Seismic Analysis of Structures

Project Work 1995-96

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SYNOPSIS

This project work deals with analysis of multistoreyed buildings subjected to earthquake forces in conjunction with dead loads and live loads. There are basically two methods adopted in seismic analysis namely Static (IS code method) and Dynamic analysis.

In Static analysis the time period is found out by an empirical formula and hence base shear is found. This base shear is then distributed.

In the Dynamic analysis there are two types of studies, the first one is known as free vibration analysis and the second one is forced vibration analysis. In this project, the authors have taken into consideration the free vibration analysis. A numerical method namely the ***HOLZER'S METHOD*** to find out the time period is used and hence the residual amplitude and lateral force acting at each floor level is found out

In this project, the authors have compared the static analysis and the free vibration (part of the dynamic analysis) with respect to the time period and the lateral loads.

A model structure consisting of three floors has been designed for withstanding earthquake forces. The amount of reinforcement needed for resisting the earthquake forces and normal conditions are compared .The frames are analyzed by computer program using direct stiffness method. A step by step procedure for the analysis of three storey building is presented.

LIMIT STATE METHOD OF DESIGN has been adopted throughout this project.

IBM compatible PC/AT is used

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CHAPTER-1

PREFACE

1.1 INTRODUCTION:

Earthquakes are one of the most destructive forces that nature unleashes on earth. They not only cause a loss of life and property but shake the morale of the people. Man normally looks upon mother earth for stability under his feet and when it itself begins to tremble, the shock he receives is indeed unnerving. People who had suffered a severe earthquake feel it as a nightmare for many years.

Since earthquakes are so for unpreventable and unpredictable, the only course open to us is to design and build the structures in such a manner that the loss of life and property is minimized. This project is dedicated towards this objective.

In preparing the material for this project, the authors have drawn freely on published literature on the subject which is gratefully acknowledged

More than 3,00,000 earthquakes occur in the world in every year, most of which, fortunately, are not of great intensity or strike un populated areas. However, some centres of severe earthquakes are located near cities and populated areas. An earthquake occurring in these centres results in the death of many people and causing destruction of structures that have not been designed to resist earthquake forces. The effects of the earthquake included destruction of standing structures, rupture of the ground, dislocation of the drainage system, initiation of massive slides and eruption of ground water.

Many parts of the world such as the circum-pacific belt including the western coasts of north and south America, New Zealand, south-east Asia, Japan, Mediterranean countries, west Asian countries, central Asia, Himalayan regions, Burma, Tibet and china suffer major tectonic earthquakes frequently.

1.2 ANALYTICAL METHOD:

The ***Seismic coefficient*** method of static analysis was the first method developed by Professor. Sano for analyzing the structure due to earthquake forces. The initial work on earthquake resistant design of buildings in Dynamic analysis was done in Japan by Muto and Suyehiro in 1920 and in United states by Biot(1943) and Freeman(1932). At that time the Dynamic response could not be computed due to lack of computational facilities. With the advent of computers in 1940, Biot(1943), Clough(1955), Fung and Barton(1958) and Merhant and Hudson(1962) have done research and developed the method of modal superposition.

The ***Response-spectrum*** method was first developed by Housner, U.S.A., for use in structural designs. With the advent of modern electronic computers, improved techniques of rigorous Dynamic analysis by step by step numerical integration was developed. Though a Dynamic approach to the problem of design of earthquake resistant structures is the most realistic one, the present methods of taking equivalent static approach for design purposes is considered to be sufficient for moderately high buildings.

In this project work , both the methods namely ***SEISMIC CO-EFFICIENT METHOD (STATIC METHOD)*** and ***RESPONSE SPECTRUM METHOD (QUASI-STATIC METHOD)*** have been taken for the analysis and compared.

CHAPTER-2
REVIEW OF LITERATURE

2.1 GENERAL

The loads acting on the structure in general can be classified into two categories. They are,

1) Static loads

2) Dynamic loads

The loads that are independent of time are called as Static loads. Dead loads and live loads acting on the structure fall under this category.

The loads that change quickly in time (in magnitude, direction and position) are called Dynamic loads. A consequence of the action of Dynamic loads is vibration of structures. The most common are vibration loads in which Dynamic forces change in accordance with a harmonic law. Such loads occur, for example, during rotation of parts of machines with unbalanced masses when the weight is in an eccentric position in relation to the axis of rotation.

Impact loads are characterized by a sudden and brief action of great intensity. Seismic forces are Dynamic loads that arise with the movements of the base of a structure during an earthquake

2.2 ABOUT EARTHQUAKES

Vibrations of earth's surface caused by waves coming from a source of disturbance inside the earth are described as "Earthquakes".By far the most important earthquakes from an engineering standpoint are of tectonic origin, that is , those associated with large scale strains in the crust of earth.One of the theories describing this phenomenon is termed Elastic Rebound Theory. It explains that the strain energy that accumulates due to deformations in earth mass get released through rupture when it exceeds the resilience of the storing material . The energy thus released is propagated in the form of waves impart energy to the media through which they pass and vibrate the structures standing on the earth's surface.

Fig.2.1 illustrates the phenomenon of rupture as is attributed to the elastic rebound theory. Section originally straight in its unstrained state, deforms through strain accumulation as shown in fig.2.1(a) and on rupture it breaks as shown in fig 2.1(b) resulting in a relative shift on the fault line. In some earthquakes, this type of movements on fault line appear on the surface. In most earthquakes, they lie deep underground

In course of time after a major earthquake, the ruptured mass heals and binds itself. Also during this period, friction along the ruptured surface permits the straining of mass again. When the accumulated energy exceeds the capacity of the material once again,earthquake occurs. Thus it has been occurring again and again from the same region where earthquakes have found to be occurred.

A major tectonic earthquake is generally preceded by small 'fore shocks' caused either by small ruptures (or) plastic deformations and is followed by 'after shocks' due to fresh ruptures on the readjustments of the fractured mass. A major shock may result from a rupture of rock over a length of 100 to 400 km and several kilometer wide and thick. The bigger is the mass that ruptures at one time, the bigger is the earthquake.

Small earthquakes may also be caused by volcanic eruptions, subsidence in mines, blasts, impounding of reservoirs and pumping of

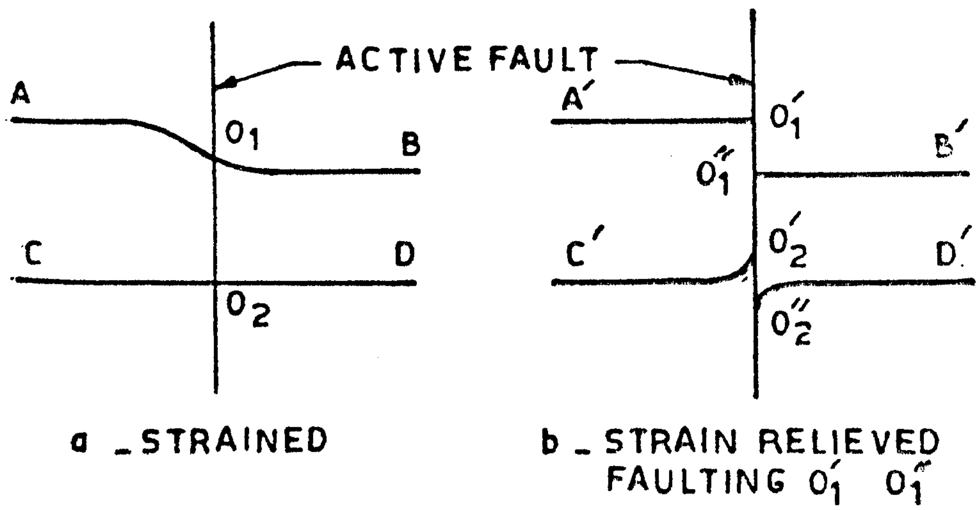


Fig. 2.1 Elastic Rebound Theory of Rupture

oils. They may cause considerable damage in small areas. But vast areas are shaken only by tectonic movements across active faults as explained above.

2.3 EPICENTER , HYPOCENTER AND EARTHQUAKE WAVES

2.3.1 EPICENTER, HYPOCENTER

The point inside the earthmass where slipping (or) fracture begins is termed as Focus (or) Hypo center and the point projected just above the focus on the earth's surface is termed as Epicenter. The position of the hypo center is determined with the help of seismograph records, which indicate the arrival times of different types of energy waves. As regards the depth of focus, it has been observed that it may lie anywhere from just below a meters to as much as 700kms, below the surface. A great majority of the earthquakes of the past had the foci within 50kms; only few were recorded with depth of focus lying between 700 to 300 kms. No earthquake has yet been recorded having focus deeper than 700 kms.

2.3.2 SEISMIC WAVES

During each earthquake of any origin elastic waves generated at the place of origin and these waves spread in all directions. It is now well established that during an earthquake three well defined types of waves are produced. These are briefly termed as P, S and L waves.

2.3.2.1 THE "P" WAVES

The P waves are primary waves(also termed "Push and Pull Waves", "longitudinal waves","compressional waves") are the fastest of the seismic waves. These are longitudinal waves in character, that is the particles vibrate in the direction of propagation. The velocity of P waves is controlled by the relationship,

$$\alpha o = \sqrt{(\mu + 2\sigma)/\gamma}$$

1. Where μ and σ are Lame's elastic constants related to the rigidity of the medium and γ is the density of the medium.

2.3.2.2 THE "S" WAVES

The "S" waves or secondary waves or shear waves are like light waves in which the particles vibrate at right angles to the direction of propagation. Their velocity is controlled by the relationship, $ß = \sqrt{(\sigma/\gamma)}$

2.3.2.3 THE "L" WAVES

The " L" waves or long waves are also called surface waves because their journey is confined to the surface layers of the earth only. In other words, they do not travel towards the interior of the earth from the point of origin of disturbances. As regards the exact nature of these waves, it has been proved that they are of two types.

The Raleigh waves in which the displacement of the particle is of a complex nature,partly in the direction of propagation and partly in right angles. The particle displacement in Love waves is practically horizontal. i.e. the direction perpendicular to the direction of its motion in its plane.

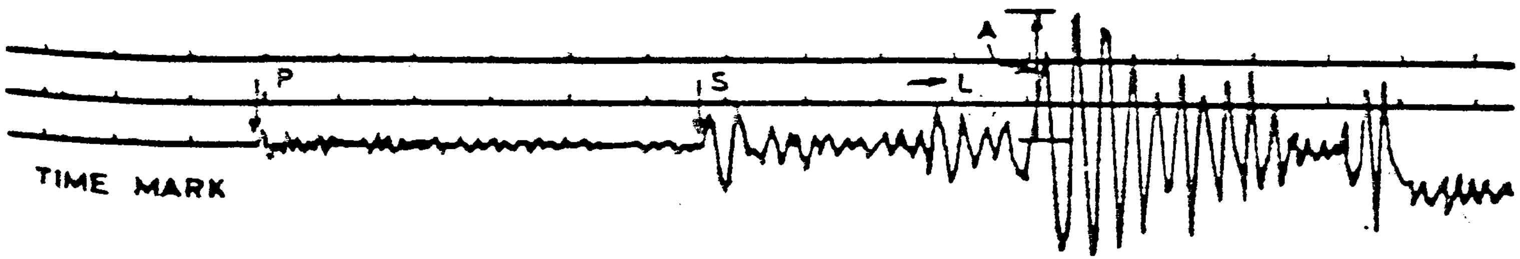


Fig. 2.2 A Typical Earthquake Record

Fig.2.2. shows a typical earthquake record of arrival times of P,S and R Waves.

2.4. INTENSITY OF AN EARTHQUAKE

It is the term expressing rating of an earthquake on the basis of its effects on the material things of the earth,the effects being observed without help of any instruments. Obviously the effects are directly related to the degree of shaking a part of the earth receives during an earthquake. They may range from simple harmless vibrations through mild jerks and shakes capable of disturbing the movable things and ringing of bells or falling of plaster to complete overturning of buildings and collapse of other structures and subsidence of crustal segments. Two scales of intensities are in vogue; the first "***Rossi-Forrel scale***",divides all the shocks into 10 arbitrary types,beginning from one and ending with 10; the second,the ***Mercalli scale*** suggests 10 types. In each scale, a number of 'Intensity' when mentioned with an earthquake denotes its rating on the basis of 'Effects' observed during and after that earthquake. Intensity is therefore, representative only of the non-instrumental observations to a great extent. Mercalli scale was later modified by Newmann and has 12 divisions as givrn below.
Modified Mercalli Intensity (MMI) scale is given in table -2.1

TABLE-2.1

MODIFIED MERCALLI INTENSITY SCALE

CLASS	DESCRIPTION OF EARTHQUAKES
I	Not felt except by a very few under specially favour able circumstances.
II	Felt only by a few persons at rest, specially on upper floor of buildings; and delicately suspended objects may swing
III	Felt quite noticeably indoors, specially on upper floors of buildings but many people do not recognize it as an earthquake; standing motor cars may rock slightly; and vibration may be felt like the passing of a truck.
IV	During the day felt indoors by many, outdoors by a few; at night some awakened, dishes, windows, doors disturbed; walls make cracking sound, sensation like heavy truck striking the building; and standing motor-cars rock noticeably.
V	Felt by nearly everyone; many awakened; some dishes, windows ,etc broken; a few instances of cracked plasters; unstable objects overturned; disturbance of trees, poles and other tall objects noticed sometimes and pendulum clock may stop.
VI	Felt by all; many animals get frightened and run

outdoors; some heavy furniture move a few instances of fallen plaster or damaged chimneys; damage slight.

VII	Everybody runs outdoors, damage negligible in buildings of good design and construction; slight to moderate in well built ordinary structures; considerable in poorly built or badly designed structures; some chimneys broken; noticed by persons driving motor-cars.
VIII	Damage slight in specially designed structures; considerable in ordinary but substantial buildings with partial collapse; very heavy in poorly built structures; panel walls thrown out of farmed structures, heavy furniture overturned, sand and mud ejected in small amounts; changes in well water; and disturbs persons driving motor-cars.
IX	Damage considerable in specially designed structures; well designed framed structures thrown out of plumb; very heavy in substantial buildings with partial collapse; buildings shifted off foundations; ground cracked conspicuously; and underground pipes broken.
X	Some well built wooden structures destroyed; most masonry and framed structures with foundations destroyed; ground badly cracked; rails bent; landslides considerable from river banks and steep

slopes; shifted sand and mud;and water splashed over banks.

XI Few ,if any, masonry structures remain standing; bridges destroyed; broad fissures in ground, underground pipelines completely out of service; earth slumps and landslips in soft ground;and rails bent greatly.

XII Total damage; waves seen on ground surface; lines of sight and level distorted; and objects thrown into the air.

2.5 MAGNITUDE OF AN EARTHQUAKE

It is the term expressing the rating of an earthquake on the basis of amplitude of seismic waves recorded as seismograms.

Dr. Charles F. Richter of the California Institute of Technology devised a scale of magnitude of the earthquakes. The magnitude is judged by the measurement of the maximum amplitude of the earthquake waves recorded on a seismogram by a standard seismograph placed at a distance of 100 Km from the epicenter.

$$M = \log_{10} A/Ao$$

Where,

M is the magnitude of the earthquake.

A is the maximum amplitude recorded by a seismograph at a distance of 100Km from epicenter

A_0 is an amplitude of one thousandth of a millimeter.

2.6. CONSEQUENCES OF AN EARTHQUAKE

An earthquake causes vibratory motion to the mass of earth through which the energy waves pass and this motion is transmitted to the engineering structures standing on the earth's surface. They get impulsive jolts in both the horizontal directions and also to some extent in the vertical direction. The vertical motion is predominant in the epicentral region, but it decreases in significance with distance from the epicenter.

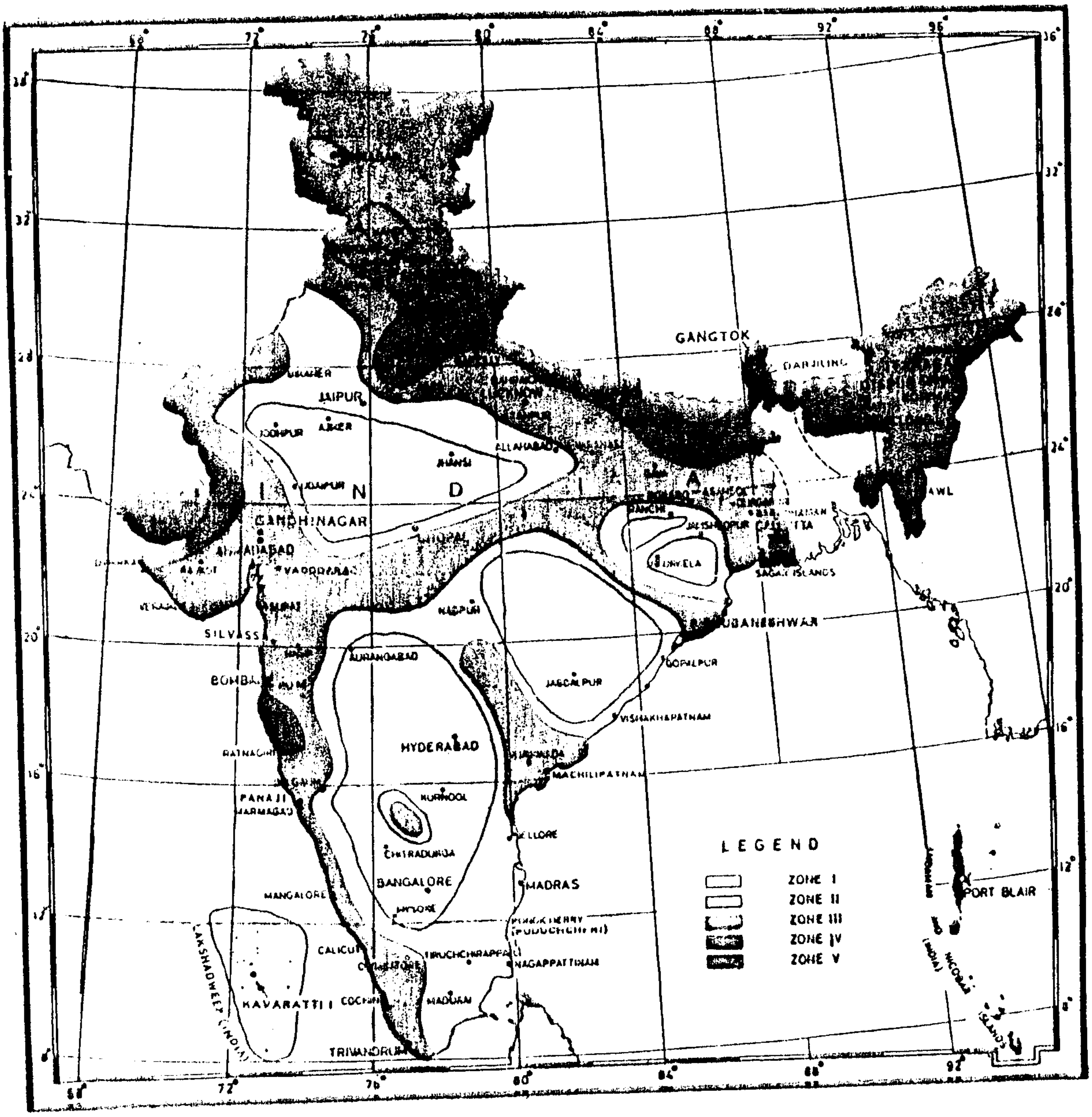
The vibratory jolts cause additional shears and moments in the structures and it may fail if they have not been adequately designed. The loss of life and property occurs directly from the failure of structures and may also take place due to indirect causes such as failure of water supply, fire caused by short circuiting electric wires or kitchen fires, flooding through failure of dams and embankments.

