SEISMIC SAFETY OF BUILDINGS IN UPGRADED EARTHQUAKE ZONES

A DISSERTATION
submitted in partial fulfilment
of the requirements for the Degree
of
MASTER OF ENGINEERING
in
EARTHQUAKE ENGINEERING
WITH SPECIALIZATION IN STRUCTURAL DYNAMICS

By
J P. DESAI

DEPARTMENT OF EARTHQUAKE ENGINEERING
UNIVERSITY OF ROORKEE
ROORKEE (U.P.)
INDIA

October 1974

C E R T I F I C A T E

Certified that the thesis entitled, "Seismic Safety of Buildings in Upgraded Earthquake Zones", which is being submitted by Mr. Jagdish Pragji Desai in partial fulfilment for the award of the degree of Master of Engineering in Earthquake Engineering with Specialization in STRUCTURAL DYNAMICS, of University of Roorkee, Roorkee, is a record of student's own work carried out by him under our supervision and guidance. The matter embodied in this thesis has not been submitted for the award of any other degree or diploma.

This is also to be certified that he has worked for a period of seven months from January to July 1974 for preparing this thesis for Master of Engineering Degree at the University.

Dr. B. C. Mathur
Associate Professor in
Structural Dynamics
Department of Earthquake
Engineering
University of Roorkee
Roorkee

Dr. A. R. Chandrasekaran
Professor in
Structural Dynamics
Department of Earthquake
Engineering
University of Roorkee
Roorkee

SYNOPSIS

The design seismic coefficients provided for in the code, are quite small. Because of this, frequent upgrading of seismic zones becomes necessary as and when major earthquakes occur. In the present study, it is shown that buildings have sufficient built-in seismic safety, so that such upgrading of seismic zones may not be viewed with a great concern. Such upgrading only helps the construction of new buildings on a more rational basis in such areas.

In the present study, multistorey reinforced concrete framed buildings are first analysed and designed by working stress method without consideration of earthquake forces and the design governed by vertical loads only. A check is then made to determine as to how much earthquake force the buildings can safely bear under the following conditions :

1. Material stresses just reaching yield point
2. Structure going in the inelastic range
3. By ultimate load analysis.

A comparison is made for the above three cases. A critical review of IS 1893 : 1970 code provisions as far as buildings are concerned is also given.

ACKNOWLEDGEMENT

The author expresses his sincere thanks to Dr. A.R. Chandrasekaran, Professor and Dr. B.C. Mathur, Associate Professor in Structural Dynamics, Earthquake Engineering Department, University of Roorkee, Roorkee for their valuable guidance and constant encouragement throughout this study.

The computational work was done on IBM 1620 installation at the Structural Engineering Research Centre, Roorkee. The author takes this opportunity to thank this organisation for the computer facility made available to him.

INDEX

CHAPTER - I

INTRODUCTION

According to IS 1893:1962[22], Bombay and the whole Decan plateau area was classified as zone zero with seismic coefficient of value zero. On December 11, 1967 a strong earthquake[4] with a magnitude of 6.5, occured at 04-21 IST with its epicentre very close to the Koyna Hydroelectric project in the Maharashtra State. There was then the realization that the stable area of Decan plateau was also susceptible to earthquake. This necessitated seismic upgrading of this area. In the revised code of IS 1893: 1970[23], Bombay was placed in Zone III with a seismic coefficient of 0.04. Due to this, buildings which had not been designed for earthquake forces are now supposed to withstand the earthquake forces corresponding to this seismic coefficient.

Most of the reinforced concrete buildings in the affected zone withstood the Koyna earthquake though they were not designed for it. This is important from the point of view of building design. It is quite possible that the structural design method adopted provides for a certain margin of safety ~~of safety~~ in such structures. Also by permitting certain amount of inelastic deformations the structure can withstand rather severe earthquake forces.

The present thesis examines the following aspects of earthquake resistance in buildings:

1. Safety of buildings if seismic coefficients are upgraded.
2. Maximum earthquake force the buildings can safely withstand.
3. Height of multistorey buildings upto which the earthquake force does not govern the design as per present IS Code.

