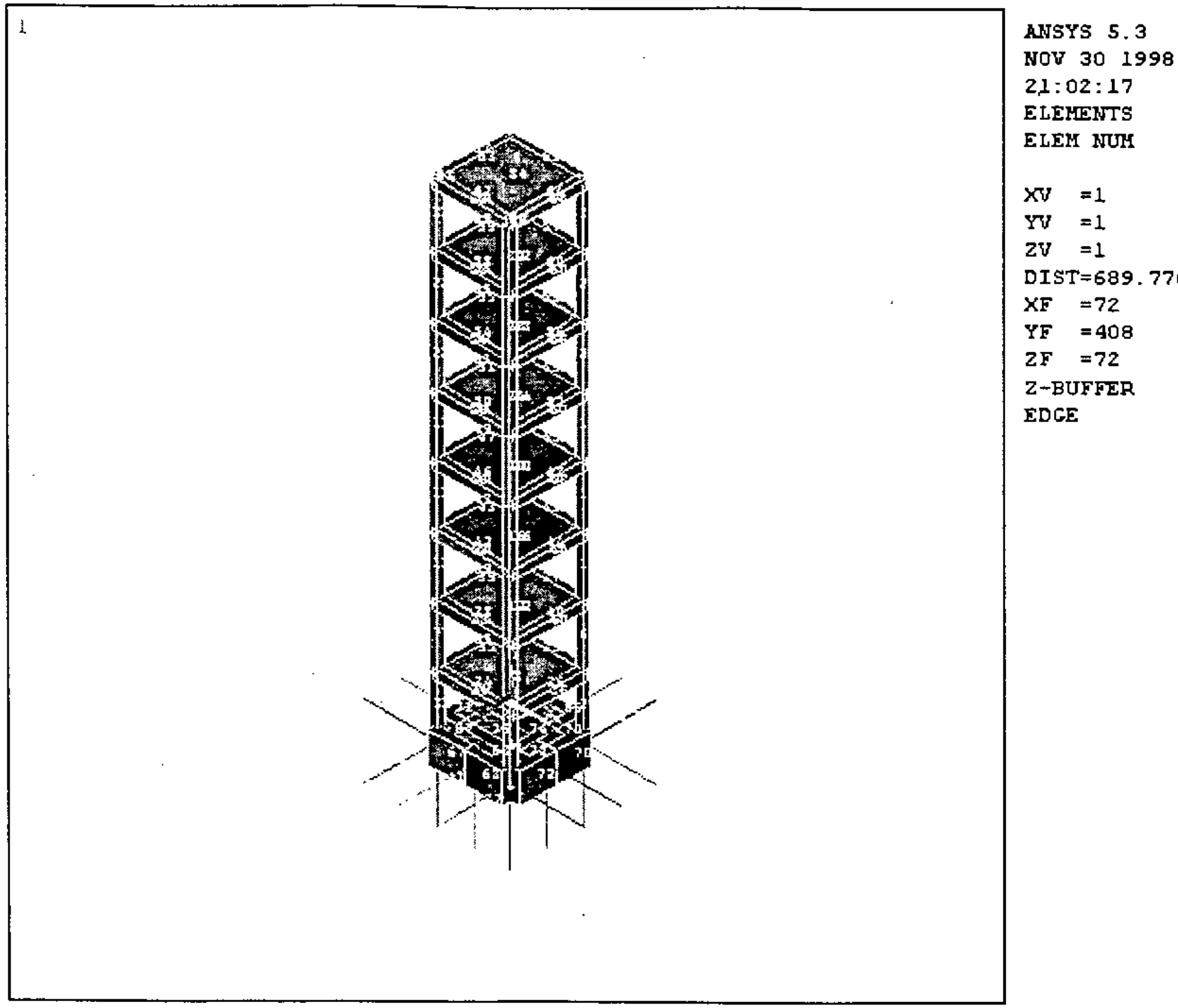


Fig. 6.12 ANSYS model of 3D building

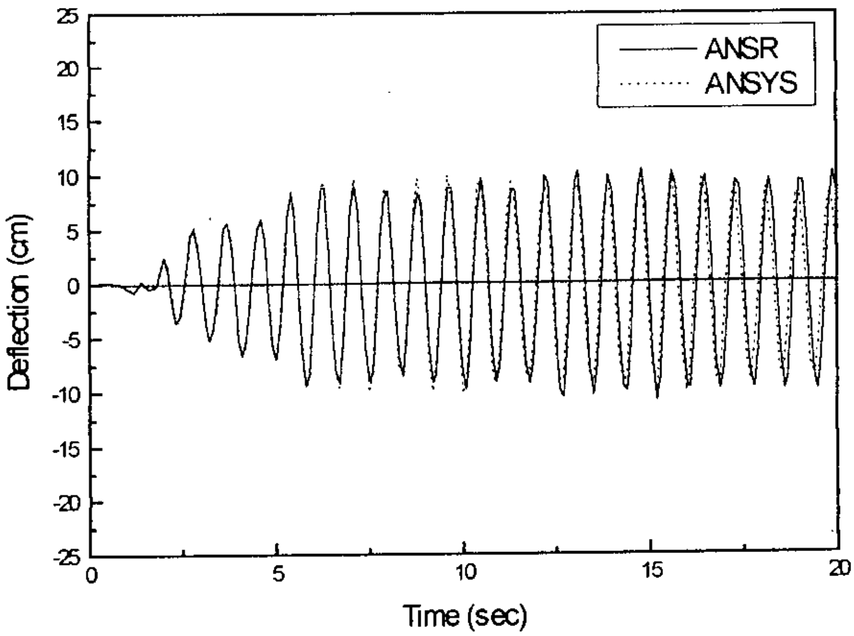


Fig. 6.13 Comparison of results for 8-story building with fixed base

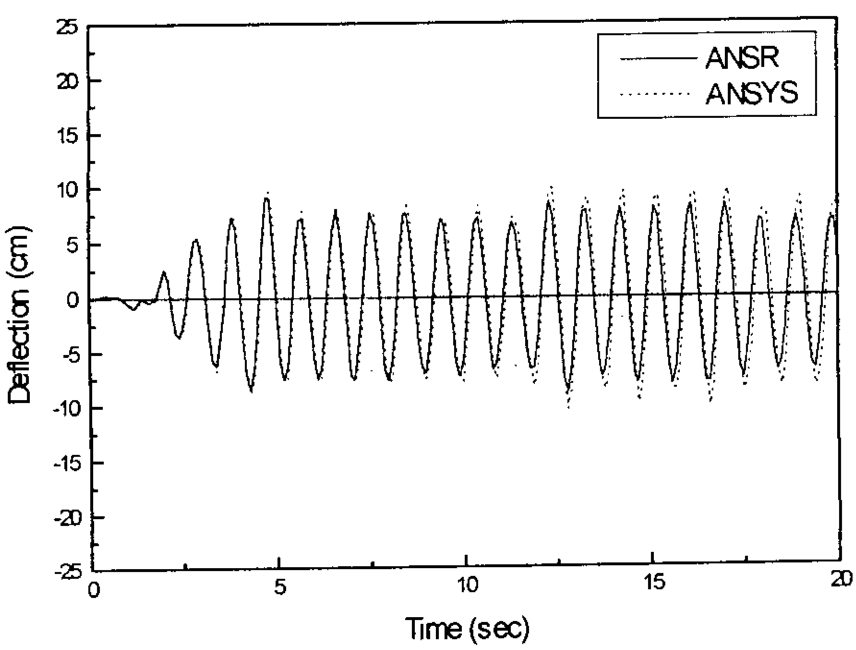


Fig. 6.14 Comparison of results for 8-story building with springs

6.6 MODAL ANALYSIS

Modal analysis of a typical 3D model was performed with ANSYS to obtain the natural frequencies and mode shapes of an eight-story building with mat foundation. The shear wave velocity (Vs) of the soil is 165 m/sec. The natural frequencies indicate that the model of the building is similar to any real building in its behavior. Besides, the comparative analysis between a fixed-base structure and the same with springs at the base give valuable information about the effect of soil-structure interaction on the natural frequencies of the building.

The 8-story building model was used to get the first 10 modes of vibration and the corresponding frequencies. The natural frequencies for the first ten modes are given in Table 6.1. As we can see, both translational and torsional modes are included. The frequencies of the translational modes are the same in the two directions for the square-shaped model. The mode shapes are presented in Figs. 6.15 to 6.21.