2.7. SEISMIC ZONING IN INDIA

The earthquake activity in different parts of the country is not necessarily of the same severity. So , depending upon the intensity expected in various area based on past records, India has been divided into five zones called Zone I, Zone II, Zone III, Zone IV and Zone V. In Zone I, the lowest intensity of earthquake is expected. The seismic zoning map of India is shown in fig 2.3

2.8. A BRIEF STUDY OF VIBRATIONS

Consider a spring-mass system shown in fig.2.4. If the mass is displaced and released, it will execute a to and fro motion. If the motion of a system repeats itself after a given interval of time, the



The territorial waters of India extend into the sea to a distance of twelve nautical miles measured from the appropriate base line.

Responsibility for the correctness of internal details shown on the maps rests with the publisher.

Based upon Survey of India map with the permission of the Surveyor General of India.

FIG. 2.3

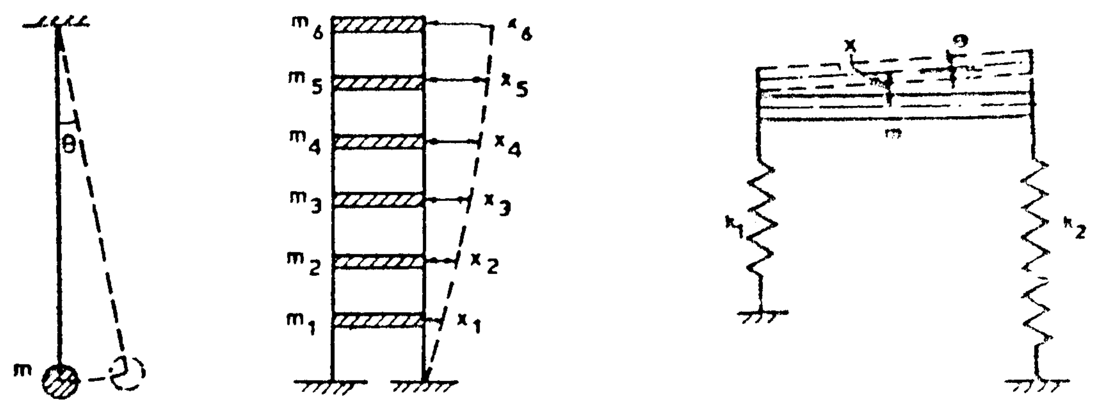
motion is termed as Vibration. The time displacement relationship for the spring mass system is shown in fig.2.5. The time required to complete one cycle of motion is called Time Period of the system and is expressed in sec per cycle. The number of cycles of motion per unit of time is called Frequency and is expressed in cycles per sec or Hertz. The maximum displacement in a cycle is called Amplitude of vibration.

2.8.1. DEGREES OF FREEDOM

The number of independent coordinates required to specify the position of a system at any instant of time is called Degrees of freedom. For a multi degree discrete mass system, the total degree of freedom of the system would be the sum of the degrees of freedom of the individual masses.

Fig.2.6. shows a system in which the position of a mass 'm' is completely defined by the value of "θ", the length "l" being known. Since, only one coordinate is required to define the position of the mass at any instant, it is a system with single degree of freedom. Consider a three storied frame shown in fig 2.6. assuming the masses to be concentrated at the floor levels.

If the structure is allowed to vibrate in one plane(horizontal), then the three displacements V_1, V_2 and V_3 for the three masses m_1 to m_3 will describe the deformed position of the structure. Therefore, the structure has three degrees of freedom.



(a) SINGLE DEGREE OF FREEDOM

(b) SIX DEGREES OF FREEDOM

(c) TWO DEGREES OF FREEDOM

FIG 2.6

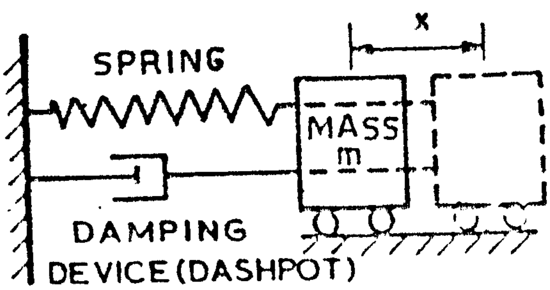


FIG 2,5

Fig. 2.3 Mass, Spring and Damping

2.8.2 TYPES OF VIBRATION

2.8.2.1 UNDAMPED FREE VIBRATION

In the absence of external exciting forces, when the mass is disturbed from its equilibrium position , it undergoes free vibration.

For dynamic equillibrium ,

$$\sum Fx = 0$$

that is

spring force + damping force + inertial force = External force

spring force = k x

damping force = $C\dot{x}$

inertial force = $m\ddot{x}$

where,

k = stiffness constant

x = displacement

C = damping coefficient

$\dot{x}$ = velocity (dy/dx)

m = mass

$\ddot{x}$ = acceleration (d^2y/dx^2)

P(t)= external force

$$m\ddot{x} + c\dot{x} + k\dot{x} = p(t)$$

when external force is zero ,
the above equation reduces to

$$m\ddot{x} + c\dot{x} + kx = 0 \text{ (free vibration equation)}$$

when damping is zero (say) ,

$$m\ddot{x} + k\dot{x} = 0$$

For a undamped single degree of freedom system, the above equation is second order , homogenius, differential equation and is an eigen value problem.

2.8.2.2 DAMPED FREE VIBRATION :

In any vibrating system, energy is dissipated due to friction and other resistances. This is termed as damping. Here, viscous damping is taken which is proportional to velocity. Therefore, the equation of motion becomes,

$$m\ddot{y} + c\dot{y} + ky = 0 \qquad \text{...(2.2)}$$

2.8.2.3 FORCED VIBRATION :

Let us consider a system excited by a force F(t).
If damping is present in the system, then the equation of motion is given by

$$m\ddot{y} + c\dot{y} + ky = F(t) \qquad \text{...(2.3)}$$

If damping is absent,

$$m\ddot{y} + ky = F(t) \qquad ...(2.4)$$

$m\ddot{y}$ - Inertial force,

$\ddot{y}$-acceleration (d^2y/dt^2)

$c\dot{y}$ - Viscous damping force

$\dot{y}$ - velocity (dy/dt)

ky - elastic force

y - displacement

For a multi degree of freedom system, the equation motion becomes,

$$[m][\ddot{y}] + [c][\dot{y}] + [k][y] = F(t) \qquad ...(2.5)$$

where,

[m] is the mass matrix.

[c] is the damping matrix

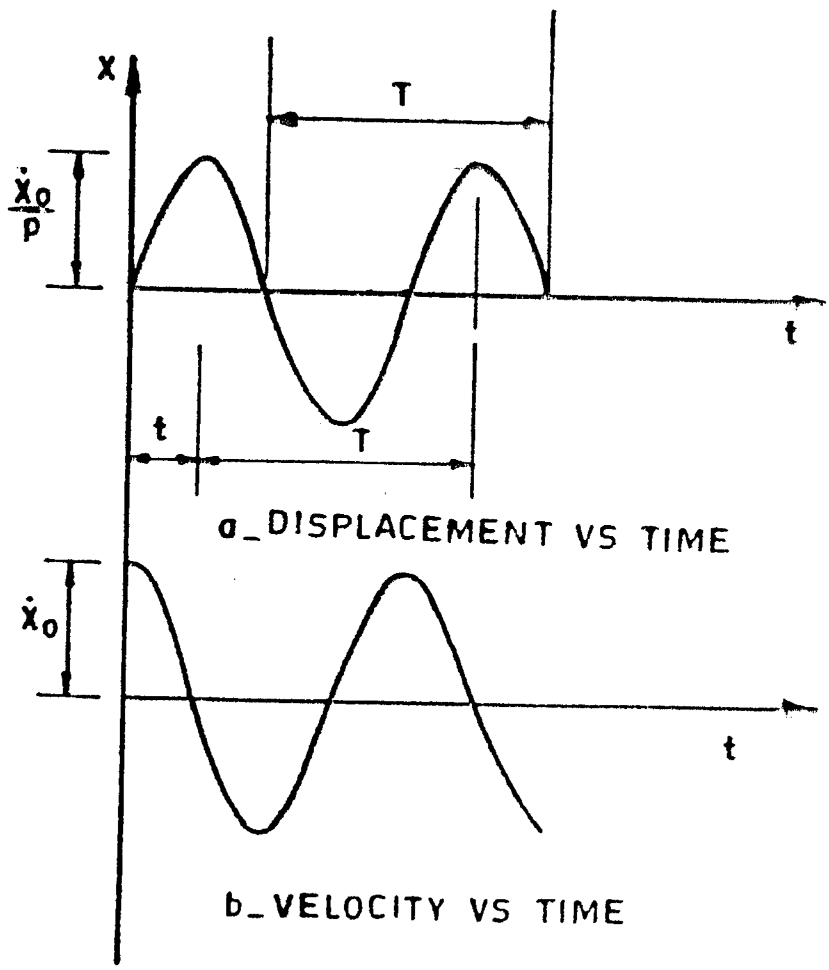
[k] is the stiffness matrix

$[\ddot{y}]$ is the acceleration vector

$[\dot{y}]$ is the velocity vector

[y] is the displacement vector

In the following chapters, Seismic coefficient method is used for the determination of seismic forces in the static analysis and Holzer's method and response spectrum method in the case of dynamic analysis are used.



x
T
$\frac{\dot{x}_0}{p}$
t
t
T
a_DISPLACEMENT VS TIME
$\dot{x}_0$
t
b_VELOCITY VS TIME

FIG 2.4

CHAPTER-3

PRINCIPLES OF ANALYSIS OF MULTISTOREYED BUILDINGS FOR EARTHQUAKE LOADS AND VERTICAL LOADS

3.1 INTRODUCTION

An earthquake causes the ground to move in all the three mutually perpendicular directions. The various parts of a structure move differently with respect to the foundation and due to this relative deformation, additional forces are exerted on the structure. For the design of a structure including its foundation, a theoretical model representing its behavior has to be derived and the ground motion expected at its site has to be assumed. The earthquake motion at a site depends on a number of factors like magnitude of the expected earthquake, distance of the site from its focus and nature of soil on which the structure rests.

For obtaining a theoretical model of the system, it has to be idealized into a spring-mass-dash pot system. Among these,estimation of masses is probably the least complicated. Usually physical lumping is adopted as for example, in the case of multistorey frames, the masses are assumed to be concentrated at floor levels.

For simplicity of analysis, multistorey buildings are generally assumed as a shear structures, that is, floor systems are infinitely rigid as compared to columns ie floors are assumed to act as rigid diaphrams and masses are assumed to be accumulated at floor levels.

The damping in a system is usually chosen arbitrarily because damping varies with strain in material and nature and detail of construction. The damping is usually assumed to be viscous for convenience in solution of vibration equations although it would be partly viscous and partly frictional in actual practice. Damping is usually specified in terms of modal damping. The percentage damping for various materials increases with strain levels and are in the following range.

Concrete - 5 to 10 %

Steel - 2 to 5 %

Masonry - 5 t0 10 %

Soil - 10 to 30 %

Timber - 2 to 5 %

3.2 MULTISTOREYED BUILDINGS

These structures are generally treated as equivalent discrete multi degree of freedom systems. At the floor level, the various masses of the floor system together with the supporting systems (columns and walls) located both above and below the floor levels are assumed to be concentrated. The horizontal shearforce acting at any level of the building is obtained by multiplying the seismic coefficient with the total effective weight of the structure above this level. It is usual in buildings to consider only the horizontal forces due to an earthquake and ignore vertical components of the shock except for certain portions where stability against overturning is a determining factor of design. With the increase in number of stories, the fundamental period of the structure increases. The seismic coefficient is proportional to the spectral acceleration in the first mode and this acceleration generally decreases with period. Therefore, the seismic coefficient for buildings decreases with increase in number of storeys.

3.3 TECHNIQUES OF ANALYSIS

In general, two broad approaches of earthquake analysis of multistorey structures in present day are,

i) Equivalent static approach

ii)Dynamic method of analysis

While a Dynamic approach to the problem of design of earthquake resistant structures is the most realistic one, the present day methods of taking equivalent static approach for seismic analysis is considered as being sufficient for moderately high buildings.

3.4 EQUIVALENT STATIC APPROACH

The equivalent static force method also called pseudo Dynamic method of analysis is adopted in most of the building codes for moderately high buildings due to its simplicity and due to the fact that many structures designed on the basis of code coefficients have withstood satisfactorily during the past earthquakes. The static horizontal forces applied based on the values of seismic coefficients to simulate the effect of earthquake. The distribution of shears along the height is adopted to be similar to that obtained by Dynamic analysis. (Design forces specified by most codes are smaller than those indicated by Dynamic elastic analysis. IS:1893-1984, i.e."**Criteria for**

Earthquake Resistant Design of Structures", suggests the use of "Seismic Coefficient Method" for buildings upto ten storeys.

3.4.1 SEISMIC COEFFICIENT METHOD

The seismic coefficient method given in IS:1893- 1984 is first developed by Prof.Sano in 1905. This method is an approximate method and it follows equivalent static load principle. In the IS code , the horizontal base shear V_b is given by,

$V_b = K.C.\alpha_0.I.\beta.W$ ..... (3.1)

Where,

V_b is the base shear

'K' is the performance factor depending upon the structural system and ductility of the construction,

'C' is a coefficient defining the flexibility of the structure depending on 'T' ,where 'T' is the fundamental period of the building in seconds,

'α_0' is the basic seismic coefficient ,

β is the soil- foundation factor and

'W' is the effective weight of the total structure.

3.4.1.1.PERFORMANCE FACTOR

The factor K has been retained in the base shear equation, as is given by IS code. The factor K is essentially a measure of the ductility of a building structure. The values of performance factor K is given in Table-3.1.

TABLE 3.1

VALUES OF PERFORMANCE FACTOR K

Sl No	Structural framing system	Values of performance factor
1	a) Moment resistant frame with appropriate ductility as given in IS 4326 - 1976 in RCC or steel.	1.0
	b)Frame as above with RC shear walls or steel bracing members designed for ductility	1.0
2	a) Frame as in (1)(a) with either steel bracing members or plain or nominally reinforced concrete in fill panels	1.3
	b) Frame as in (1)(a) in combination with masonry infills.	1.6
3	Reiforced concrete framed buildings [not covered by the above]	1.6

3.4.1.2 FUNDAMENTAL PERIOD (T) OF BUILDINGS AND FLEXIBILITY FACTOR 'C'

For calculating the earthquake forces acting on a building requires a determination of its vibration period.Any reasonably exact determination of the period of a multistorey building involves a complicated and time consuming calculation. Also this calculation cannot be made until the building is designed, but the building cannot be designed until the period is known. Accordingly, for practical use, a simple but inexact equation was devised. It is thought that errors in periods due to the use of this equation would not seriously affect the calculated value of the base shear.

There are several empirical formulae available for estimating period of buildings. Among them, a widely used one for moment resisting frames without bracings or shearwalls is

$$T=0.1*N \text{ seconds} \qquad (3.2)$$

Where,

N is the number of storeys.

Another formula which may be applied generally is,

$$T=0.09*H/\sqrt{D} \qquad (3.3)$$

Where,

H is the height of the building in meters

D is the dimension of the building in meters in a direction parallel to the applied seismic force.

In equation 3.1 ,the flexibility coefficient 'C' is established as a function of the period of the buildings: the shorter the period or stiffer the structure,the larger the coefficient and greater the forces produced in the building by an earthquake. This is in an accordance with Dynamic theory. The variation of C with T is shown in fig.3.1.

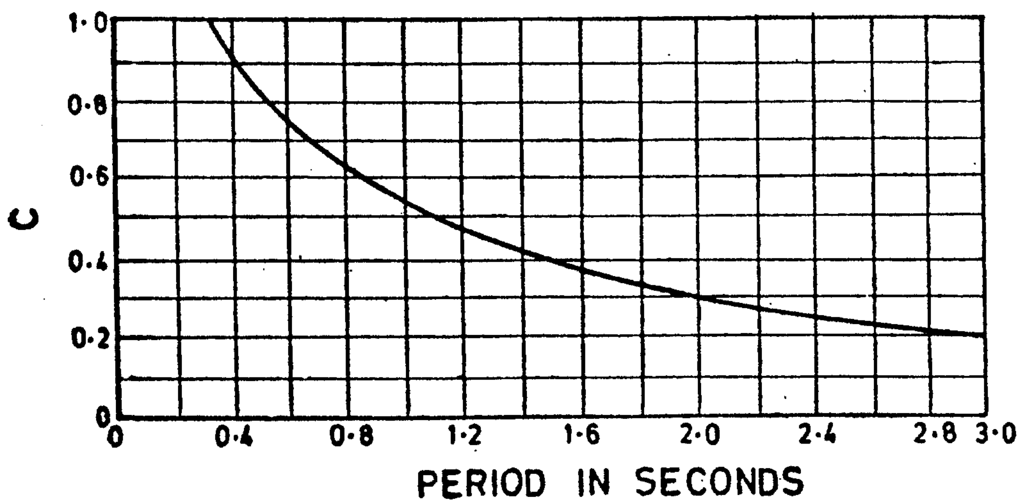
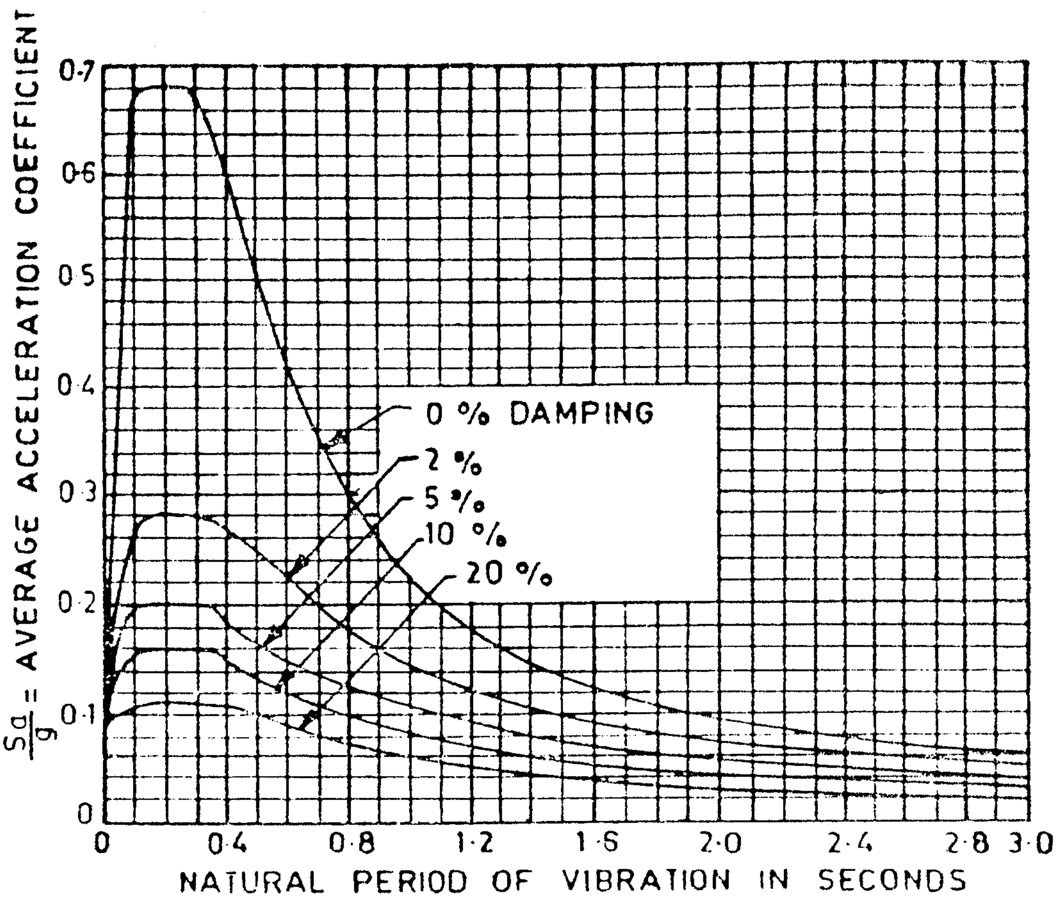


FIG. 3.1 *C Versus* PERIOD



$\frac{Sa}{g}$ = AVERAGE ACCELERATION COEFFICIENT
0·7
0·6
0·5
0·4
0·3
0·2
0·1
0
0 % DAMPING
2 %
5 %
10 %
20 %
0
0·4
0·8
1·2
1·6
2·0
2·4
2·8
3·0
NATURAL PERIOD OF VIBRATION IN SECONDS

3.4.1.3 SEISMIC COEFFICIENTS

For the purpose of determining the seismic forces, India is classified into five zones as shown in fig.3.2. Each zone is allotted a basic horizontal seismic coefficient 'α_0' described in Table-3.2.