In multistorey buildings, the design[12] is governed by one of the following combination of forces:

1. Vertical loads
2. Vertical loads + wind loads
3. Vertical loads + Earthquake loads.

Wind and earthquake forces being occasional in nature, an increase of $33\frac{1}{3}$ % in working stresses is allowed when design is governed by one of them. Further, when the earthquake force is governing, some reduction in live loads is also allowed.

Analysis for wind loads has not been dealt with in the present study, as the parameter for wind loads is the exposed area of the building whereas for earthquake forces, it is mass and stiffness distribution, and hence no common ground for comparison can be specified. Also wind force is not

not considered to act when earthquake is governing because, there is little possibility of earthquake accompanied by high winds.

The present thesis deals mainly with the study of multistorey reinforced concrete framed buildings. Analysis and design is illustrated by the case study of a seven-storeyed building as given in Appendix - I. Buildings of 1, 4, 7, 10 15 and 20 storeys are designed by working stress method, without consideration of earthquake forces and the design governed by vertical loads only. A check is then made to determine as to how much earthquake force the building can safely bear under the following conditions.

1. Without sustaining any damage and
2. With moderate damage but without total collapse.

These aspects are studied by allowing the stress to rise upto yield stress in the first case and by allowing the structure to go into the inelastic range in the second. The relationship between the increase in material stresses and the seismic coefficient has been determined.

Seismic coefficient method for computing earthquake forces has been used in the present stydy. The presents earthquake code has given this method for buildings not exceeding 40 m in height. For buildings greater than

40 m in height and upto 90 m, modal analysis is recommended. However, as an alternative seismic coefficient method has also been permitted for zone I to zone IV. Hence the adoption of seismic coefficient method for buildings upto 20 storeys, in this study is justified.

The following are the limitations of the present study :

(i) Effect of torsion is not considered.

(ii) Effect of simultaneous occurance of earthquake force along both axes has not been considered.

(iii) The horizontal relative displacement due to earthquake forces between two successive floors has not been investigated. This may result in P-Δ effect.

(iv) The non-structural damage due to earthquake has not been considered. Though the building may withstand a severe earthquake but the cost of repairs sometimes are very heavy.

(v) The effect of change in aspect ratio of building has not been considered.

For frames designed either for vertical loads (or vertical loads + earthquake forces with allowable $33\frac{1}{3}\%$ increase in stresses), the seismic coefficients withstood are computed for the following conditions.

(1) Material stresses just reaching yield point.

(2) Structure going in the inelastic range. The concept of reduction factor is used.

(3) By ultimate load analysis.

A comparison is made for the above three cases. A critical review of the IS 1893 : 1970 code provisions as far as buildings are concerned is also given.

In the present study, it has been established that very high seismic coefficients can be withstood by buildings in the yield, inelastic and ultimate stage respectively. Compared to these, the design seismic coefficients given in the code are very small. However, as and when a major earthquake occurs, upgrading of zones becomes necessary. It is shown that building have sufficient built-in seismic safety, so that such upgrading of zones may not be viewed with a great concern. However, such upgrading does help the construction of new buildings on a more rational basis.

CHAPTER - II

SEISMIC SAFETY IN THE ELASTIC RANGE

The object of this chapter is to compute the seismic coefficient (S.C.) withstood by the building which is designed only for vertical loads, for the following conditions.

CASE 1 : When an increase of $33\frac{1}{3}\%$ in working stresses is allowed.

CASE 2 : When the material stresses just reach the yield point.

Seismic safety of the building has been investigated by lateral load analysis with seismic coefficient method.

For static analysis, live loads have been taken as per IS 456 : 1964 and for lateral load analysis, live loads as per IS 1893 : 1970 have been taken.

Lateral load analysis[13]

The details of the building chosen for this study, are given in Appendix I. Kani's method has been used here for lateral load analysis. The procedure adopted for computing storey moments and stiffnesses is as follows.