The same building model was also analyzed with the fixed-base arrangement (Table 6.1). The results indicate that the frequency of vibration decreases for the flexible-base condition. That means the period will be higher for buildings with flexible foundations. This effect is more prominent for the first few modes of vibration.

Table 6.1 Summary of results of modal analysis

		Flexible base (with springs)		Fixed base	
Mode	Direction	Frequency (Hz)	Period (sec)	Frequency (Hz)	Period (sec)
1	X	1.06	0.95	1.19	0.84
2	Z	1.06	0.95	1.19	0.84
3	θ	1.47	0.68	1.47	0.68
4	X	3.69	0.27	3.75	0.26
5	Z	3.69	0.27	3.75	0.26
6	θ	4.41	0.23	4.39	0.22
7	X	6.66	0.15	6.60	0.16
8	Z	6.66	0.15	6.60	0.16
9	θ	7.36	0.14	7.37	0.14
10	X	9.51	0.10	9.54	0.10

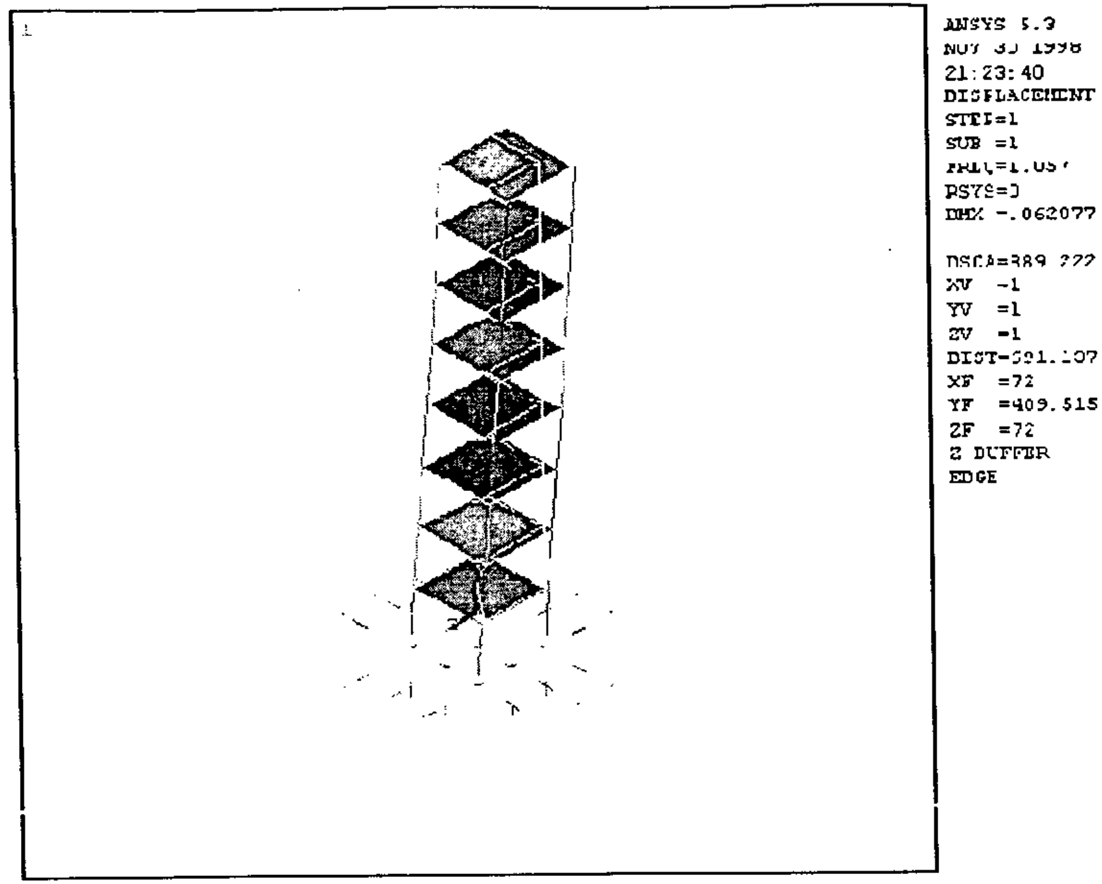


Fig. 6.15 First translational mode in x-direction

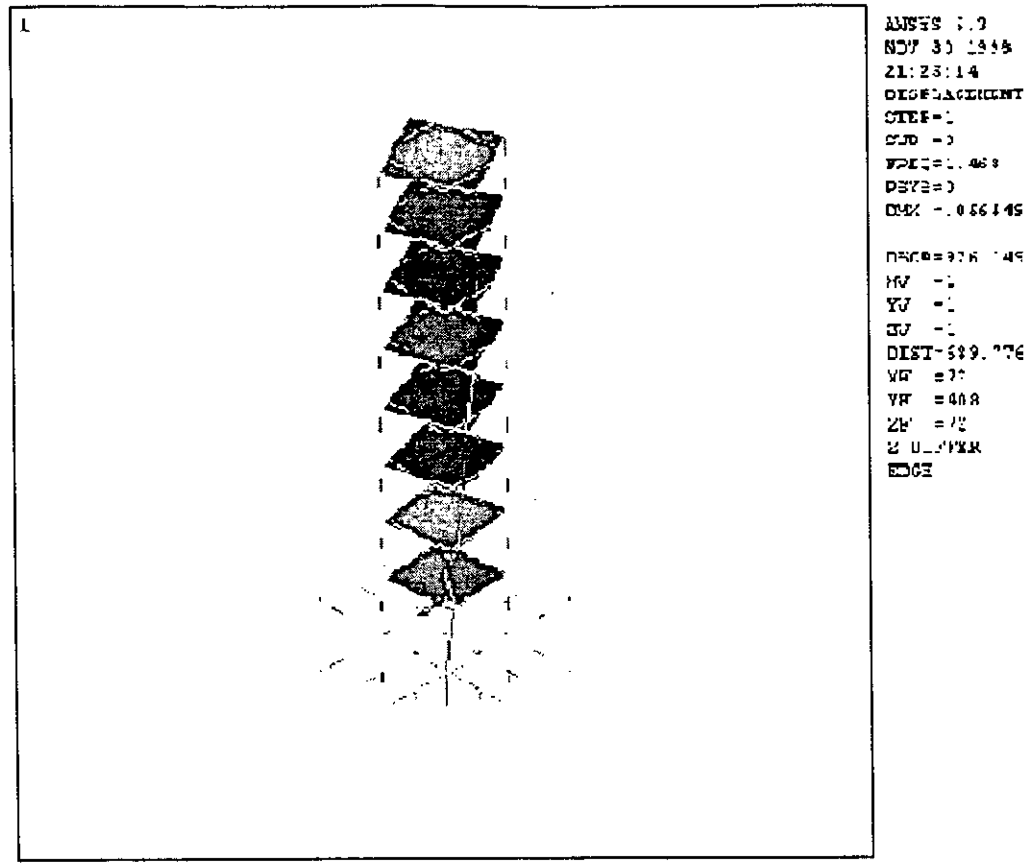


Fig. 6.16 First rotational mode

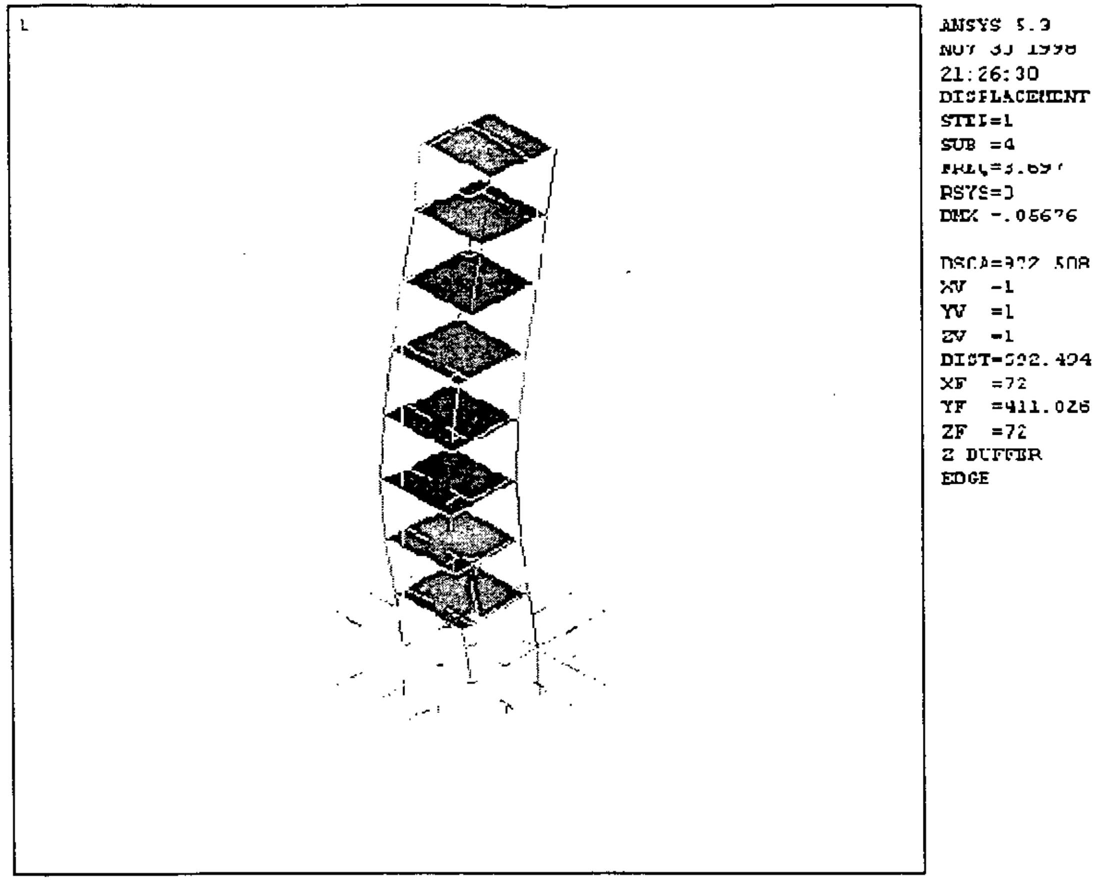


Fig. 6.17 Second translational mode in x-direction

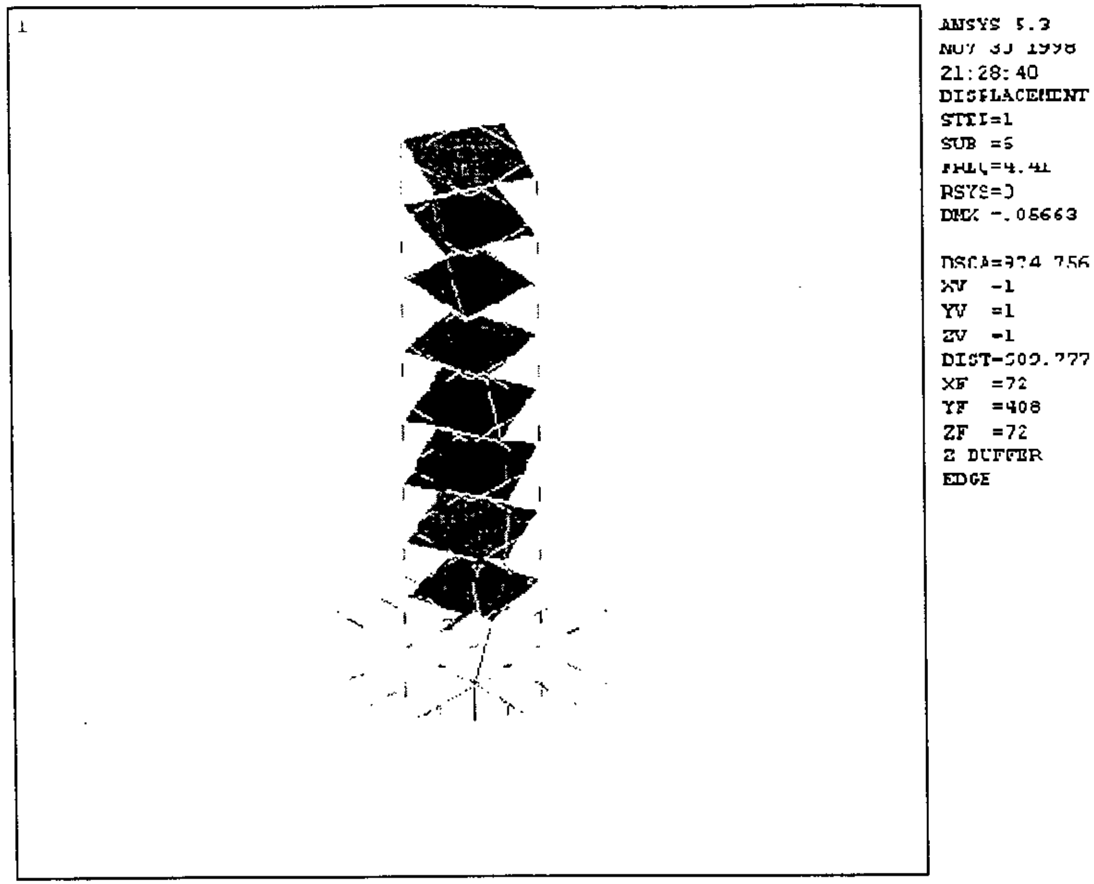


Fig. 6.18 Second rotational mode

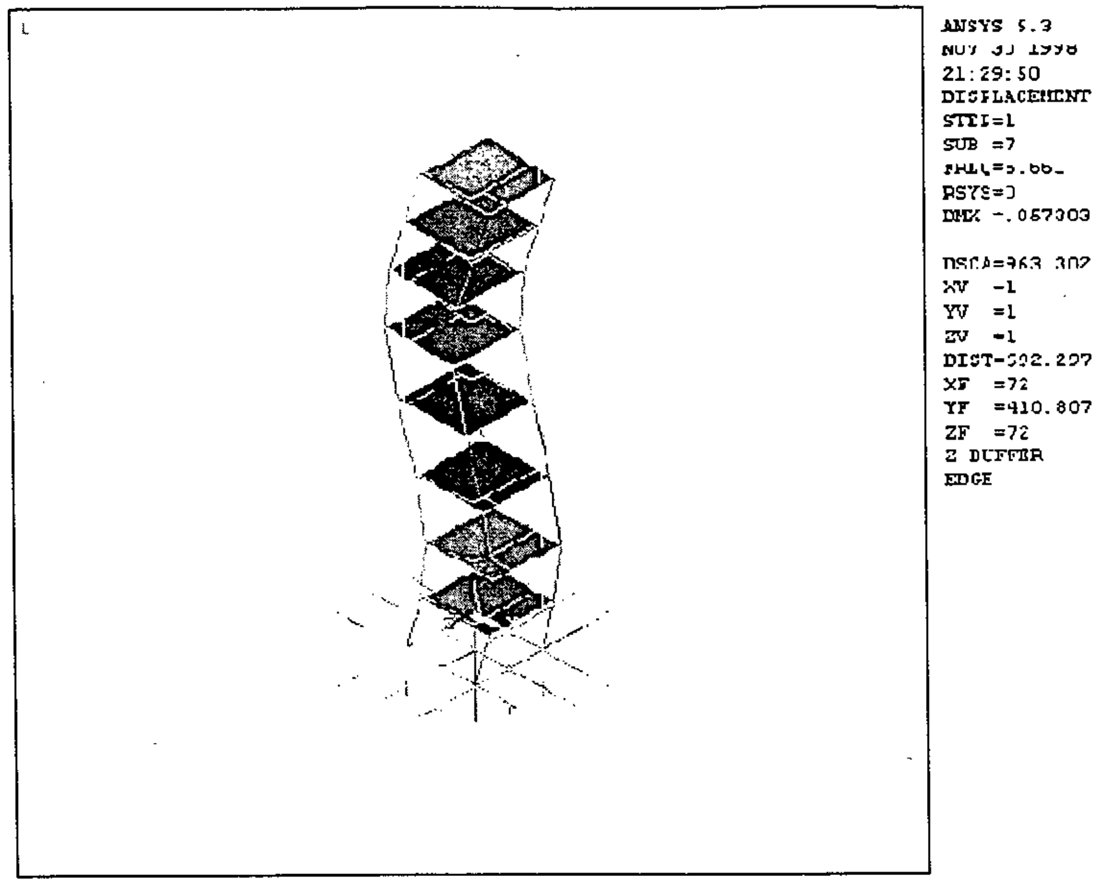


Fig. 6.19 Third translational mode in x-direction

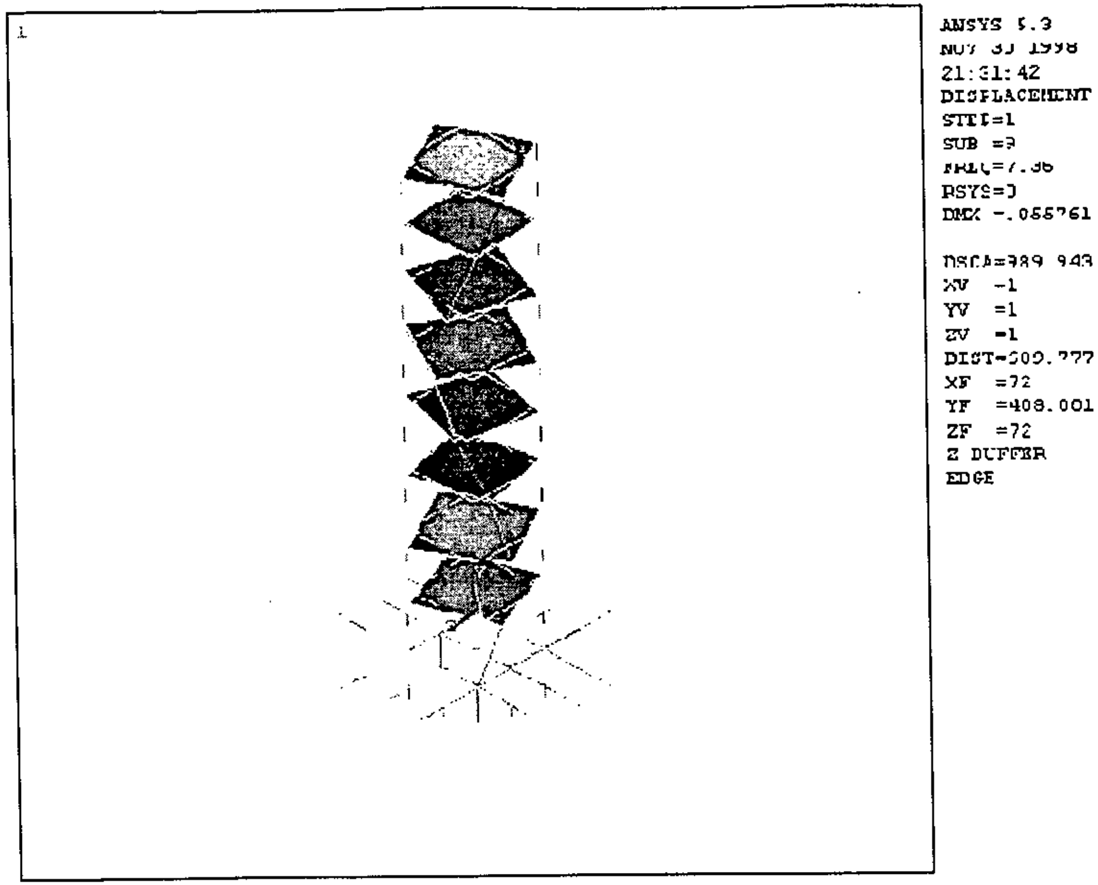


Fig. 6.20 Third rotational mode

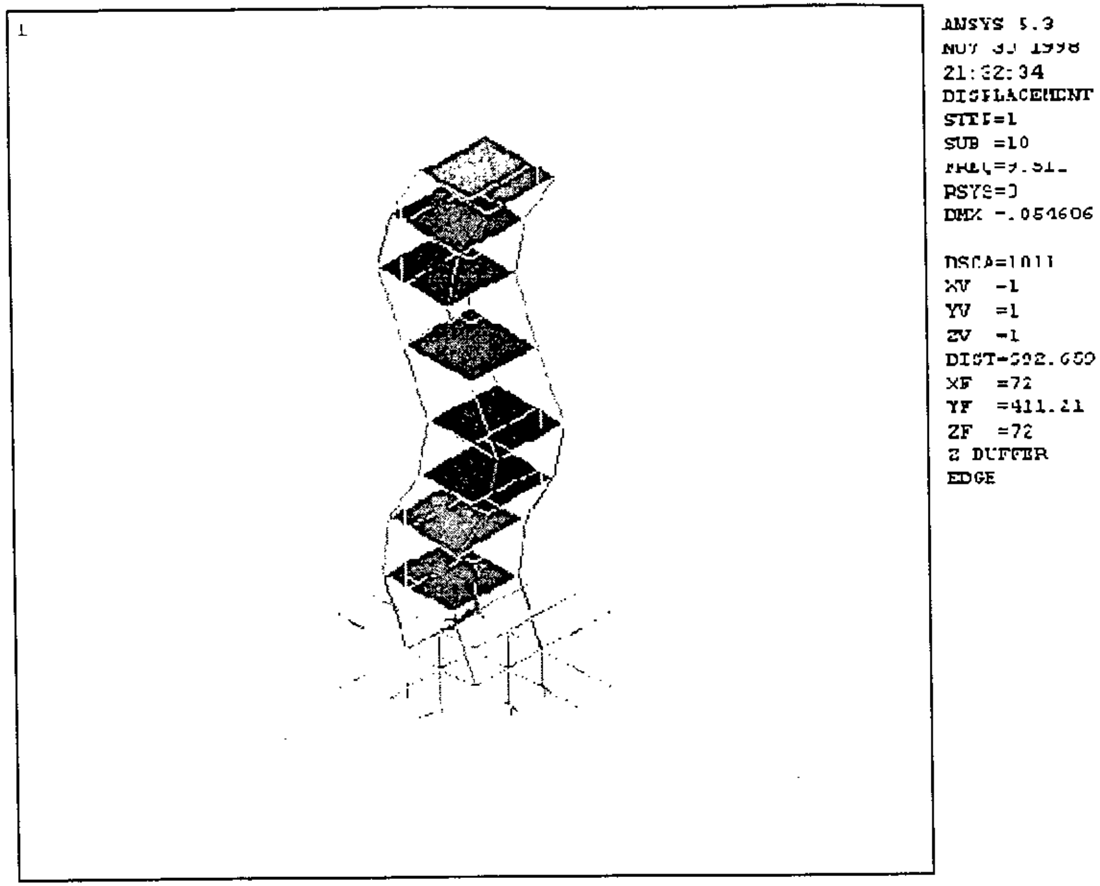


Fig. 6.21 Fourth translational mode in x-direction

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

7.1 CONCLUSIONS