Seismic coefficient = ground aceleration / acceleration due to gravity

TABLE-3.2

VALUE OF 'α_0'

Zone	Basic horizontal seismic co_efficient co_efficient 'αo'
I	0.01
II	0.02
III	0.04
IV	0.05
V	0.08

To arrive at design seismic coefficients, the basic values given in table 3.2. are multiplied by suitable factors to take care of foundations, importance of the structure and certain other conditions depending on flexibility of structure.

3.4.1.4 IMPORTANCE FACTOR 'I':

The term 'I' establishes higher seismic design factors for facilities deemed essential to public welfare and which should remain functional for use after a major earthquake. These essential facilities would include hospitals, fire stations and communication and power stations. The values of importance factor 'I' as given by IS code is given in Table-3.3

TABLE-3.3

VALUES OF IMPORTANCE FACTOR 'I'

Type of structures	Importance factor 'I'
1)Dams	3.0
2) Containers of inflammable or poisonous gases or liquids	2.0
3)Important service and community structures: Hospitals, water towers and tanks; schools,important bridges; Important power houses; Monumental structures; Emergency buildings like Telephone exchange and fire brigade; large assembly structures like cinema, assembly halls and subway stations.	1.5
All others	1.0

3.4.1.5 SOIL FOUNDATION FACTOR (ß):

The term ß is a soil-foundation factor. It involves not only the soil conditions at the site but also the surface mechanism of the earthquake, the distance from source to site, surface and subsurface topography and the travel path of the earth vibrations from source to site. The values of ß is given in table-3.4.

TABLE 3.4

VALUES OF β FOR DIFFERENT SOIL FOUNDATION SYSTEMS

Type of soil mainly constituting the foundation	values of β for					
	Piles passing through any soil but resting on type I	Piles not covered under tabular column 2	Raft foundations	RCC footing with tie beams	RCC footings without tie beams	Well foundations
1	2	3	4	5	6	7
TYPE.I Rock or Hard soil	1.0	-	1.0	1.0	1.0	1.0
TYPE II Medium soils	1.0	1.0	1.0	1.0	1.2	1.2
TYPE III Soft soils	1.0	1.2	1.0	1.2	1.5	1.5

3.4.2. DISTRIBUTION OF FORCES ALONG THE HEIGHT

One of the earliest assumptions is to have a uniform seismic coefficient along the height. In other words, the lateral force at any floor level is equal to a constant multiplied by the weight at that level. However, Dynamic analysis of buildings indicates that seismic coefficient varies along the height with a maximum at the top reducing to zero at the base. One type of distribution of accelerations which is in extensive use is an inverted triangle. This is suitable for buildings whose mass and stiffness per floor is same for all storeys. A study of damage to buildings during earthquakes indicates that generally the top portions suffer more damage than the lower ones.This is confirmed by Dynamic analysis of buildings.

For design purposes, it is necessary to distribute the base shear into its equivalent static forces to be concentrated at each storey level.

It is assumed by the IS code that the building vibrates dominantly or deflects at its first mode or fundamental mode in which the deflection is varying from zero at the base to maximum at the roof in parabolic fashion. On this assumption, the distribution given by IS code is,

$$Q_i = \frac{V_b * W_i * h_i^2}{\sum_{i=1}^{i=N} W_i * h_i^2} \qquad ...(3.4)$$

Where, Q_i is the lateral force at floor 'i'

V_b is the base shear as given by equation 3.1

W_i is the effective weight considered to be acting at floor level 'i'

h_i is the height of the floor i above the base of the building
N is the number of storeys.

3.5 RESPONSE SPECTRUM METHOD

For important buildings and for buildings having more than 10 storeys,it is advisable to find forces by dynamic analysis. The analysis would involve the determination of periods and modes of vibration for the first few modes. Usually it is sufficient to consider the first three or four modes. The total response can be determined either by evaluating the timewise response of the structure for an accelerogram or by using spectral curve. The later procedure which is recommended in the IS code is described below. The load Q_i^r acting at any floor level i in the r^{th} mode of vibration is given by the following equation.

$$Q_i^{(r)} = \frac{\phi_i^{(r)} \sum_{j=1}^{j=n} W_j * \phi_j^{*(r)}}{\sum_{j=1}^{j=n} W_j((\phi_j^{(r)}))^2} * \frac{S_a^{(r)}}{g} * \beta * I * F_o \qquad \text{--- 3.5}$$

where,

W_i is the weight of the floor i

$\phi_i^{(r)}$ the mode shape coefficient at floor i obtained from free vibration analysis,

r the mode of vibration,

S_a the spectral acceleration value corresponding to the appropriate natural period of vibration and damping of the structure,

ß a co_efficient depending on soil foundation system, I the importance factor,

Fo the seismic zone factor,

g the acceleration due to gravity and

N the no of storeys.

The shear force at any storey in a particular mode can be obtained by summing up the loads on the masses above that storey.

$$V_i^{(r)} = \sum_{i=1}^{i=n} Q_i^{(r)} \qquad \text{----------- } 3.6$$

The subscript i=1 refers to the topmost storey.

The total response would be some combination of response of individual modes. An upper bound of the total response would be obtained by summing up the absolute maximum response of individual modes.This method is termed as ***"MODULUS SUPERPOSITION METHOD"***. This would be generally too conservative .A method of combination which is popular for design purposes is to obtain the square root of the sum of the squares of the modal responses. This method is termed as ***"QUADRATIC SUPERPOSITION"*** .The earthquake response is largest in the first mode and decreases rapidly with increase in modes. It is usual to consider the first three modes for design purposes.

The IS recommendation for the shear, V_i ,acting at floor level i, is a combination of modulus and quadratic superposition.

$$V_i = (1-\gamma)\sum \left| V_i^{(r)} \right| + \gamma * \sqrt{\left(\sum (V_i^{(r)})^2\right)} \quad \text{---- } 3.7$$

This is, sum of (1-γ) times modulus value and gamma times the quadratic value where gamma is the function of the height in metres as given below.

HEIGHT (m)	GAMMA γ
Upto 20	0.4
40	0.6
60	0.8
90 & above	1.0

3.5.1 HOLZER'S METHOD

Holzer's method is a numerical technique to determine the natural period of vibration in different modes of a building of 'n' storeys. Refer figure 3.5

By summing the inertia forces from the free end,

Shear force at a level below mass $m_{i-1} = \sum_{j=1}^{j=i-1} m_j \ddot{x}_j$

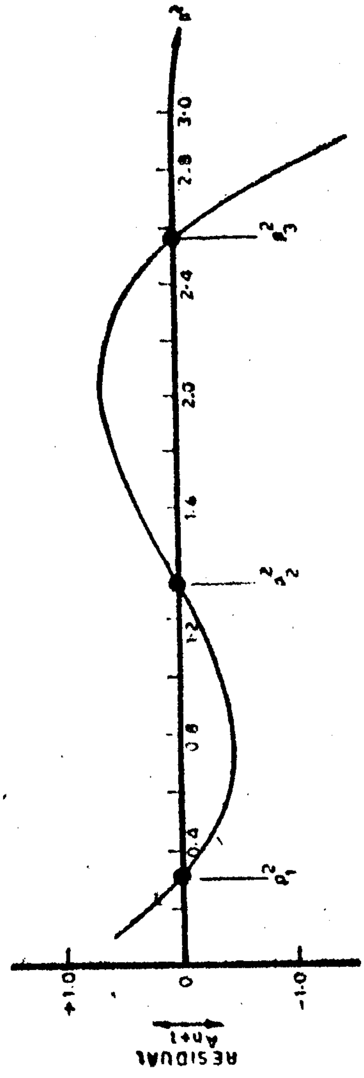
Spring force at that level corresponding to the difference of displacement of adjoining masses. $= K_{i-1} (x_i - x_{i-1})$

Equating the two , we get

$$\sum_{j=1}^{j=i-1} m_j \ddot{x}_j = K_{i-1} (x_i - x_{i-1}) \quad \text{-------3.8}$$

Putting x= A sin (pt) in eqn 3.8, we get

j=i-1



Residual as a Function of Frequency in Holzer Method

FIG 3.5

$$\sum_{j=1} m_j \left(- A_j P^2 \sin pt\right) = K_{i-1} \left(A_i \sin pt - A_{i-1} \sin pt\right) \quad \text{-------3.8}$$

$$A_i = A_{i-1} - \frac{P^2}{k_{i-1}} \sum_{j=1}^{j=i-1} m_j A_j \quad \text{---3.9}$$

The above equation gives a relationship between any two successive amplitudes. Starting with an arbitrary valueof A1 (say 500), amplitudes of all other masses can be found. Finally, An+1 should work out to zero. Due to the fixity of the building at the base for certain values of P^2. A plot of An+1 Vs P^2 would have the shape as shown in the figure 3.6.

The intersection of the curve with P^2 axis would give various values of P^2. Incidently, this procedure also gives mode shape with a substitution in eqn 3.9 with a correct value of P^2.

3.6. EFFECTIVE WEIGHT:

The effective weight at any floor level would be equal to dead weight of the floor system and half the weight of supporting column system both above and below the floor plus effective live load. In the case of live load, only a portion of the live load may be taken as effective for three reasons. The first is that only a part of the maximum live load will probably be acting at the time of earthquake, the second is that non rigid mounting of the live load absorbs part of the earthquake energy and the third is that the specified liveload includes, as part of it, impact effect of the loads which need not be considered since earthquake loads act on the mass only.

The IS code makes the following recommendations. For various loading cases as specified in IS:875, the horizontal earthquake forces may be calculated for the full dead load and percentage of live loads as given in table-3.5.

PERCENTAGE OF LIVE LOADS

Intensity of live load (N/m^2)	percentage of live load
2000,2500,3000,Stairs and balconies	25%
4000,5000,7500,10000, Garage	50%

TABLE-3.5

The percentage of design live load given in table 3.5 will also be used for calculating stresses due to normal vertical loads for combining those due to earthquake forces. Under earthquake conditions, the whole frame may be assumed loaded with live load except the roof. Alternatively, the live load may be assessed according to actual expectations as far as possible and in that case for calculating horizontal earth quake forces only that part of live load should be considered which possesses mass. Earthquake coefficient should not be applied to impact effects.

3.7 DIRECT STIFFNESS PROCEDURE FOR PLANE FRAME ANALYSIS

Direct stiffness procedure is used for analysing the frame for the loads coming on the structure. Plane frame analysis is carried out using computer programs. Three degrees of freedom has been considered for each member in the frame(as shown below) having two displacements and one rotation at each end.

Equation of equilibrium of the frame system is given by,

$[K]\{Y\} = \{P\}$

Where,

[K] - system stiffness matrix or global stiffness matrix.

{Y} - Displacement vector in global coordinate directions.

{P} - Load vector corresponding to degrees of freedom (Y)

After assembling the global stiffness matrix , boundary conditions are applied and then the displacements are found by ,

$\{Y\} = [K]^{-1}\{P\}$

Then, the member forces are found by the relation,

$[k]\{d\} = \{Q\}$

where,

[k] - element stiffness matrix

{d} - element displacement vector

{Q} - element or member forces.

Computer programs for the above direct stiffness matrix method has been given in appendix B1.

CHAPTER-4

ANALYSIS AND DESIGN OF MULTISTOREYED BUILDINGS FOR EARTHQUAKE LOADS AND VERTICAL LOADS

4.1 ANALYSIS

A step by step procedure for the analysis of multistoreyed buildings for the combined effect of earthquake loads and vertical loads are explained below.

STATIC ANALYSIS

Step 1:

The effective weight at each floor level is calculated as per article 3.4.1.6. This weight is assumed to be lumped at each floor level. The total effective weight of the building 'W' is calculated as the sum of effective weight of individual floors.

Step : 2

The fundamental time period of the building is determined using equations (3.2) and (3.3). Then using fig 3.1 , the value of flexibility coefficient 'C' is obtained. Now, the performance factor 'K', basic seismic coefficient 'α_0', importance factor 'I' and soil-foundation factor 'ß' are obtained from table 3.1 to table 3.4 respectively. After determining the above constants and effective weight 'W'of the building ,the total horizontal base shear is determined by using equation 3.1

Step : 3

Having obtained the base shear , it has to be distributed vertically through the height of the building. That is, using equation (3.4) the lateral force at each floor is calculated.

Step 4:

The frame is now analysed for the known loads is the earthquake force ,liveload and the dead load.

DYNAMIC ANALYSIS
(FREE VIBRATION)

Step 1:

The load calculations are done as described in the static analysis.

Step 2:

The time period is calculated using Holzer's method.

Step 3:

The residual amplitude is calculated for the first three modes of time periods.

Step 4:

Response spectrum method is adopted to find the lateral force at each node corresponding to each mode.

Step 5:

Total response is found out which is the combination of lateral loads of three modes.

Step 6:

Design is carried out as described earlier.

Computer programs for the analysis and design are developed by the authors and are given in the appendix.

4.2 ASEISMIC DESIGN OF STRUCTURES

A structure is designed to carry various types of loads-permenant,semi-permenant, movable and moving in occasional.

The effective length of the columns is usually assumed as center to center distance between the floors.

For simplicity of analysis, multistoreyed buildings are assumed as shear structures.(ie the floor system are infinitly rigid as compared to columns.

Following the above steps, lateral forces were obtained at different floor levels of a building and are given in figure 4.5-4.12.

Various members of a three storeyed frame were analysed for two different loading cases as below.

I case - Dead load +Live load

II case - Dead load + Live Load + Earth quake load

Limit state method is adopted for the design of columns and beams.

In the first case, a factor of safety for loads is taken as 1.5 . as per IS 456-1876.

In the second case, a factor of safety of 1.2 is adopted and also a proportion of live load is considered as per IS 1893-1984.

A comparision has been made between the amount of steel required for various members of a three storeyed building for the above said different loading conditions.

4.2. ILLUSTRATIVE EXAMPLE

Buildings having from three storey levels to ten storey levels are taken for the present work.The detailed step by step procedure for the analysis of three storey building is given here.

Plan of the three storey building is given in figure 4.3.

The following data are assumed :

Floor slab thickness = 150 mm

Dimension of column = 300 mm x 600 mm(for the first three floors)

= 300 mm x 500 mm(for the next three floors ie 4,5&6th floor)

= 300 mm x 400 mm(for the next three floors ie 7, 8 & 9th floor)

= 300 mm x 300 mm(for top floor ie 10th floor)

Dimension of beam = 250 mm x 500 mm

Intensity of live load =3000 N/m^2

Dead load per unit area =5000 N/m^2

Concrete grade = M15

Building is assumed to be located in seismic zone -IV in India.

Step : 1

The effective weight at each floor level is to be calculated. It includes the dead load and effective live load.Referring to table 3.5 , the percentage of design live load for calculating earthquake forces is 25% at all floors. No live load need be considered at roof level.

Effective weight at all floors = 5000 + (25/100 *3000)

= 5750 N/m^2

Effective weight at roof = 5000 N/m^2

(No live load is considered at roof).

From fig. 4.3.,

The plan area of the building is 20 m * 14 m = 280 m^2

Effective weight at roof level = 280 * 5000

= 1400000 N

=1400 KN

Effective weight at all floors = 280 * 5750

(except the roof) = 1610000 N

= 1610 KN

Total effective weight W of the building =1400+(1610* 2)

= 4620 KN

Step : 2

Fundamental time period (T) of the building :

i) From equation (3.2) ,

T = 0.1 * N

Where,

N= number of storeys

T = 0.1 * 3 = 0.3 seconds

ii) Also from equation (3.3) ,

T = 0.09 * H/√D seconds where,

H = total height of the building in meters = 3 * 3.5 =10.5m

D = dimension of the building in meters measured parallel to the applied seismic force

= 14 m

Therefore ,

$$T = 0.09 * 10.5 / \sqrt{14} = .2526 \text{ seconds}$$

From (i) and (ii) ,

Taking T = 0.2763 seconds.

Having known the time period of the building (T) , the flexibility coefficient can be obtained.

From fig 3.1 , for time period T =0.2763 seconds ,

flexibility coefficient ' C '= 1.0 From table 3.1 ,

Performance factor 'K' = 1.0 From table 3.2 ,

for seismic zone -IV ,

the value of basic horizontal seismic coefficient'α_0 ' is

$$\alpha_0 = 0.05 \text{ From table 3.3,}$$

The value of importance factor 'I' = 1.0 From table 3.4 ,

The value of soil-foundation factor for the assumed combined R.C.C footing , ß = 1.0

Now, using equation 3.1 ,

Total base shear $V_b = K.C.\alpha_0.I.ß.W$

Substituting the above values ,

$V_b = 1 * 1 * 0.05 * 1 * 1* 4620$

$V_b = 231$ KN

Having obtained the total base shear , it is distributed through the height of the building i.e. at each floor level, using equation 3.4 ,

$$Q_i = \frac{V_b \; W_i \; h_i^2}{\quad}$$

$$\sum_{I=1}^{i=n} W_i \; h_i^2$$

where,

Q_i is the lateral force at i_{th} floor ,

W_i is the effective weight at floor 'i',
h_i is the height of floor 'i' above the base of the building.The lateral force calculations are shown in table 4.1

LATERAL FORCE CALCULATIONS FOR A THREE STOREY BUILDING

MASS No.	W_i (KN) (I)	h_i (m) (II)	$W_i h_i^2$ KN-m² (III)	Q_i (KN) (IV)=(III)/A
1	1400	10.5	154350	140.95
2	1610	7.0	78890	72.04
3	1610	3.5	19722.5	18.01

$\Sigma W_i h_i^2$=252962.5 - (A)

STEP 3:

Quasi Dynamic analysis

The equivalent K_c of a single column is

$$K_c = 12EI/L^3$$

Taking L as 3.5 m,(C/C distance between the floors)

E as 5700 √ fck N/mm² and I as moment of inertia of only the concrete section, for a column of size 300mm width and 500mm depth.