By lumping the masses at floor levels, the total vertical load at roof level works out as 884 tonnes and the total vertical load at typical floor level works out as 891 tonnes, for the building under considera-tion.

Referring to para 4.2 of IS 1893:1970, the base shear is given by

$$V_B = c \cdot \alpha_h \cdot \beta \cdot W \quad \ldots 2.1$$

Where c = flexibility coefficient,

α_h = seismic coefficient of the particular zone,

β = soil foundation factor, and

W = total load on which seismic force is to be computed.

Also the distribution of forces along height is given by

$$Q_i = V_B \times \frac{W_i \; h_i^2}{\sum_1^n W_i \; h_i^2} \quad \ldots 2.2$$

Where Q_i = lateral forces at roof or floor i,

V_B = the base shear as worked out in 2.1

W_i = the weight considered to be acting at the level of the roof or floor i,

h_i = height of the roof or floor i above the base of the building, and

n = number of storeys including the basement floors.

Q forces (t) computed at various floor levels for the seven storeyed building under consideration, are given in the table below for seismic coefficient of 0.01.

Floor No.	7	6	5	4	3	2	1
Q(t)	12.1	8.9	6.3	4.0	2.3	1.0	0.2

The earthquake force is assumed to be acting along one of the principal axis of the building at a time when the structure is designed for the horizontal seismic force only. Hence the forces computed above will act upon the entire building either in

(i) transverse direction or in
(ii) longitudinal direction.

These forces along any particular direction (transverse or longitudinal) are resisted by all the frames along that direction.

The Q forces at any storey level are distributed to various frames in proportion to their stiffnesses in that direction.

The stiffness of a frame at any storey level is taken equal to the sum of the column stiffnesses in that storey and the distribution of Q forces to various frames is made accordingly.

Storey moment is computed as $V_h \cdot h/3$ where

V_h = shear in the storey and

h = height of the storey measured from top

Analysis is completed by computing the floorwise storey moments and stiffnesses. Kani's method[18] has been used here for computations.

Case I

For the beam, only under-reinforced section is taken so that the steel stress rises from 1400 to 1867 kg/cm². Columns are designed such that the reinforcement is symmetrical. For the column, the stress in concrete is the governing factor initially beacuse the vertical loads govern the design at this stage. Bending stresses in column concrete are allowed to rise from 50, 70 and 85 Kg/cm² to 66.7, 93.3 and 113.3 Kg/cm² respectively, for M150, M200 and M250 concrete. For the beam, and a straight live load is taken for the increase in stress against resisting moment developed.

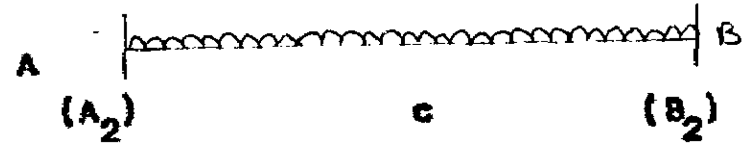
The computations of moments in the beam and moment and the direct load on the column are illustrated for a seven storey building in Appendix I. Similar computations are repeated for single, four, ten, fifteen and twenty storey buildings, keeping plan dimensions and the storey height same. The results obtained for the critical frame, for the beam and column are given in Table 2.1 and 2.2 respectively. The key points A, C, B are the external, centre and internal points respectively for the critical frame.

The above as well as all subsequent sets of calculations have been repeated (1) for ten percent reduced relative stiffness of columns and (2) for ten percent reduced relative stiffness of beam, in order to see their effects on the results obtained. A two to five percent variation in results were observed. The variation ~~of ten percent~~ in the relative stiffness which is assumed as ten percent is reasonable because that is a difference which may occur due to differences in designs adopted.

Case 2

When the stress in steel in the beam is allowed to rise upto yield stress, a bilinear pattern of increase in stress VS seismic coefficient ~~withstood~~ is obtained. Again, either the beam or the column governs the design. For the column, the corresponding stresses in concrete are taken as 95, 125 and 155 Kg/cm² respectively for M150, M200 and M250 concrete. For the seismic coefficient found earlier, the corresponding seismic coefficient for increased stress is found out and the bilinear ~~law~~ pattern is ~~established~~ obtained.