Effects of soil-structure interaction on response of tall buildings with mat foundation are investigated in the present study. The analyses were performed for both static and seismic loading. Based on the numerical studies on mat foundation the conclusions are as follows:

- The mat can be modelled as a grid using beam-column element of proportionate dimensions to represent the thick slab. The grid elements have slightly larger bending moments at the nodes in comparison with the thick plate elements, but the deflections in the two cases are almost the same.
- For static loads, the maximum settlements of the building decrease with increase in soil stiffness, but not in a linear proportion. The trend is same whether the superstructure is considered or not, but the values of the settlements are little higher if the superstructure is not considered.
- The differential settlements also decrease in similar fashion with and without the superstructure, but more sharply if the superstructure is considered.
- The maximum bending moment in the mat remains almost the same for the range of practical values of soil modulus.
- The mat thickness does not influence the maximum settlement significantly, but does so in the case of differential settlements. Also the bending moments are smaller for thicker mats.

For seismic loading, based on the numerical studies on six to ten storied buildings with relatively large height to width ratio, the following conclusions can be drawn:

- The period of vibration of the building increases if the soil-structure interaction effects are considered.
- The general trend is that the base shear coefficient decreases with increased soil flexibility. In other words, considering soil-structure interaction generally reduces seismic forces. For stiff soils, the response is almost the same as the fixed-based condition.
- The total building drift increases with increase in soil flexibility, indicating that rocking is a dominant mode of vibration.
- The base shear coefficients are less for more flexible buildings with greater heights.
- SSI Effect not only depends on building configuration but also is very much influenced by ground motion characteristics.
- Torsional response and increased base shear is observed in buildings with mass eccentricity. Due to soil-structure interaction effects the base shear coefficient as well as the torsional rotation at the topmost floor is reduced.

7.2 RECOMMENDATIONS FOR FUTURE STUDY

From the present research, the following recommendations can be made for future study:

- The models were analyzed in linearly elastic range in this study. Nonlinear analysis of the building models may be performed for seismic analysis.
- Buildings with different shapes and geometries should be used to have further knowledge about the response.
- Buildings with multiple bays should be analyzed to more closely represent actual structures.
- The effect of frequency dependent soil parameters may be studied.

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