I $= bd^3/12$

$= 300 \times 600^3 / 12$

$= 5.4 \times 10^9 \text{ mm}^4$

E $= 5700*(\sqrt{15}) = 22076 \text{ N/mm}^2$

K_c $= 12 \times 22076 \times 5.4 \times 10^9 / 3500^3$

$= 333650$ N/m

Similarly, for a column of 300mm x 600mm,

K_c = 19308 N/m

For the column of 300mm x 400mm,

K_c = 9885 N/m

For the top floor (10th floor),

K_c =4170 N/m

There are 18 columns of size 300mm x 600mm in all the three floors. Since the columns act in parallel, the total "K" per floor, for the bottom three floors would be,

K =18 x 33365 = 600680 N/m

For the next three floors,

K =18 x 19308 =347544 N/m

For the next three floors,

K =18 x 9885 =177930 N/m

For the top floor (ie 10th floor),

K =18 x 4170 =75060 Kg/m

The mass and stiffness values are tabulated below.

MASS NO	MASS (m) KN sec^2/m	STIFFNESS (k) KN/m
1	140	600680
2	161	600680
3	161	600680

APPLICATION OF HOLZER'S METHOD:

(as described in article 3.5.1)

Initially let us assume P^2=501.

A	m	mA	ΣmA	$(P^2/k^{i-1})\ \Sigma mA$	A_i
2	3	4 2x3	5	6	7 2–6
1.00	140.00	140.00	140.00	0.12	0.88
0.88	161.00	142.20	282.20	0.24	0.65
0.65	161.00	104.31	386.51	0.32	0.33

FOR P**2 = 791.

A	m	mA	ΣmA	$(P^2/k^{i-1})\Sigma mA$	A_i
2	3	4 2x3	5	6	7 2–6
1.00	140.00	140.00	140.00	0.18	0.82
0.82	161.00	131.32	271.32	0.36	0.46

0.46	161.00	73.80	345.11	0.45	0.00

FOR P**2 = 6001.

A	m	mA	ΣmA	$(\bar{P}^2/k_{i-1})\Sigma mA$	A_i
	2	3	4	5	6	7
			2x3			2–6
1.00	140.00	140.00	140.00	1.40	–0.40
–0.40	161.00	–64.18	75.82	0.76	–1.16
–1.16	161.00	–186.13	–110.31	–1.10	–0.05

FOR P**2 = 6101.

A	m	mA	ΣmA	$(P^2/k^{i-1})\Sigma mA$	A_i
	2	3	4	5	6	7
			2x3			2–6
1.00	140.00	140.00	140.00	1.42	–0.42
–0.42	161.00	–67.93	72.07	0.73	–1.15
–1.15	161.00	–185.78	–113.71	–1.15	0.00

FOR P**2 = 12321.

A	m	mA	ΣmA	$(P^2/k^{i-1})\Sigma mA$	A_i
	2	3	4	$\bar{5}$	6	7

2x3 2–6

1.00	140.00	140.00	140.00	2.87	−1.87
−1.87	161.00	−301.33	−161.33	−3.31	1.44
1.44	161.00	231.46	70.12	1.44	0.00

The last value of A is the residue and is plotted vs P^2in the graph. The graph gives the mode shape. The cutting points are obtained from the graph which represent the frequency of vibration of the building at different modes of vibration. From the frequencies, we can get the natural time periods of vibration using the formula,

$$T = \frac{2\pi}{\sum P^2}$$

where, T is the natural period of vibration corresponding to that particular mode. P is the frequency of vibration.
From the graph the cutting points are,

$P1^2 = 794.8$, $P2^2 = 6099.07$, $P3^2 = 12320.42$

$$T_1 = (2 \times \pi) \div (\sum P1^2)$$
$$= (2 \times 3.141592654) \div (794.8)$$
$$= 0.2228681 \text{ sec}$$

similarly, T_2=0.0804541 sec. and T_3=0.0566066 sec.

It is to note that the time period obtained by using emperical formulae (in seismic coefficient method) is 0.26 seconds and by exact analysis is 0.2228. A comparative graph of time period has been drawn between the two methods (fig 4.5)

APPLICATION OF MODE SUPERPOSITION:

	Ist mode	II mode	III mode
PERIOD	0.223 sec	0.081 sec	0.057sec
Mode participation factor "C" $=\sum m\phi / \sum m\phi^2$	1.223	–0.297	0.0677
MODE S_1	1	1	1
SHAPE S_2	0.81475	–0.4222	–1.87
S_3	0.4559	–0.154	1.44

EARTH QUAKE RESPONSE:

Corresponding to the above period s and assuming a damping of 5% in each vibration the spectral acceleration obtained from figure is as follows:

$sa^{(1)}/g=0.2$, $sa^{(2)}/g=0.2$ and $sa^{(3)}/g=0.2$.

for this problem, ß = 1.0

I = 1.0

Fo= 0.25 corresponding to the zone IV.

The lateral load and shear force in each mode of vibration have been obtained u following equation:

$$Q_1^{(1)} = W_1 \times \Phi_1^{(1)} \times C^{(1)} \times \frac{s_a^{(1)}}{g} \times \beta \times I \times F_o$$

where,

$$C^{(1)} = \frac{\sum m\,(\phi^{(1)})}{\sum m\,(\phi^{(1)})^2}$$

$W_1 = 1378$ KN

$s_1^{(1)} = 1$

$C^{(1)} = 1.223$

$\beta = 1$

$I = 1$

$s_a^{(1)}/g = 0.2$

$F_o = 0.25$

$\Rightarrow Q_1^{(1)} = 84.40$ KN

MASS NO	Ist MODE		IInd MODE		IIIrd MODE	
	Q (KN)	V (KN)	Q (KN)	V (KN)	Q (KN)	V (KN)
1	84.40	84.40	–20.37	–20.37	4.64	4.64
2	79.08	163.48	9.87	–10.49	–9.99	–5.34
3	44.25	207.7	27.04	16.54	7.67	2.32

TOTAL RESPONSE:

The total response is obtained as a combination of the response in three modes. The shear force at each floor can be obtained from the equation:

$V_i = (1-\gamma)\Sigma|V_i^{(r)}| + \gamma * \sqrt{(\Sigma(V_i^{(r)})^2)}$

$\Rightarrow V_i = (1-\gamma)a_i + \gamma \times \sqrt{b_i}$

where, $a_i = \Sigma|V_i^{(r)}|$ $b_i = (\Sigma(V_i^{(r)})^2)$

(This is as per the Is code recommendation for the shear, V_i, acting at floor level i, is the combination of modulus and quadratic super position.) The factor GAMMA depends on the height of the building. ie,

HEIGHT (m)	GAMMA
Upto 20	0.4
40	0.6
60	0.8
90 & above	1.0

As the height of the building is 10.5 m,

GAMMA = 0.4 Various values of a_i and b_i are given in the following table.

MASS NO	a_i	$\sqrt{b_i}$	V
1	109.42	86.95	100.43
2	179.33	163.91	173.16
3	226.61	208.42	219.33

$V_i = (1 - \gamma) a_i + \gamma \times \sqrt{b_i}$

$\Rightarrow V_i = (1 - 0.4) \times 109.42 + 0.4 \times (86.95)$

$= 100.4361$ KN.

COMPARISON OF THE TWO METHODS:

Forces obtained from empirical method (IS code procedure) and Response spectrum method (for a THREE storey building given in the fig) are as in the following table.

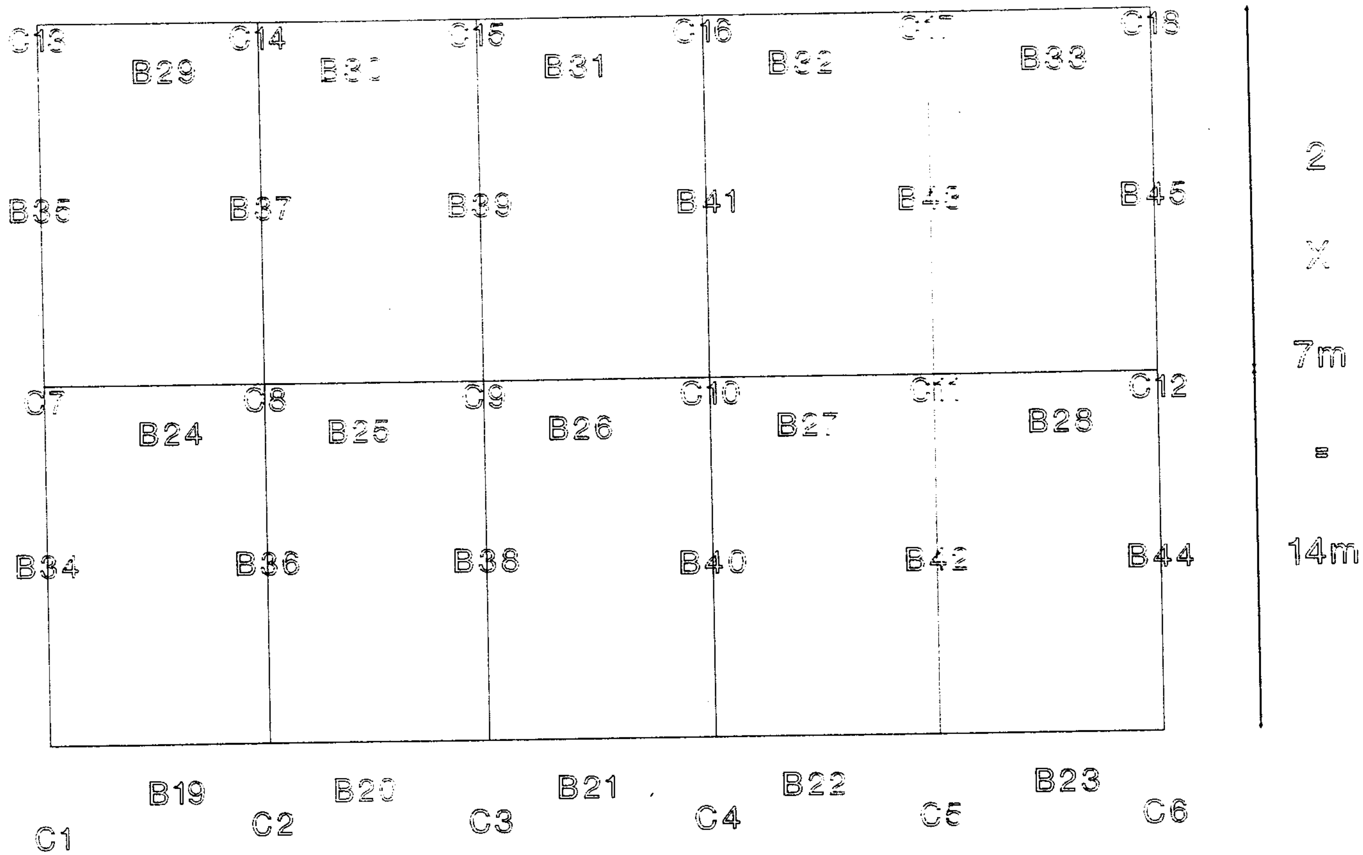
SEISMIC COEFFICIENT METHOD	RESPONSE SPECTRUM METHOD
Lateral force at each floor i	Lateral force at each floor i
14.21	100.44
79.56	173.17

226.61 219.33

VARIATION OF REINFORCEMENT FOR A THREE STOREYED STRUCTURE

MEMBER	I STOREY		II STOREY		III STOREY		
	I CASE	II CASE	I CASE	II CASE	I CASE	II CASE	
1	1440	8789	1440	2602	1440	1627	COLUMN
2	1440	8458	1440	2277	1440	1301	COLUMN
3	1440	8458	1440	2277	1440	1301	COLUMN
4	1440	8458	1440	2277	1440	1301	COLUMN
5	1440	8458	1440	2277	1440	1301	COLUMN
6	1440	8783	1440	2602	1440	1627	COLUMN
7	1440	9759	1440	6181	1440	5205	COLUMN
8	1440	9759	1440	6181	1440	5205	COLUMN
9	1440	9759	1440	6181	1440	5205	COLUMN
10	1440	9759	1440	6181	1440	5205	COLUMN
11	1440	9759	1440	6181	1440	5205	COLUMN
12	1440	9759	1440	6181	1440	5205	COLUMN
13	1440	9493	1440	2602	1440	3253	COLUMN
14	1440	9493	1440	4554	1440	3904	COLUMN
15	1440	9493	1440	4554	1440	3904	COLUMN

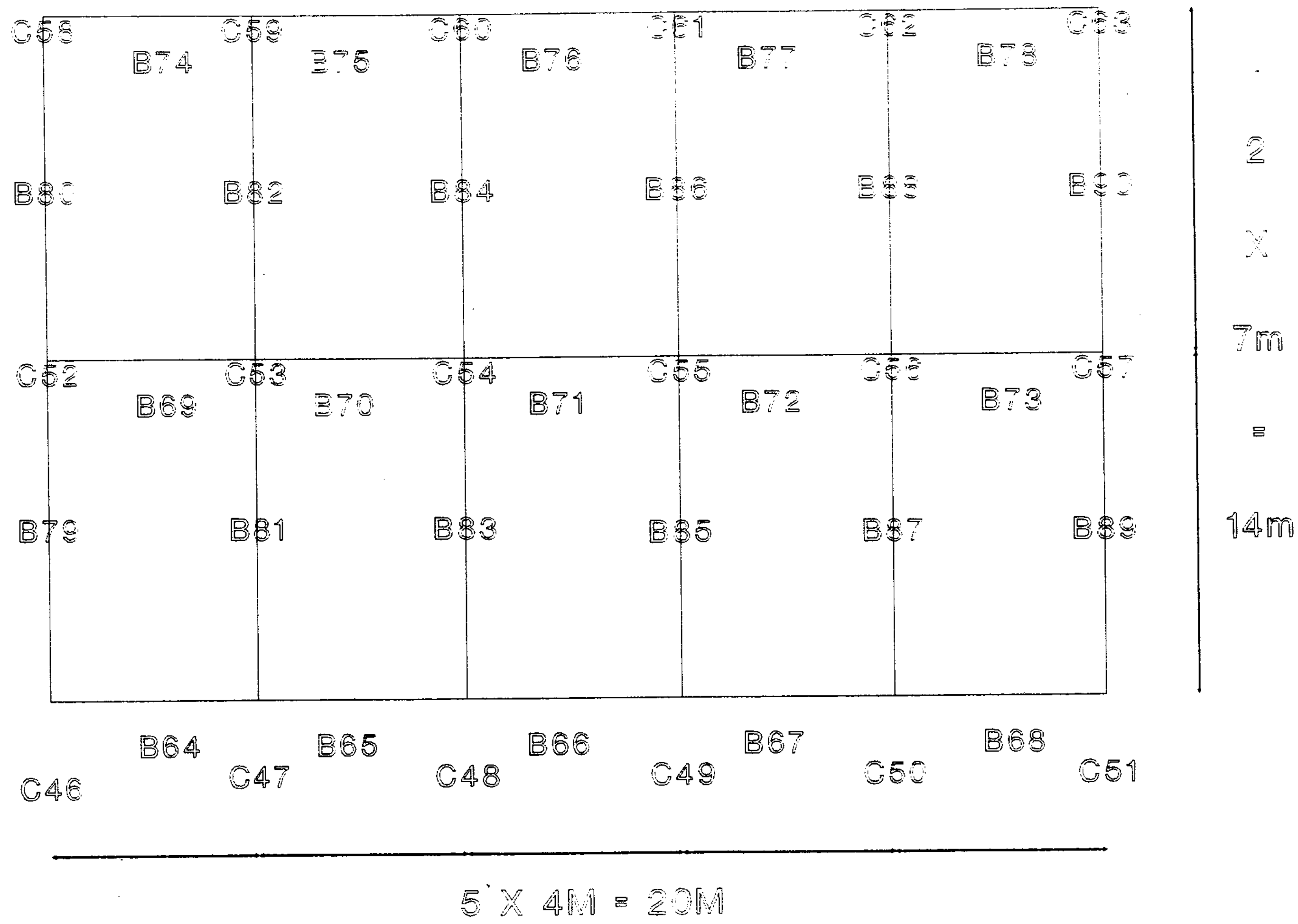
16	1440	9493	1440	4554	1440	3904	COLUMN
17	1440	9493	1440	4554	1440	3904	COLUMN
18	1440	9493	1440	2602	1440	3253	COLUMN
19	256	256	256	256	256	256	Beam
20	256	256	256	256	256	256	Beam
21	256	256	256	256	256	256	Beam
22	256	256	256	256	256	256	Beam
23	256	256	256	256	256	256	Beam
24	256	256	256	256	256	256	Beam
25	256	256	256	256	256	256	Beam
26	256	256	256	256	256	256	Beam
27	256	256	256	256	256	256	Beam
28	256	256	256	256	256	256	Beam
29	256	256	256	256	256	256	Beam
30	256	256	256	256	256	256	Beam
31	256	256	256	256	256	256	Beam
32	256	256	256	256	256	256	Beam
33	256	256	256	256	256	256	Beam
34	256	1162	392	1113	421	743	Beam
35	256	1175	392	1129	421	748	Beam
36	903	1375	878	1322	927	951	Beam
37	903	1366	878	1329	927	918	Beam
38	903	1375	878	1322	927	951	Beam
39	903	1366	878	1329	927	918	Beam
40	903	1375	878	1322	927	951	Beam
41	903	1366	878	1329	927	918	Beam
42	903	1375	878	1322	927	951	Beam