Table for beam moments at key points (tm)



CONDITION OF LOADING	SINGLE STOREY			4 - STOREY			7 - STOREY		
	A	C	B	A	C	B	A	C	B
1. Dead load (D. L.)	-6.8	+12.6	-6.4	-7.4	+7.7	-7.1	-8.0	+7.2	-7.5
2. Normal live load (N. L. L.)	-1.2	+2.26	-1.2	-2.86	+2.93	-2.75	-3.0	+2.8	-2.9
3. D. L. + N. L.L	- 8	+14.86	-7.6	-10.26	+10.63	-9.85	-11.0	-10.4	-10.4
4. Reduced live load (R. L. L.)	0.0	0.0	0.0	- 0.72	+0.73	-0.69	-0.75	+0.7	-0725
5. Earthquake Load (E. L) S.C. = 0.01	+0.233	-	∓ 0.226	±1.17	-	∓ 1.14	±1.71	-	∓ 1.69
6. Seismic Coeff. resisted for 33 1/3 % Increase in stresses		0.165			0.058 (First storey)			0.035 (Second storey)	

Table Contd --

Table 2.1 (Continued)

CONDITION OF LOADING	10- Storey			15 - storey			20 - storey		
	A	C	B	A	C	B	A	C	B
1.	-8.1	+7.0	-7.8	-9.3	+5.83	-9.1	-9.55	+5.51	-9.4
2.	-3.1	+2.7	-3.0	-3.55	+2.22	-3.45	-3.64	+2.1	-3.58
3.	-11.2	+9.7	-10.8	-12.85	+8.05	±12.55	-13.19	+7.61	-12.98
4.	-0.8	+0.7	-0.8	-0.89	+0.55	-0.86	-0.91	+0.52	-0.895
5.	± 2.27	-	∓2.24	± 2.74	-	∓ 2.70	± 3.21	-	∓ 3.2
6.		0.026			0.025			0.022	
		(Fourth storey)			(Sixth storey)			(Seventh storey)	

TABLE 2.2

Table for Moment (tm) and Direct Load (t) on Column

CONDITION OF LOADING	Single Sotrey Moment	Single Sotrey Load	4-storey Moment	4-storey Load	7- storey Moment	7- storey Load
1. D. L. + N. L. L.	8	13. 3	5.95	68.9	5. 6	117. 3
2. D. L. + R. L. L.	6.7	15.7	4.73	61.2	4.5	106.8
3. EQ Load (S. C. = 0.01)	0.287	0.0675	~~0.0675~~ 0.87	~~0.87~~ 1.02	1.24	2.34
4. S. C. resisted for 33 1/3 % increase in concrete stress	0.1575		0.0725		0.06	

Table Contd... ...

Table 2.2 (Continued)

CONDITION LOADING	10-storey		15 - storey		20 - storey	
	Moment	Load	Moment	Load	Moment	Load
1.	6.4	167.4	6.75	250.6	7.05	334.2
2.	5.08	152.2	5.36	228.2	5.59	304
3.	2.1	4.75	3.3	9.14	5	15
4.		0.045		0.045		0.04