5 X 4M = 20M

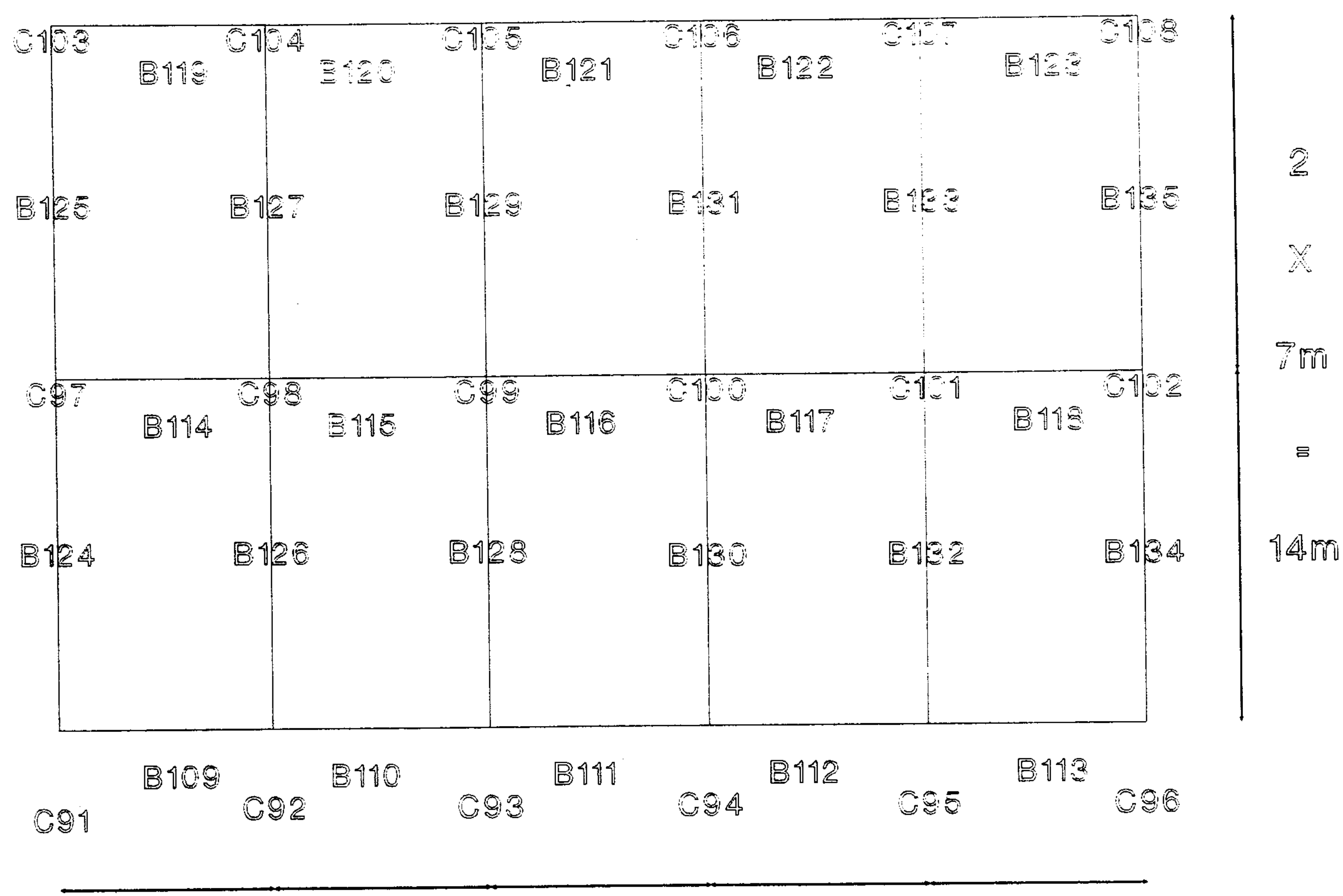
PLAN OF THE FIRST FLOOR

* C1-C18:-COLUMN, B19-B45:-BEAM



PLAN OF THE SECOND FLOOR

* C46-C63:-COLUMN, B64-B90:-BEAM



PLAN OF THE THIRD FLOOR

* C91-C108:-COLUMN, B109-B135:-BEAM

TIME PERIOD Vs NO OF STOREYS

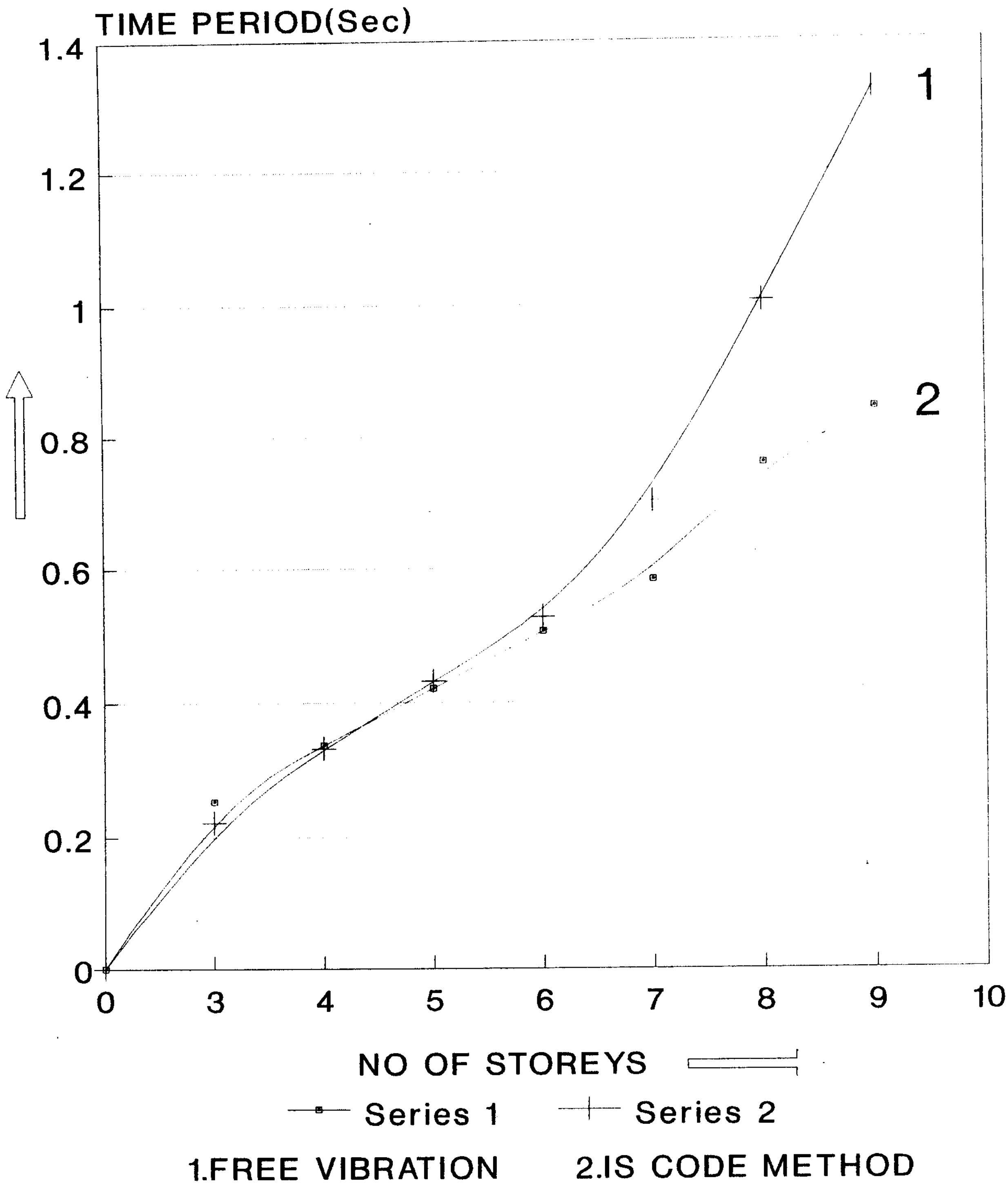
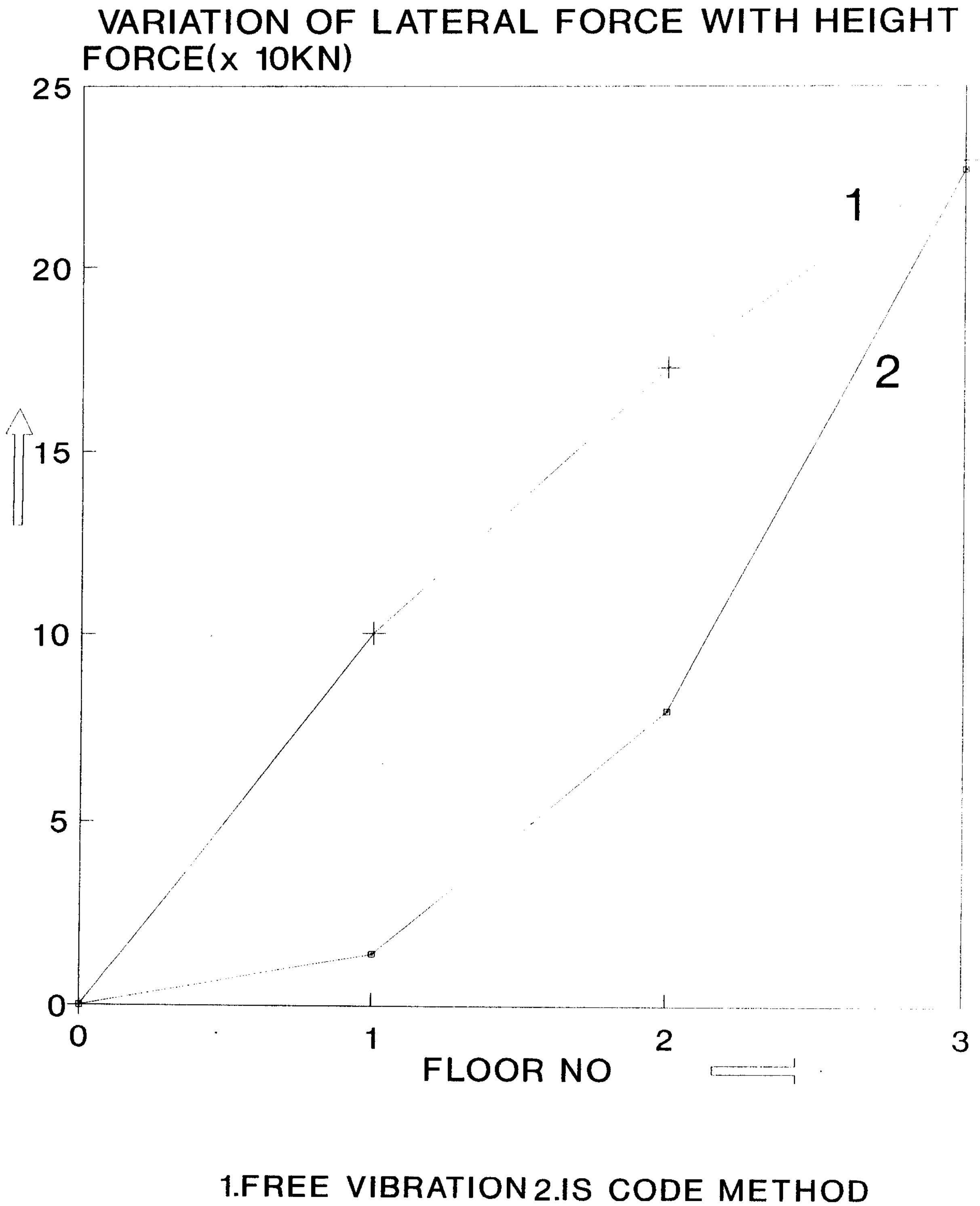
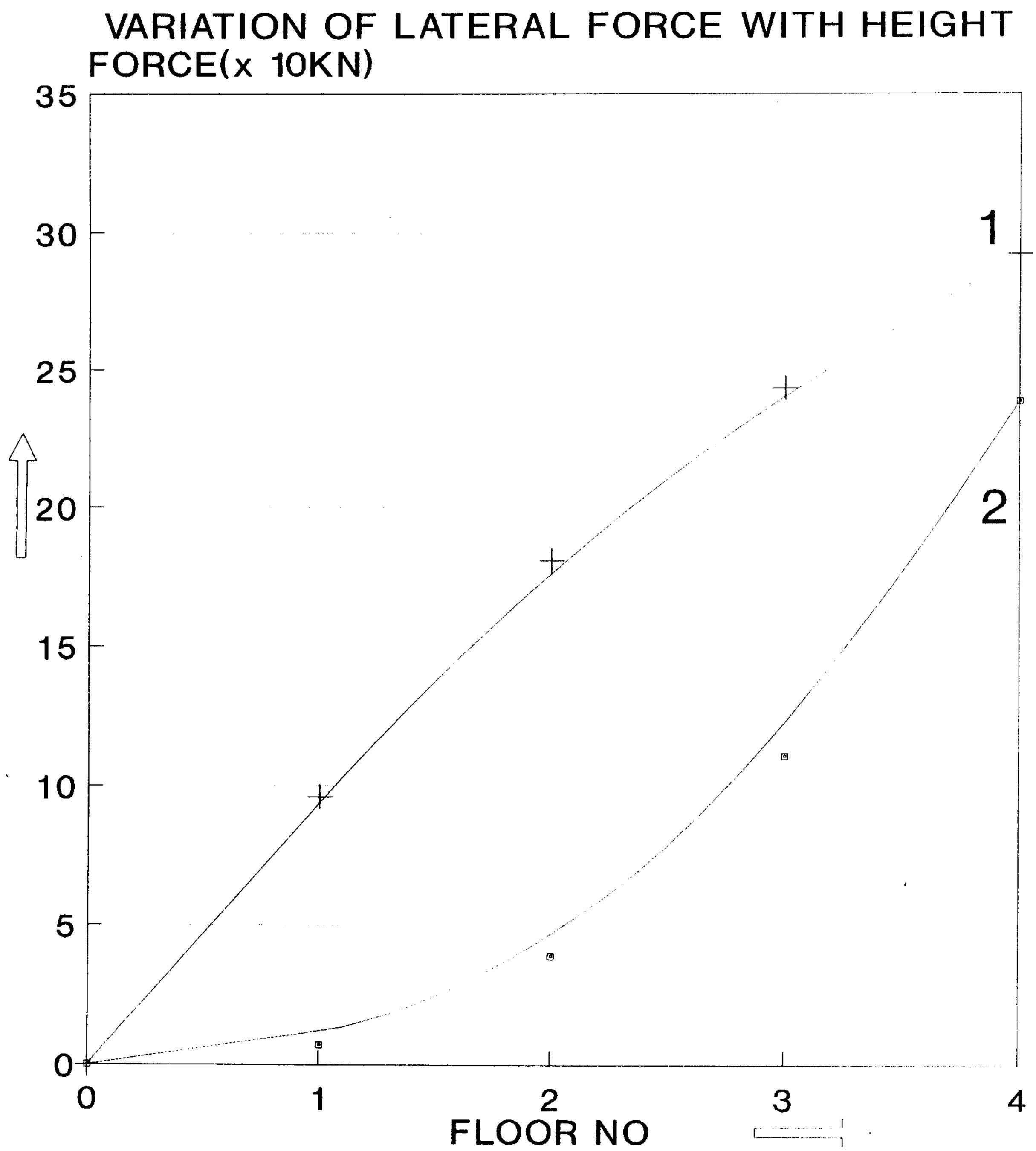


FIG 4.4

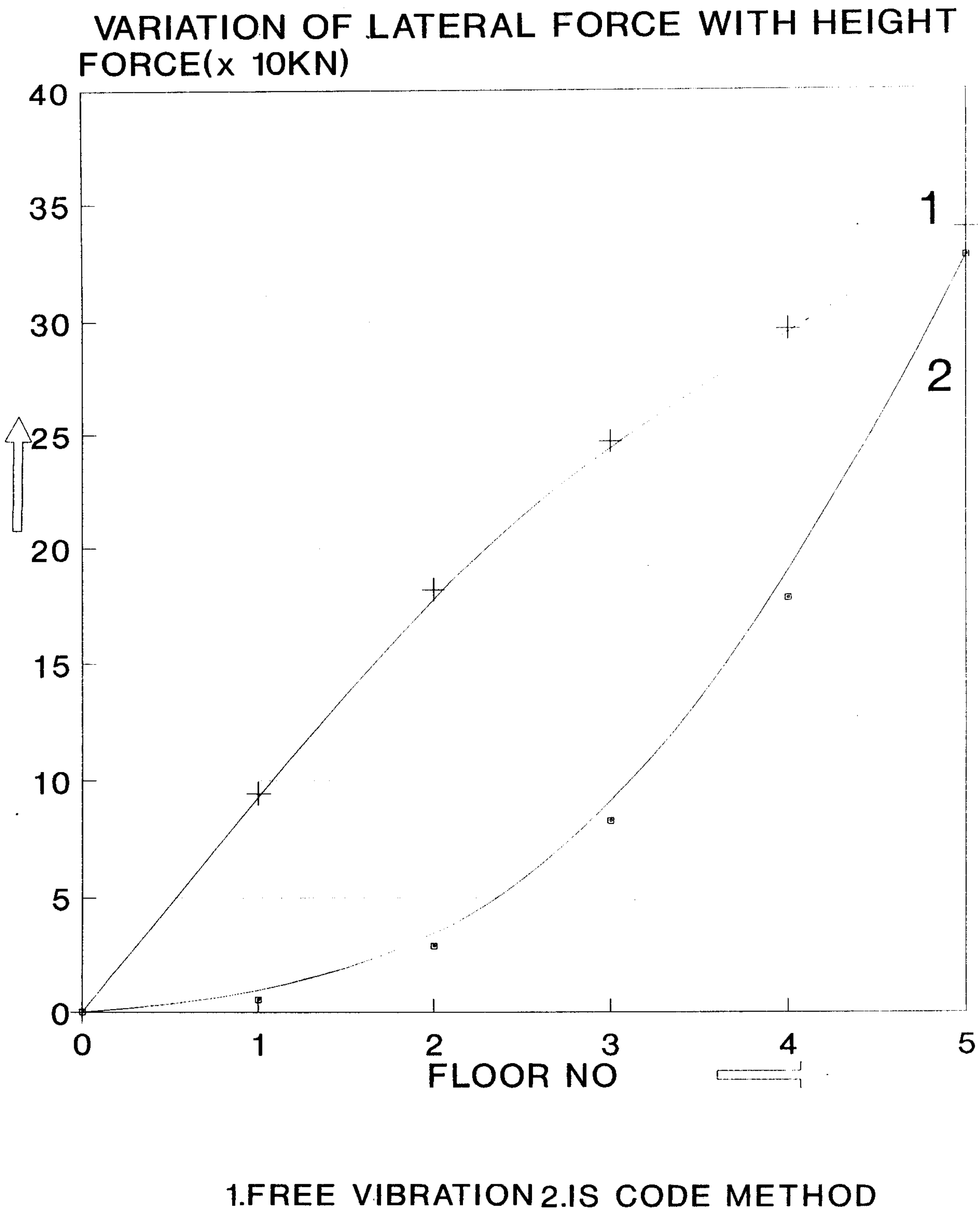


* FOR 3 STOREY BUILDING FIG 4.5



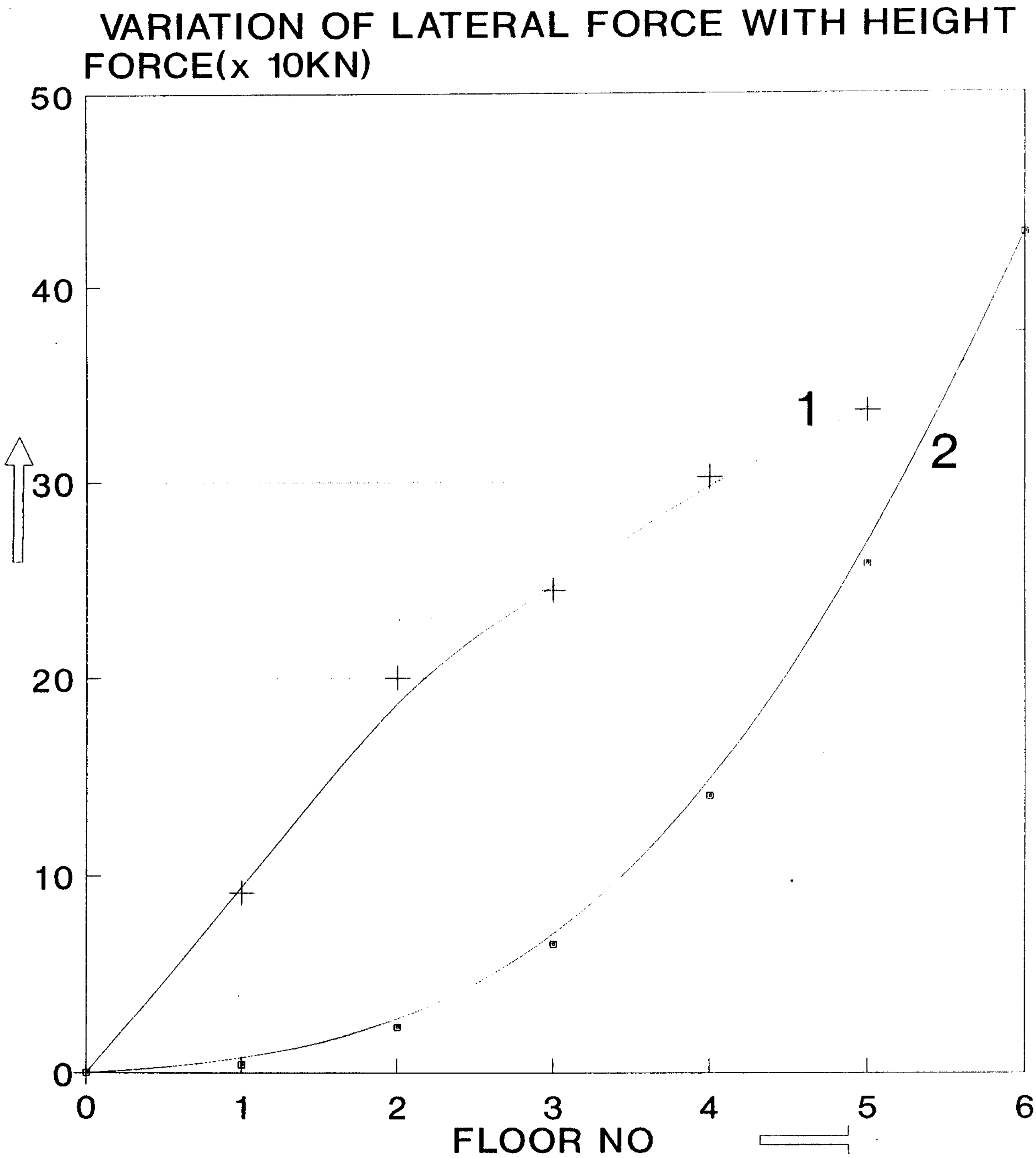
1.FREE VIBRATION 2.IS CODE METHOD

* FOR 4 STOREY BUILDING FIG 4.6



* FOR 5 STOREY BUILDING

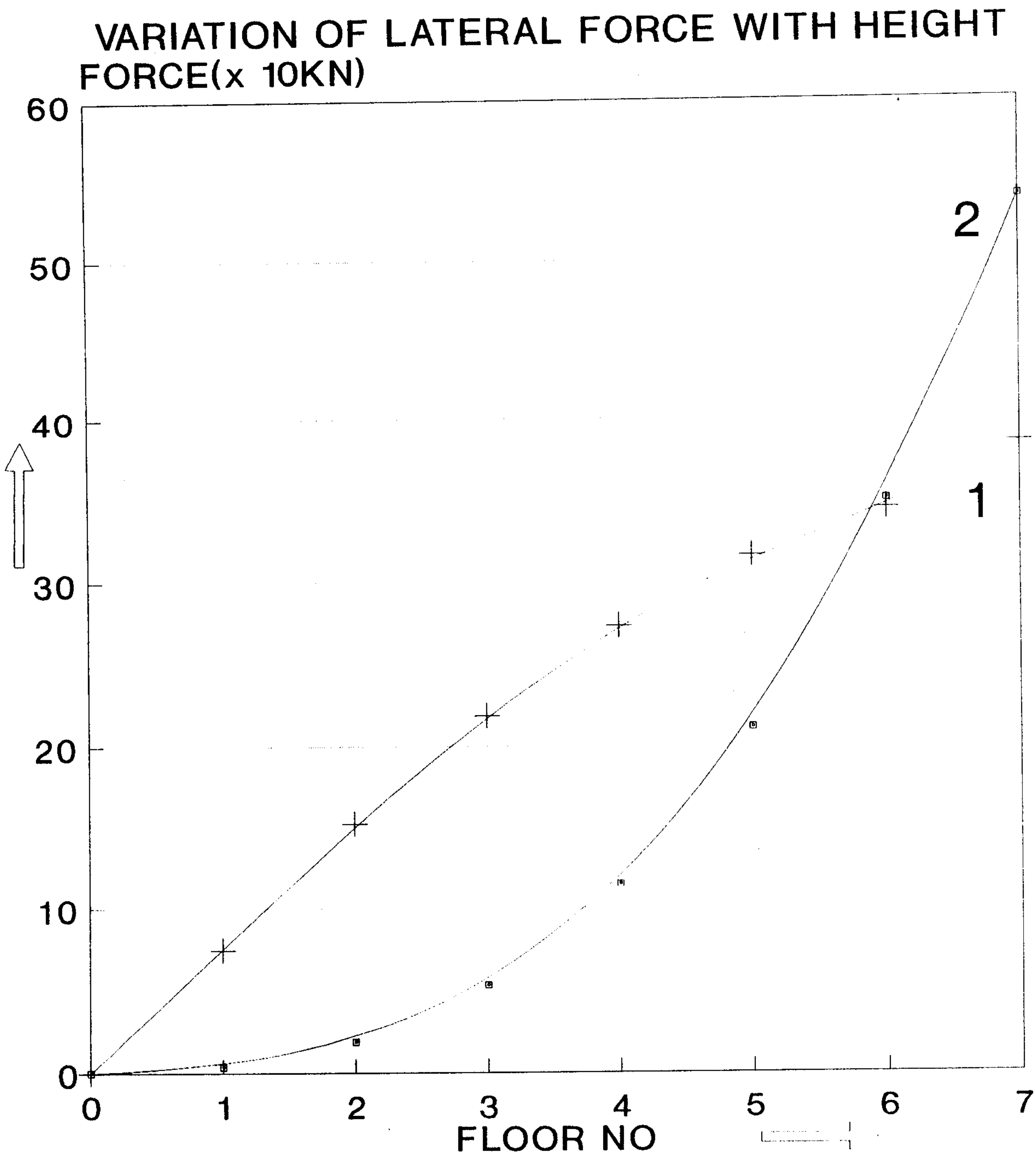
FIG 4.7



1.FREE VIBRATION 2.IS CODE METHOD

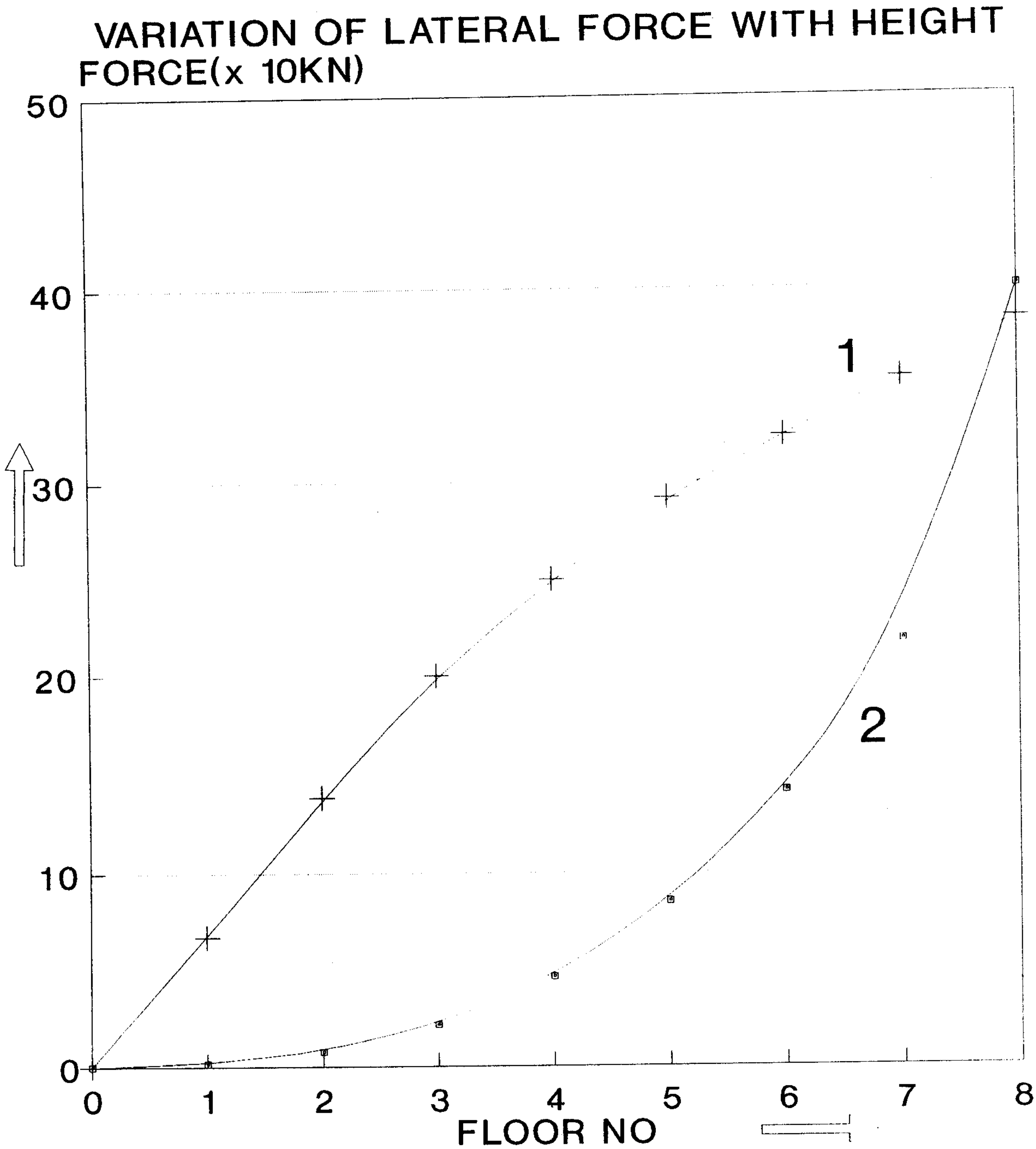
* FOR 6 STOREY BUILDING

FIG 4.8



1.FREE VIBRATION 2.IS CODE METHOD

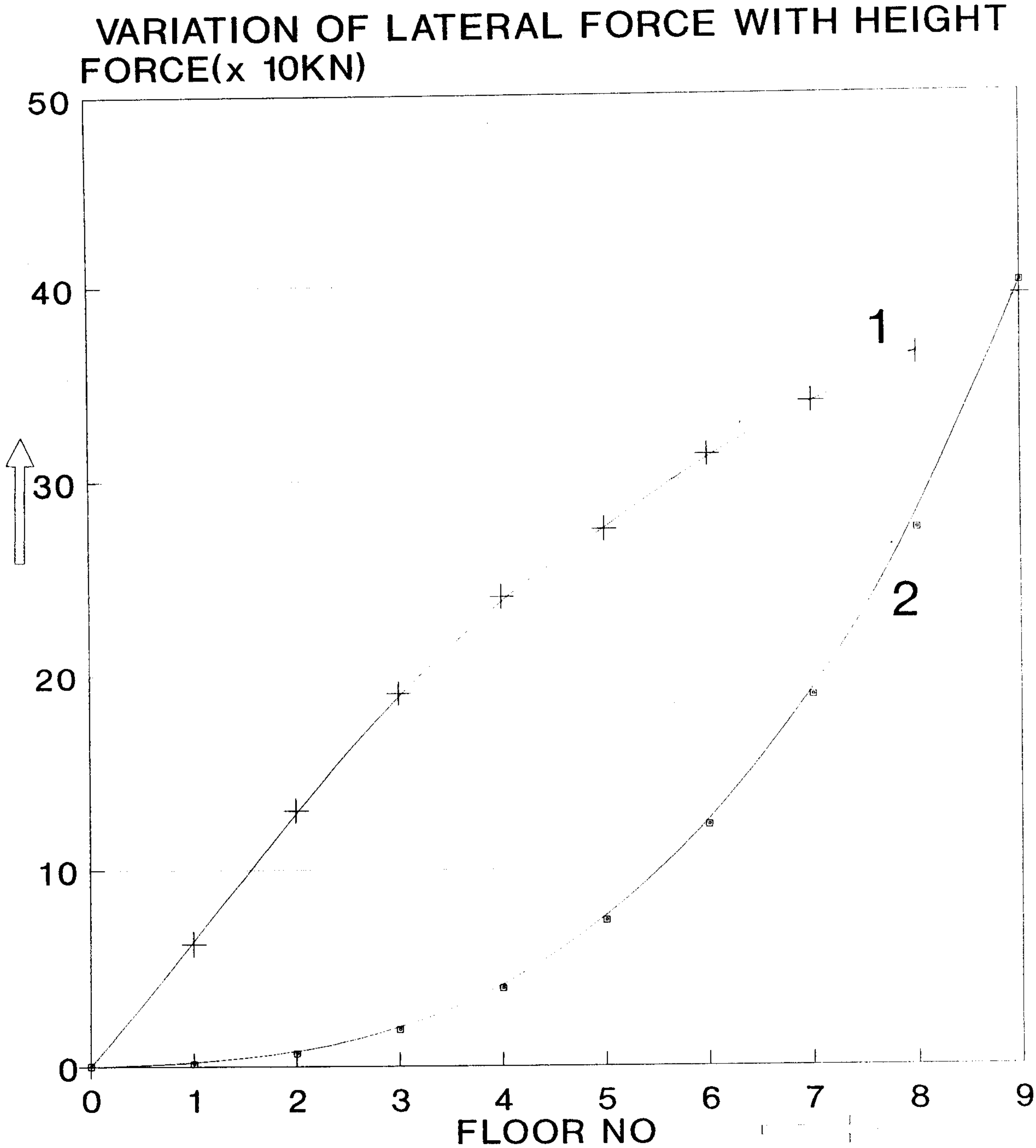
* FOR 7 STOREY BUILDING FIG 49



1.FREE VIBRATION 2.IS CODE METHOD

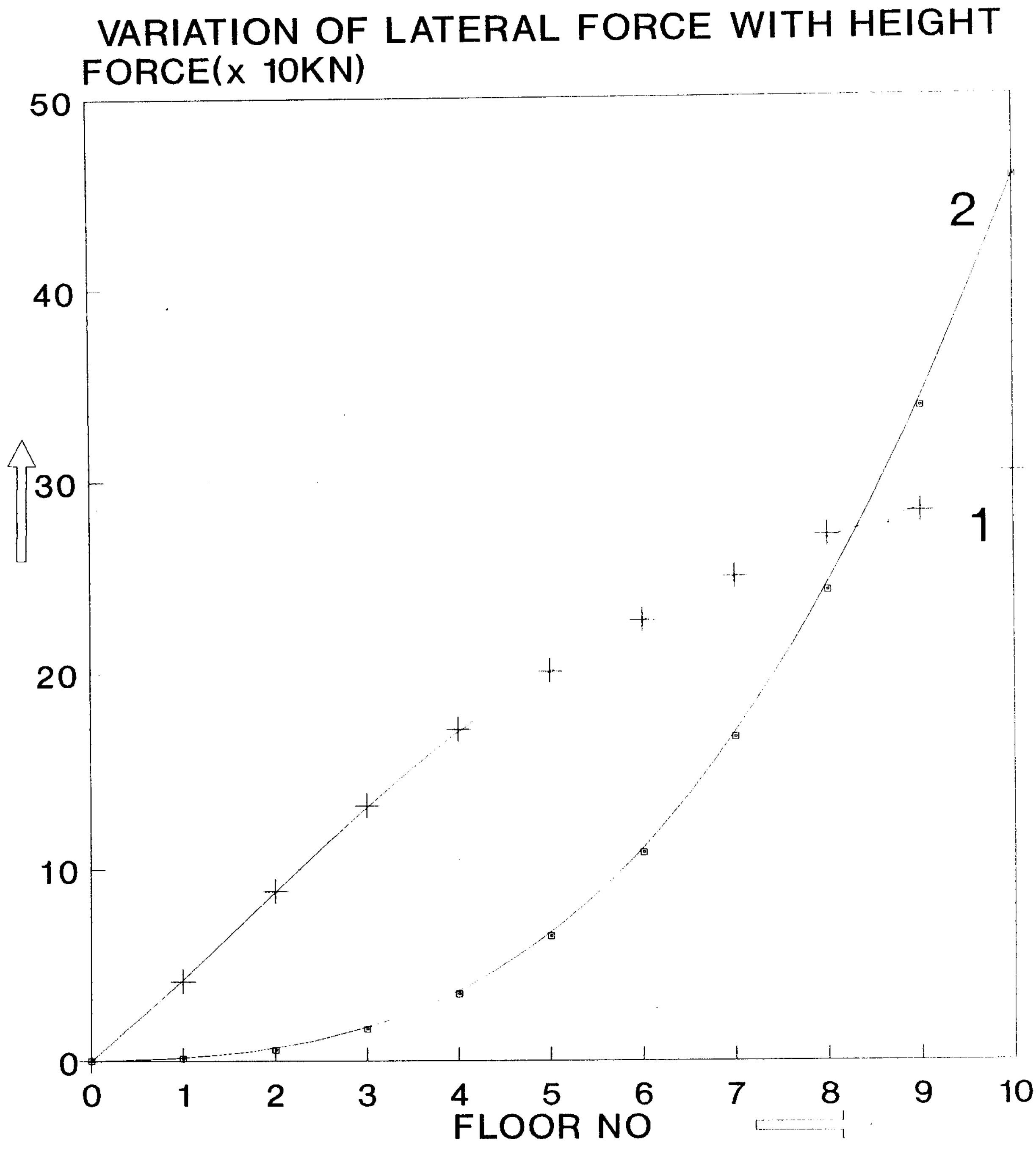
* FOR 8 STOREY BUILDING

FIG 4.10



1.FRE VIBRATION 2.IS CODE METHOD

* FOR 9 STOREY BUILDING FIG 4.11



1.FREE VIBRATION 2.IS CODE METHOD

* FOR 10 STOREY BUILDING FIG 4.12

Summary and Conclusion

II) LATERAL FORCES

By the carefull study of the figures 4.5 - 4.12 , we can conclude that the seismic coefficient method suggests upper bound values and the variation is parabolic.In case of the exact analysis we get results which are fairly reasonable and can be used for design purposes

III) REINFORCEMENT

VARIATION IN COLUMNS :

The variation in the amount of steel to resist the earthquake forces in addition to the normal loading conditions,is greater in the ground floor and decreases with the height..This may be due to the flexiblity of the structure.The flexiblity is zero at the base and the maximum at the top most floor.Hence the variation is less in top most floor.

The coloumns are assumed to provide the flexiblity and hence the variation is high in the case of columns.

VARIATION IN BEAMS:

The floor is assumed to act as a rigid diaphragm and hence the variation is not as dominant as in coloumns

The variation in the beams normal to the erthquake forces remain unaffected due to the rigid diaphragm action of the floors.