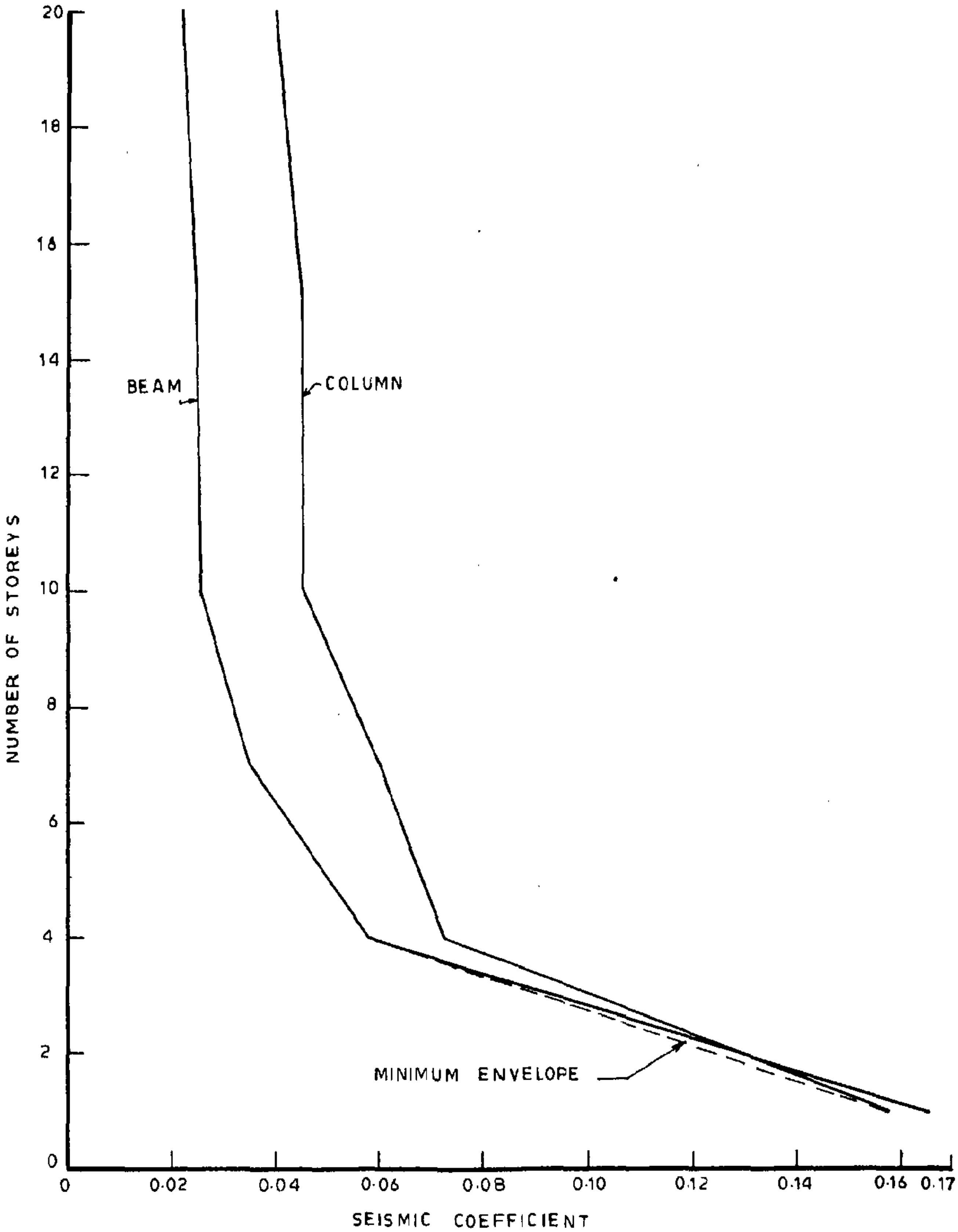


FIG. 2.1 – NUMBER OF STOREY VS SEISMIC COEFFICIENT FOR AN INCREASE OF $33\frac{1}{3}\%$ IN WORKING STRESSES

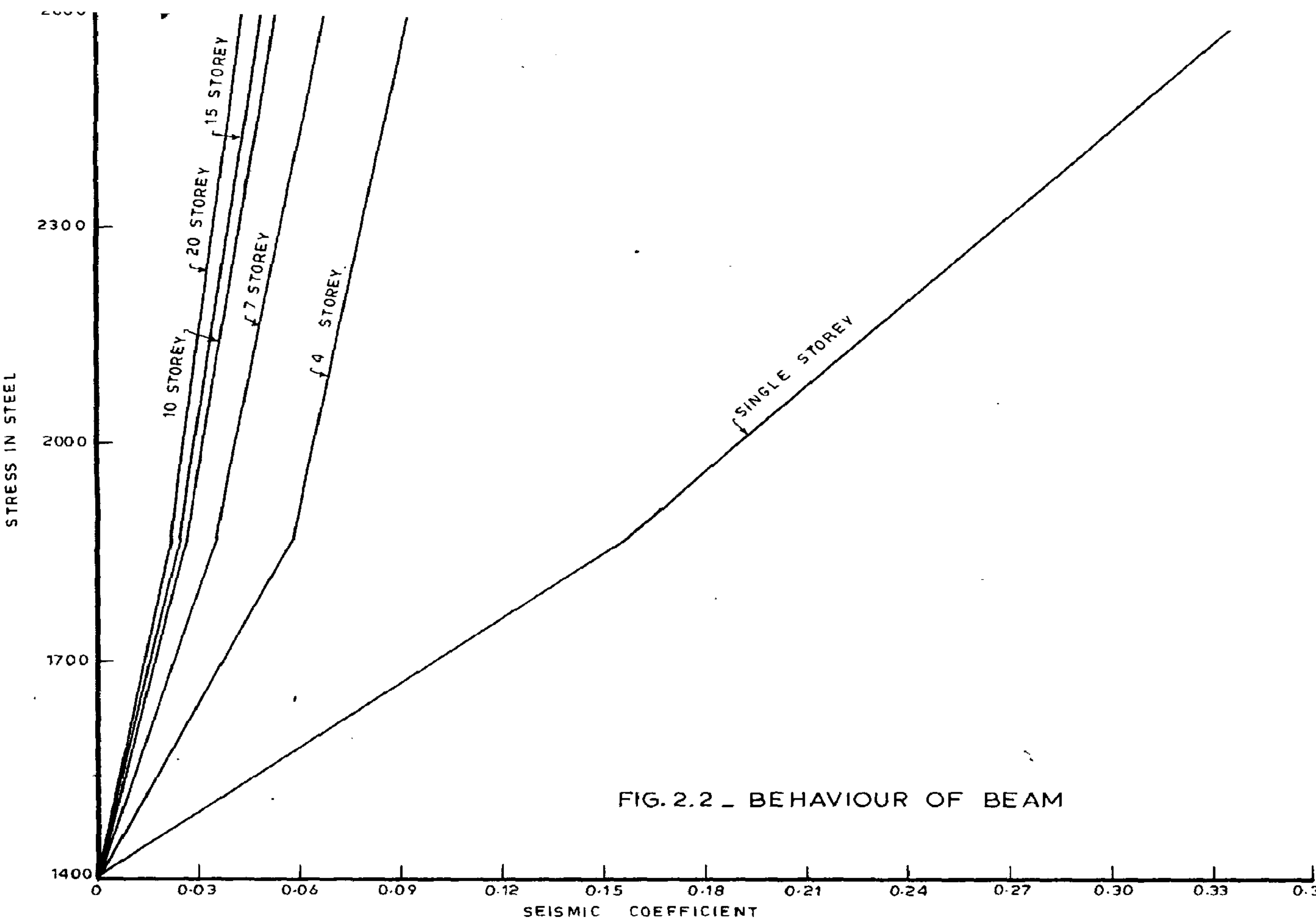