The variation in the beams along the earthquake forces are very low when compared to the columns.

```
C       PROGRAM NUMERICAL ANALYSIS
          REAL AM(100),AK(100),A(100),ARES(20002)
          REAL AFM(12,12),TPRD(12),SABYG(12),C(12),IMP,AI(12),V(12,12)
          REAL Q(12,12),WHT(12),MASS,STIFF,BI(12),SROOTBI(12),VN(12)
          REAL HEIGHT(12),WHSQR(12),QE(12),VE(12)
          REAL PSQR(20002),PSQRFIN(20),LAT(20),MM(20)
          INTEGER CO
          OPEN(3,FILE='LAT.IN')
          OPEN(4,FILE='LAT.OUT')
          READ(3,*)NS
          READ(3,*)(AM(I),I=1,NS)
          READ(3,*)(AK(I),I=1,NS)
          M=1
          CO=1
          DO 10 B=1,200000,10
                  A(1)=1.0
                  SMA=0.0
                  DO 20 I=1,NS
                          EMA = AM(I) * A(I)
                          SMA = SMA + EMA
                          ONEBYK=1.0/ak(i)
                          FA = B * ONEBYK * SMA
                          A(I+1)=A(I) - FA
20                CONTINUE
                  ARES(M) = A(NS+1)
                  PSQR(M)=B
                  M=M+1
10        CONTINUE
1010      FORMAT(80('-'))
C         INTERPOLATION TO GET THE FINAL VALUE OF PSQR
          J=1
          DO 40 I=1,19998
          IF(((ARES(I).LT.0).AND.(ARES(I+1).GT.0)).OR.
     * ((ARES(I).GT.0).AND.(ARES(I+1).LT.0)))THEN
          PSQRFIN(J)= PSQR(I+1)+((PSQR(I+1)-PSQR(I))/(ARES(I+1)-ARES(I))
     * *(0.0-ARES(I+1)))
          J=J+1
          ENDIF
40        CONTINUE
          WRITE(4,*)'THE RESULTS OF THE FREE VIBRATION ANALYSIS'
          WRITE(4,*)'THE CUTTING POINTS ARE:'
          WRITE(4,102)
102       FORMAT('THE VALUE OF P} FOR A=0 ')
          WRITE(4,109)
109       FORMAT(34('-'))
C         FINAL VALUE OF THE RESIDUE
          DO 51 J=1,3
             A(1)=1.0
             AFM(J,1)=1.0
             SMA=0.0
                  DO 52 I=1,NS
                          EMA = AM(I) * A(I)
                          SMA = SMA + EMA
                          ONEBYK=1.0/ak(i)
```

```
                    FA = PSQRFIN(J) * ONEBYK * SMA
                    A(I+1)=A(I) - FA
                    AFM(J,I+1)=A(I+1)
52            CONTINUE
51      CONTINUE
         WRITE(4,1010)
         WRITE(4,*)'                  P}                   TIME PERIOD'
         WRITE(4,*)'                  ----------------------------'
         DO 55 I=1,3
         TPRD(I)=(1/(SQRT(PSQRFIN(I))))*2.0*3.141592654
         WRITE(4,*)TPRD(I),PSQRFIN(I)
55      CONTINUE
         WRITE(4,1010)
         DO 56 I=1,3
         IF((TPRD(I).LE.0.3).AND.(TPRD(I).GT.0.0))SABYG(I)=0.2
         IF((TPRD(I).LE.0.8).AND.(TPRD(I).GT.0.3))
     *  SABYG(I)=0.13+(0.07/0.5)*(0.8-TPRD(I))
         IF((TPRD(I).LE.1.2).AND.(TPRD(I).GT.0.8))
     *  SABYG(I)=0.1+(0.03/0.4)*(1.2-TPRD(I))
         IF((TPRD(I).LE.1.6).AND.(TPRD(I).GT.1.2))
     *  SABYG(I)=0.06+(0.025/0.4)*(1.6-TPRD(I))
         IF((TPRD(I).LE.3.0).AND.(TPRD(I).GT.1.6))
     *  SABYG(I)=0.045+(0.015/1.4)*(3.0-TPRD(I))
         IF(TPRD(I).GT.3.0)SABYG(I)=0.045
56      CONTINUE
C       TO GET THE ZONE AND TO REDUCE SABYG
         READ(3,*)ZONE
         WRITE(4,*)'THE SITE IS SITUATED IN THE ZONE NO:',ZONE
         WRITE(4,1010)
         DO 57 I=1,3
         IF(ZONE.EQ.1)SABYG(I)=SABYG(I)*0.05
         IF(ZONE.EQ.2)SABYG(I)=SABYG(I)*0.1
         IF(ZONE.EQ.3)SABYG(I)=SABYG(I)*0.2
         IF(ZONE.EQ.4)SABYG(I)=SABYG(I)*0.25
         IF(ZONE.EQ.5)SABYG(I)=SABYG(I)*0.4
57      CONTINUE
C       TO READ THE SIESMIC COEFFICIENTS
         READ(3,*)BETA
         WRITE(4,*)'THE SOIL FOUNDATION FACTOR,   IS:    ',BETA
         WRITE(4,1010)
         READ(3,*)IMP
         WRITE(4,*)'IMPORTANCE FACTOR I IS:              ',IMP
         WRITE(4,1010)
C       TO CALCULATE THE MODE PARTICIPATION FACTOR
         DO 58 I=1,3
         EMFI=0.0
         EMFISQR=0.0
         DO 59 J=1,NS
         EMFI=EMFI+AM(J)*AFM(I,J)
         EMFISQR=EMFISQR+AM(J)*(AFM(I,J)**2)
59      CONTINUE
         C(I)=EMFI/EMFISQR
58      CONTINUE
         WRITE(4,*)'           MODE PARTICIPATION FACTOR'
```

```
      WRITE(4,14)
      WRITE(4,*)(C(I),I=1,3)
      WRITE(4,1010)
C     CALCULATION OF THE WEIGHT OF DIFFERENT STOREYS
      DO 72 I=1,NS
      WHT(I)=AM(I)*9.81
72    CONTINUE
      DO 73 I=1,3
      V(I,1)=0.0
      DO 74 J=1,NS
      Q(I,J)=WHT(J)*AFM(I,J)*C(I)*SABYG(I)*BETA*IMP
      V(I,J)=V(I,J)+Q(I,J)
      V(I,J+1)=V(I,J)
74    CONTINUE
73    CONTINUE
      DO 75 J=1,NS
75    CONTINUE
      DO 76 J=1,NS
76    CONTINUE
C     CALCULATION OF GAMMA
      READ(3,*)HGT
      IF(HGT.LE.20)GAMMA=0.4
      IF((HGT.LT.60).AND.(HGT.GT.20))GAMMA=0.4+(0.4/40)*(HGT-20)
      IF((HGT.LT.90).AND.(HGT.GT.60))GAMMA=0.8+(0.2/20)*(HGT-60)
      IF(HGT.GE.90)GAMMA=1.0
      DO 77 I=1,NS
      AI(I)=0.0
      BI(I)=0.0
      DO 78 J=1,3
      AI(I)=AI(I)+ABS(V(J,I))
      BI(I)=BI(I)+(V(J,I)**2)
      SROOTBI(I)=SQRT(BI(I))
78    CONTINUE
77    CONTINUE
      WRITE(4,1010)
      DO 81 I=1,NS
      VN(I)=AI(I)*(1-GAMMA)+SROOTBI(I)*GAMMA
81    CONTINUE
C     TO CALCULATE THE LATERAL FORCE AT EACH NODE
      DO 106 I=1,NS
      MM(I)=VN(I)
106   CONTINUE
      LAT(1)=VN(1)
      DO 104 I=2,NS
      LAT(I)=VN(I)-MM(I-1)
104   CONTINUE
C     EMPERICAL METHOD OF ANALYSIS
      READ(3,*)D
      T=0.09*HGT/(SQRT(D))
C     INTERPOLATION FOR C
      IF(T.LE.0.3)CE=1.0
      IF((T.GT.0.3).AND.(T.LE.0.6))CE=0.75+(0.25/0.3)*(T-0.3)
      IF((T.GT.0.6).AND.(T.LE.1.2))CE=0.475+(0.275/0.6)*(T-0.6)
      IF((T.GT.1.2).AND.(T.LE.1.6))CE=0.375+(0.1/0.4)*(T-1.2)
```

```
      IF((T.GT.1.6).AND.(T.LE.2.0))CE=0.3+(0.075/0.4)*(T-1.6)
      IF((T.GT.2.0).AND.(T.LE.3.0))CE=0.2+(0.1/1.0)*(T-2.0)
      IF(T.GT.3.0)CE=0.2
      IF(ZONE.EQ.1)ALPHA0=0.01
      IF(ZONE.EQ.2)ALPHA0=0.02
      IF(ZONE.EQ.3)ALPHA0=0.04
      IF(ZONE.EQ.4)ALPHA0=0.05
      IF(ZONE.EQ.5)ALPHA0=0.08
      TOTWHT=0.0
      DO 83 I=1,NS
      TOTWHT=TOTWHT+WHT(I)
83    CONTINUE
      TOTVE=CE*ALPHA0*IMP*BETA*TOTWHT
      READ(3,*)(HEIGHT(I),I=1,NS)
      EWHSQR=0.0
      DO 84 I=1,NS
      WHSQR(I)=WHT(I)*HEIGHT(I)**2
      EWHSQR=EWHSQR+WHSQR(I)
84    CONTINUE
      VE(1)=0.0
      DO 85 I=1,NS
      FAC=WHSQR(I)/EWHSQR
      QE(I)=TOTVE*FAC
      VE(I)=VE(I)+QE(I)
      VE(I+1)=VE(I)
85    CONTINUE
C     COMPARISION OF THE EMPERICAL AND MODAL ANALYSIS
      WRITE(4,1001)
1001  FORMAT(5X,']',2X,'EMPERICAL METHOD',1X,']',
     * 'MODE SUPERPOSITION    ',']')
      WRITE(4,1002)
1002  FORMAT(5X,']',2X,'----------------',1X,
     * ']',4X,'------------',4X,']')
      WRITE(4,1003)
1003  FORMAT(5X,']',2X,'---BASE SHEAR---',1X,']',4X,'-BASE SHEAR-'
     * ,4X,']')
      WRITE(4,1004)
1004  FORMAT(5X,']',2X,'        Vb          ',1X,']',4X,'      Vb       '
     * ,4X,']')
      WRITE(4,*)'     ]------------------]--------------------]'
      DO 86 I=1,NS
      WRITE(4,1006)VE(I),VN(I)
1006  FORMAT(5X,']',5X,F8.2,6X,']',6X,F8.2,6X,']')
86    CONTINUE
      WRITE(4,1010)
      WRITE(4,*)'THE LATERAL FORCE AT EACH NODE IS:'
      WRITE(4,14)
      DO 105 I=1,NS
      WRITE(4,1007)LAT(I)
1007  FORMAT(10X,F8.2)
105   CONTINUE
      WRITE(4,1010)
      STOP
      END
```

INPUT FILE FOR THE PROGRAM TO FIND OUT THE LATERAL LOADS

```
3
140
161
161
600680
600680
600680
4
1
1
10.5
14
3.5 7.0 10.5
```

P^2	TIME PERIOD (sec)
0.2228681	794.8112000
0.0804541	6099.0720000
0.0566066	12320.4200000

EMPERICAL METHOD SHEAR Vb	MODE SUPERPOSITION SHEAR Vb
14.21	100.4 4
79.56	173.17
226.61	219.33

THE LATERAL FORCE AT EACH NODE FROM FREE VIBRATION IS:

FLOOR I	100.44
FLOOR II	72.73
FLOOR III	46.16

```
C       PROGRAM FOR ANALYSIS OF PLANE FRAMES USING DIRECT
C       STIFFNESS METHOD
        DIMENSION X(75),Y(75),S(99,99),AML(75,6),
     1  F(99),IP(75),IQ(75)
        DIMENSION A(75),AI(75),U(99),BK(6,6),P(75),D(100)
        OPEN(1,FILE='ANA.IN')
        OPEN(2,FILE='ANA.OUT')
        READ(1,*)NN,NE,NBC,E
        DO 10 I=1,NN
        READ(1,*)J,X(J),Y(J)
  10    CONTINUE
 413    FORMAT(/,9X,I2,20X,F12.2,14X,F12.2)
        DO 15 I=1,NE
        READ(1,*)J,IP(J),IQ(J),A(J),AI(J)
  15    CONTINUE
 415    FORMAT(5X,I2,15X,I2,10X,I2,10X,F12.2,5X,F12.2)
        N=NN*3
        DO 20 I=1,N
  20    F(I)=0.0
        DO 24 I=1,NE
        DO 24 J=1,6
  24    AML(I,J)=0.0
        READ(1,*)NJL
        IF(NJL.EQ.0)GOTO 12
        DO 25 I=1,NJL
        READ(1,*)LJN
        N1=LJN*3-2
        N2=N1+1
        N3=N2+1
  25    READ(1,*)F(N1),F(N2),F(N3)
  12    READ(1,*)NML
        IF(NML.EQ.0)GOTO 34
        DO 30I=1,NML
  30    READ(1,*)J,(AML(J,MN),MN=1,6)
  34    DO 35 I=1,N
        DO 35 J=1,N
  35    S(I,J)=0.0
C       GENERATION OF STIFFNESS MATRIX
        DO 40 M=1,NE
        NB=IP(M)
        NF=IQ(M)
        H=X(NF)-X(NB)
        V=Y(NF)-Y(NB)
        AL=SQRT(H*H+V*V)
        CA=H/AL
        SA=V/AL
        C2=CA**2
        S2=SA**2
        CS=CA*SA
        T1=E*A(M)/AL
        T2=12.0*E*AI(M)/AL**3
        T3=4.0*E*AI(M)/AL
        T4=6.0*E*AI(M)/AL**2
        IB=3*NB-2
```

```
      IF=3*NF
344   FORMAT(3X,I2,5X,I2,9X,I2,5X,F10.5,5X,F13.4,10X,F10.5,/)
      S(IB,IB)=S(IB,IB)+(C2*T1+S2*T2)
      S(IB,IB+1)=S(IB,IB+1)+CS*T1-CS*T2
      S(IB,IB+2)=S(IB,IB+2)+SA*T4
      S(IB,IF-2)=S(IB,IF-2)-C2*T1-S2*T2
      S(IB,IF-1)=S(IB,IF-1)-CS*T1+CS*T2
      S(IB,IF)=S(IB,IF)+SA*T4
      S(IB+1,IB+1)=S(IB+1,IB+1)+S2*T1+C2*T2
      S(IB+1,IB+2)=S(IB+1,IB+2)-CA*T4
      S(IB+1,IF-2)=S(IB+1,IF-2)-CS*T1+CS*T2
      S(IB+1,IF-1)=S(IB+1,IF-1)-S2*T1-C2*T2
      S(IB+1,IF)=S(IB+1,IF)-CA*T4
      S(IB+2,IB+2)=S(IB+2,IB+2)+T3
      S(IB+2,IF-2)=S(IB+2,IF-2)-SA*T4
      S(IB+2,IF-1)=S(IB+2,IF-1)+CA*T4
      S(IB+2,IF)=S(IB+2,IF)+0.5*T3
      S(IF-2,IF-2)=S(IF-2,IF-2)+C2*T1+S2*T2
      S(IF-2,IF-1)=S(IF-2,IF-1)+CS*T1-CS*T2
      S(IF-2,IF)=S(IF-2,IF)-SA*T4
      S(IF-1,IF-1)=S(IF-1,IF-1)+S2*T1+C2*T2
      S(IF-1,IF)=S(IF-1,IF)+CA*T4
      S(IF,IF)=S(IF,IF)+T3
      S(IB+1,IB)=S(IB,IB+1)
      S(IB+2,IB)=S(IB,IB+2)
      S(IF-2,IB)=S(IB,IF-2)
      S(IF-1,IB)=S(IB,IF-1)
      S(IF,IB)=S(IB,IF)
      S(IB+2,IB+1)=S(IB+1,IB+2)
      S(IF-2,IB+1)=S(IB+1,IF-2)
      S(IF-1,IB+1)=S(IB+1,IF-1)
      S(IF,IB+1)=S(IB+1,IF)
      S(IF-2,IB+2)=S(IB+2,IF-2)
      S(IF-1,IB+2)=S(IB+2,IF-1)
      S(IF,IB+2)=S(IB+2,IF)
      S(IF-1,IF-2)=S(IF-2,IF-1)
      S(IF,IF-2)=S(IF-2,IF)
      S(IF,IF-1)=S(IF-1,IF)
      F(IB)=F(IB)+(CA*AML(M,1)-SA*AML(M,2))
      F(IB+1)=F(IB+1)+CA*AML(M,2)+SA*AML(M,1)
      F(IB+2)=F(IB+2)+AML(M,3)
      F(IF-2)=F(IF-2)+CA*AML(M,4)-SA*AML(M,5)
      F(IF-1)=F(IF-1)+SA*AML(M,4)+CA*AML(M,5)
      F(IF)=F(IF)+AML(M,6)
  40  CONTINUE
C     APPLICATION OF BOUNDARY CONDITIONS
      DO 60 I=1,NBC
      READ(1,*)JB,SB
      S(JB,JB)=1.0
      F(JB)=SB
      DO 70 J=1,N
      IF(J.EQ.JB)GOTO 65
      S(JB,J)=0.0
      S(J,JB)=0.0
```

```
   65    CONTINUE
   70    CONTINUE
   60    CONTINUE
         CALL GAUSS(S,F,U,N)
         WRITE(2,234)
  234    FORMAT(//,2X,'MEMBER FORCES IN LOCAL COORDINATE SYSTEM ',//)
         WRITE(2,2378)
 2378    FORMAT(1X,'MEMBER',1X,'NODE',7X,'AXIALFORCE',7X,'SHEARFORCE',
     1   7X,'BENDINGMOMENT',//)
C        CALCULATION OF MEMBER END FORCES
         DO 100 I=1,NE
         NB=IP(I)
         NF=IQ(I)
         H=X(NF)-X(NB)
         V=Y(NF)-Y(NB)
         AL=SQRT(H*H+V*V)
         CA=H/AL
         SA=V/AL
         C2=CA*CA
         S2=SA*SA
         CS=CA*SA
         T1=E*A(I)/AL
         T2=12.0*E*AI(I)/AL**3
         T3=4.0*E*AI(I)/AL
         T4=6.0*E*AI(I)/AL**2
         BK(1,1)=T1
         BK(2,1)=0.0
         BK(3,1)=0.0
         BK(4,1)=-T1
         BK(5,1)=0.0
         BK(6,1)=0.0
         BK(1,2)=0.0
         BK(2,2)=T2
         BK(3,2)=-T4
         BK(4,2)=0.0
         BK(5,2)=-T2
         BK(6,2)=-T4
         BK(1,3)=0.0
         BK(2,3)=-T4
         BK(3,3)=T3
         BK(4,3)=0.0
         BK(5,3)=T4
         BK(6,3)=0.5*T3
         DO 110 II=1,6
         BK(II,4)=-BK(II,1)
         BK(II,5)=-BK(II,2)
  110    BK(II,6)=BK(II,3)
         BK(3,6)=0.5*T3
         BK(6,6)=T3
         IB=3*NB-2
         IF=3*NF
         D(1)=CA*U(IB)+SA*U(IB+1)
         D(2)=CA*U(IB+1)-SA*U(IB)
         D(3)=U(IB+2)
```

```
      D(4)=CA*U(IF-2)+SA*U(IF-1)
      D(5)=-SA*U(IF-2)+CA*U(IF-1)
      D(6)=U(IF)
      DO 120 L=1,6
      P(L)=0.0
      DO 120 J=1,6
120   P(L)=P(L)+BK(L,J)*D(J)
      DO 130 L=1,6
130   P(L)=P(L)-AML(I,L)
      WRITE(2,206)I,NB,P(1),P(2),P(3)
      WRITE(2,207)I,NB,P(4),P(5),P(6)
207   FORMAT(8X,I3,3(3X,F14.4),/)
206   FORMAT(1X,I3,4X,I3,3(3X,F14.4),/)
100   CONTINUE
      STOP
      END

C       GAUSSIAN ELIMINATION
        SUBROUTINE GAUSS(A,C,X,N)
        DIMENSION A(99,99),C(99),X(99)
        DO 10 I=1,N-1
        DO 10 J=I+1,N
        D=A(J,I)/A(I,I)
        DO 30 K=1,N
   30   A(J,K)=A(J,K)-A(I,K)*D
        C(J)=C(J)-C(I)*D
   10   CONTINUE
        DO 40 I=N,1,-1
        TOT=0.0
        DO 50 J=I+1,N
   50   TOT=TOT+A(I,J)*X(J)
        X(I)=(C(I)-TOT)/A(I,I)
        TOT=0
   40   CONTINUE
        RETURN
        END
```

INPUT FILE FOR FRAME ANALYSIS

```
12 15 9 22.1E6
1 0 0
2 7 0
3 14 0
4 0 3.5
5 7 3.5
6 14 3.5
7 0 7
8 7 7
9 14 7
10 0 10.5
11 7 10.5
12 14 10.5
1 1 4 0.18 0.0054
2 2 5 0.18 0.0054
3 3 6 0.18 0.0054
4 4 5 0.125 2.6E-3
5 5 6 0.125 2.6E-3
6 4 7 0.18 0.0054
7 5 8 0.18 0.0054
8 6 9 0.18 0.0054
9 7 8 0.125 2.6E-3
10 8 9 0.125 2.6E-3
11 7 10 0.18 0.0054
12 8 11 0.18 0.0054
13 9 12 0.18 0.0054
14 10 11 0.125 2.6E-3
15 11 12 0.125 2.6E-3
3
10
100.4 0 0
7
72.7 0 0
4
46.2 0 0
6
14
0 -25 29.16 0 -25 -29.16
15
0 -25 29.16 0 -25 -29.16
9
0 -28.75 33.55 0 -28.75 -33.55
10
0 -28.75 33.55 0 -28.75 -33.55
4
0 -28.75 33.55 0 -28.75 -33.55
5
0 -28.75 33.55 0 -28.75 -33.55
1 0
2 0
3 0
```

NUMBER OF NODES =12
NUMBER OF ELEMENTS =15
NUMBER OF BOUNDARY CONDITIONS = 9
ELASTIC MODULUS =0.2210E+08

NODE NUMBER	X COORDINATE	Y COORDINATE
1	0.00	0.00
2	7.00	0.00
3	14.00	0.00
4	0.00	3.50
5	7.00	3.50
6	14.00	3.50
7	0.00	7.00
8	7.00	7.00
9	14.00	7.00
10	0.00	10.50
11	7.00	10.50
12	14.00	10.50

MEMBER	BACK NODE	FORE NODE	AREA	MOMENT OF INERTIA
1	1	4	0.18	0.01
2	2	5	0.18	0.01
3	3	6	0.18	0.01
4	4	5	0.13	0.00
5	5	6	0.13	0.00
6	4	7	0.18	0.01
7	5	8	0.18	0.01
8	6	9	0.18	0.01
9	7	8	0.13	0.00
10	8	9	0.13	0.00
11	7	10	0.18	0.01
12	8	11	0.18	0.01
13	9	12	0.18	0.01
14	10	11	0.13	0.00
15	11	12	0.13	0.00

NUMBER OF JOINT LOADS = 3

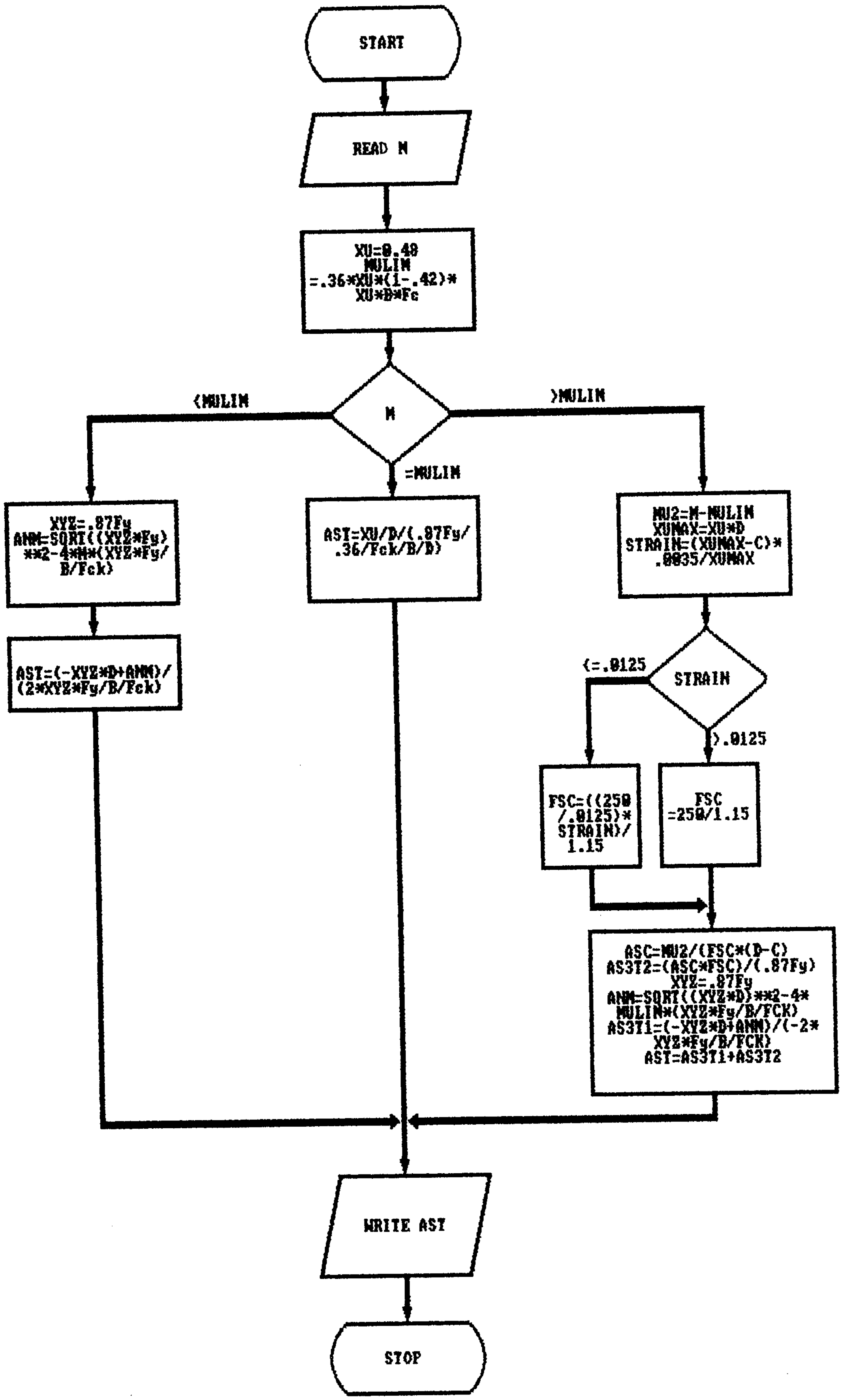
1	1	4	0.18000	0.0054	3.50000
2	2	5	0.18000	0.0054	3.50000
3	3	6	0.18000	0.0054	3.50000
4	4	5	0.12500	0.0026	7.00000
5	5	6	0.12500	0.0026	7.00000
6	4	7	0.18000	0.0054	3.50000
7	5	8	0.18000	0.0054	3.50000
8	6	9	0.18000	0.0054	3.50000
9	7	8	0.12500	0.0026	7.00000
10	8	9	0.12500	0.0026	7.00000
11	7	10	0.18000	0.0054	3.50000
12	8	11	0.18000	0.0054	3.50000
13	9	12	0.18000	0.0054	3.50000
14	10	11	0.12500	0.0026	7.00000
15	11	12	0.12500	0.0026	7.00000

NODAL DISPLACEMENTS IN GLOBAL COORDINATE SYSTEM

NODE	HORIZONTAL DEFLECTION	VERTICAL DEFLECTION	ROTATION (IN RADIANS)
1	0.000000	0.000000	0.000000
2	0.000000	0.000000	0.000000
3	0.000000	0.000000	0.000000
4	0.006244	-0.000001	0.002510
5	0.006187	-0.000148	0.002071
6	0.006149	-0.000141	0.002285
7	0.015781	-0.000006	0.002288
8	0.015664	-0.000245	0.002008
9	0.015610	-0.000229	0.002149
10	0.023021	-0.000011	0.001651
11	0.022796	-0.000291	0.001155
12	0.022706	-0.000266	0.001300

MEMBER FORCES IN LOCAL COORDINATE SYSTEM

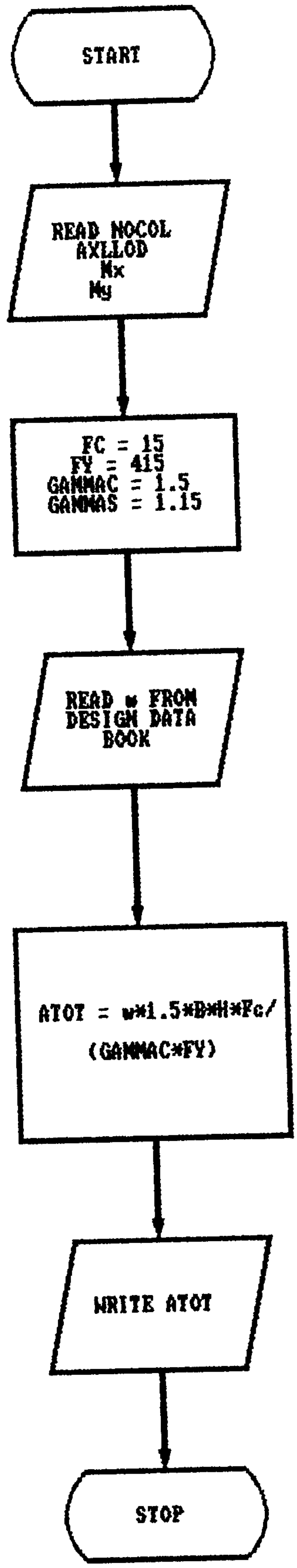
MEMBER	NODE	AXIALFORCE	SHEARFORCE	BENDINGMOMENT
1	1	1.6449	61.8570	-193.8256
	4	-1.6449	-61.8570	-22.6740
2	2	168.5927	85.5986	-220.4149
	5	-168.5927	-85.5986	-79.1801
3	3	159.7621	71.8300	-203.6117
	6	-159.7621	-71.8300	-47.7935
4	4	22.4656	-3.1851	81.8238
	5	-22.4656	60.6851	141.7216
5	5	14.9873	-1.9139	72.0184
	6	-14.9873	59.4139	142.6292
6	4	4.8300	38.1228	-59.1498
	7	-4.8300	-38.1228	-74.2799
7	5	109.8217	78.1204	-134.5598
	8	-109.8217	-78.1204	-138.8616
8	6	100.3482	56.8428	-94.8358
	9	-100.3482	-56.8428	-104.1142
9	7	46.1768	-0.9945	72.8534
	8	-46.1768	58.4945	135.3580
10	8	21.4053	-0.5296	67.7722
	9	-21.4053	58.0296	137.1849
11	7	5.8245	11.5999	1.4263
	10	-5.8245	-11.5999	-42.0264
12	8	51.8568	53.3524	-64.2689
	11	-51.8568	-53.3524	-122.4647
13	9	42.3186	35.4395	-33.0708
	12	-42.3186	-35.4395	-90.9675
14	10	88.7988	5.8245	42.0267
	11	-88.7988	44.1755	92.2021
15	11	35.4414	7.6813	30.2629
	12	-35.4414	42.3187	90.9677



FLOW CHART FOR DESIGN OF BEAMS

```
C       BEAM DESIGN
C       ASSUME THE SIZE OF THE BEAM AS 250x500
        INTEGER DIAM
        REAL M
        OPEN(1, FILE = 'BEAMIN.FOR')
        OPEN(2, FILE = 'BEAM.OUT')
        READ(1,*)NOBEAM
        DO 100 I = 1,NOBEAM
        READ(1,*)M
        FY=250.
        FCK=15.
        B=250.
        DE=500.
        C=40.
        D=DE-C
        IF(FY.EQ.250)XUMAXBYD=0.53
        IF(FY.EQ.415)XUMAXBYD=0.48
        IF(FY.EQ.500)XUMAXBYD=0.46
        MULIM=0.36*XUMAXBYD*(1-0.42*XUMAXBYD)*B*D*D*FCK
        IF(M.LT.MULIM)N=1
        IF(M.EQ.MULIM)N=2
        IF(M.GT.MULIM)N=3

        IF(N.EQ.1)THEN
C       SINGLY REINFORCED SECTION
        BFCK=B*FCK
        XYZ=0.87*FY
        ANM=SQRT((XYZ*D)**2-4*M*(XYZ*FY/BFCK))
        AS1T=(-XYZ*D+ANM)/(-2*XYZ*FY/BFCK)
        ENDIF

        IF(N.EQ.2)THEN
        AS2T=XUMAXBYD/(0.87*FY/0.36/FCK/B/D)
        ENDIF

        IF(N.EQ.3)THEN
C       DOUBLY REINFORCED SECTION
        MU2=M-MULIM
        XUMAX=XUMAXBYD*D
        STRAIN=(XUMAX-C)*0.0035/XUMAX
        IF (FY.EQ.250)THEN
        IF (STRAIN .LE. 0.0125)FSC=((250.0/0.0125)*STRAIN)/1.15
        IF(STRAIN .GT. 0.0125 )FSC=250/1.15
        ENDIF
        IF (FY.EQ.415)THEN
        IF (STRAIN .LE. 0.002075)FSC=((415.0/0.002075)*STRAIN)/1.15
        IF(STRAIN .GT. 0.002075)FSC=415/1.15
        ENDIF
        IF (FY.EQ.500)THEN
        IF (STRAIN .LE. 0.0025)FSC=((500.0/0.0025)*STRAIN)/1.15
        IF(STRAIN .GT. 0.0025 )FSC=500/1.15
        ENDIF
        ASC=MU2/(FSC*(D-C))
        AS3T2=(ASC*Fsc)/(0.87*FY)
```

```
      BFCK=B*FCK
      XYZ=0.87*FY
      ANM=SQRT((XYZ*D)**2-4*MULIM*(XYZ*FY/BFCK))
      AS3T1=(-XYZ*D+ANM)/(-2*XYZ*FY/BFCK)
      AS3T=AS3T1+AS3T2
      ENDIF
      READ(1,*)DIAM

C     SUMMARY

      WRITE(2,*)'                     SUMMARY OF THE DESIGN'
      WRITE(2,10003)

      IF(N.EQ.1)THEN
      WRITE(2,*)'                     SINGLY REINFORCED SECTION'
      WRITE(2,*)'THE REINFORCEMENT IS:',AS1T,'mm}'
      NOBARS=AS1T/(3.141592654*((DIAM/2)**2))
      NOBARS=NINT(NOBARS+0.5)
      WRITE(2,10004)NOBARS,DIAM
10004 FORMAT(1X,'PROVIDE ',I4,' NOS OF ',I3,' mm DIA BAR')
      ENDIF

      IF(N.EQ.2)THEN
      WRITE(2,*)'                     BALANCED REINFORCED SECTION'
      WRITE(2,*)'THE REINFORCEMENT IS:',AS2T,'mm}'
      NOBARS=AS2T/(3.141592654*((DIAM/2)**2))
      NOBARS=NINT(NOBARS+0.5)
      WRITE(2,10005)NOBARS,DIAM
10005 FORMAT(1X,'PROVIDE ',I4,' NOS OF ',I3,' mm DIA BAR')
      ENDIF

      IF(N.EQ.3)THEN
      WRITE(2,*)'                     DOUBLY REINFORCED SECTION'
      WRITE(2,*)'THE TOTAL REINFORCEMENT IS:',AS3T,'mm}'
      WRITE(2,*)'THE TENSION REINFORCEMENT IS:',AS3T1
      WRITE(2,*)'THE TENSION REINFORCEMENT IS:',AS3T2


      NOBARS1=AS3T1/(3.141592654*((DIAM/2)**2))
      NOBARS1=NINT(NOBARS1+0.5)
      NOBARS2=AS3T2/(3.141592654*((DIAM/2)**2))
      NOBARS2=NINT(NOBARS2+0.5)
      WRITE(*,10006)NOBARS1,DIAM
10006 FORMAT(1X,'PROVIDE ',I4,' NOS OF ',I3,' mm DIA BAR AS
     * TENSION REINFORCEMENT')
      WRITE(*,10007)NOBARS2,DIAM
10007 FORMAT(1X,'PROVIDE ',I4,' NOS OF ',I3,' mm DIA BAR AS
     * COMPRESSION REINFORCEMENT')
      WRITE(*,10003)
10003 FORMAT('-------------------------------------------------------')
      ENDIF
100   CONTINUE
      STOP
      END
```



FLOW CHART FOR THE DESIGN OF COLUMNS

```
C       SUBROUTINE  COLUMN
C       COLUMN DESIGN
C       ASSUME COLUMN SIZE AS 250x500,STEEL=FE415,MIX=M15
        REAL MuX,MuY,MX,MY,Nu,OMEGA,NOBARS
        INTEGER CNO
        OPEN(1,FILE='eq.FOR')
        OPEN(2,FILE='eqAST1.FOR')
        READ(1,*)NOCOL
        DO 10 K=1,NOCOL
        READ(1,*)AXLLOD,MX,MY
        MX=ABS(MX)*1E6
        MY=ABS(MY)*1E6
        AXLLOD=AXLLOD*1E3
        H=600.
        B=300.
        C=60.
        FC=15.
        FY=415.
        GAMMAC=1.5
        GAMMAS=1.15
        cno = k
        N=AXLLOD
        HBYB=H/B
        CBYH=C/H
        MuX=(GAMMAC/1.5)*(MX/(FC*B*H*H))
        MuY=(GAMMAC/1.5)*(MY/(FC*B*B*H))
        Nu=(GAMMAC/1.5)*(N/(FC*B*H))
        WRITE(*,10001)MuX,MuY,Nu
10001   FORMAT('INPUT OMEGA FOR:fx=',F5.3,'fy=',F5.3,'Nu=',F3.1)
        READ(*,*)OMEGA
        ATOT=(OMEGA*1.5*B*H*FC)/(GAMMAC*FY)
        IF (ATOT.LT.(0.0035*B*H))ATOT=.0035*B*H
        ASTI=ATOT/4.
        WRITE(2,101)CNO,ATOT
101     FORMAT(5X,I3,10X,F10.0,'mm}')
10      CONTINUE
        STOP
        END
```

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