FIG. 2.2 _ BEHAVIOUR OF BEAM

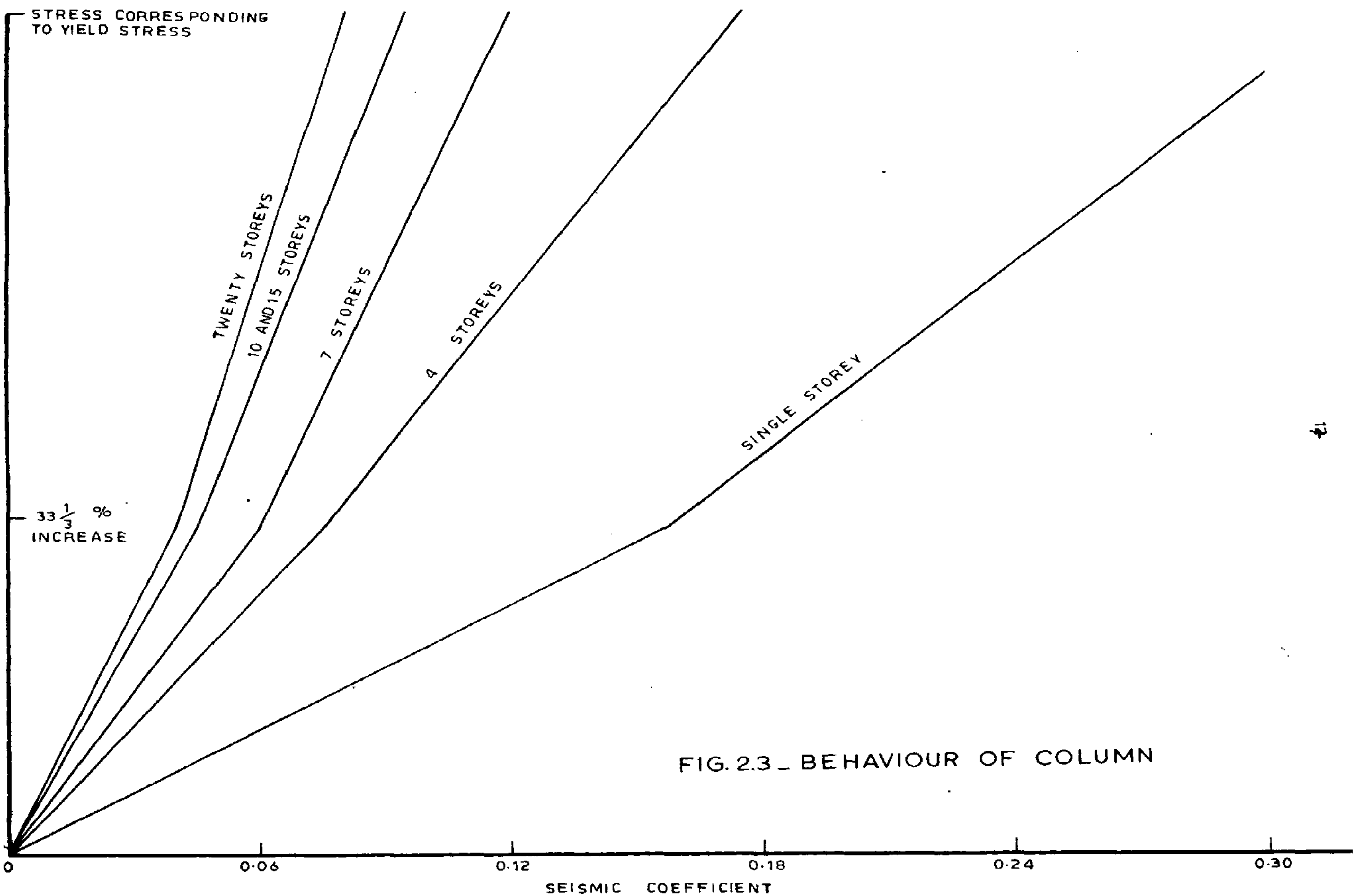


FIG. 2.3 _ BEHAVIOUR OF COLUMN

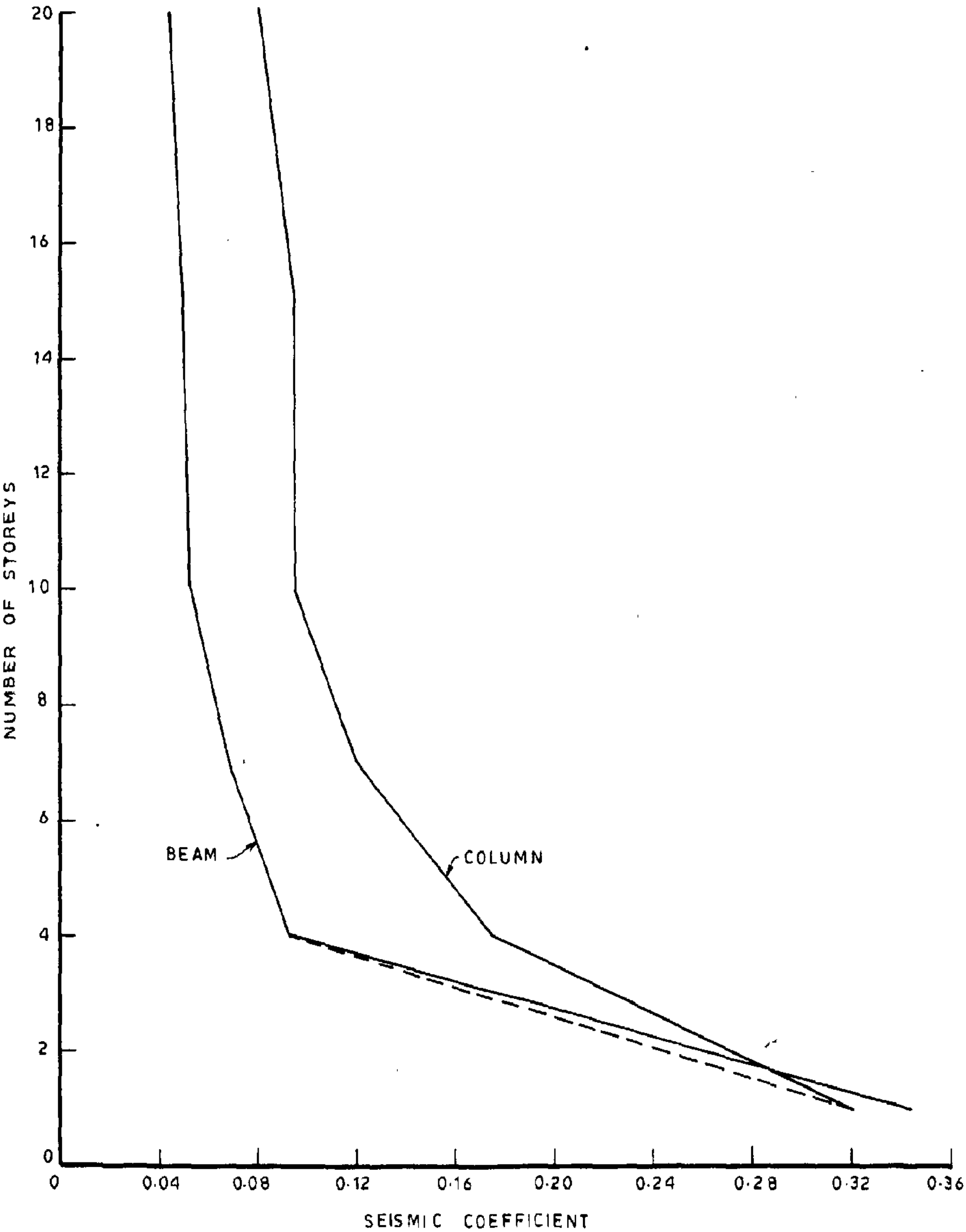


FIG. 2.4_ NUMBER OF STOREY VS SEISMIC COEFFICIENT AT YIELD

TABLE 2.3

No. of Storeys	S.C. at 33 $\frac{1}{3}$% increase in steel stress		S.C. at Yield	
	Beam	Column	Beam	Column
1	0.165	0.1575	0.344	0.32
4	0.058	0.0725	0.093	0.175
7	0.035	0.06	0.068	0.12
10	0.026	0.045	0.0525	0.095
15	0.025	0.045	0.0497	0.095
20	0.022	0.04	0.0435	0.08

From the computations shown in table 2.3 and from figures 2.1 to 2.4, it is seen that,

i) For framed buildings in R.C.C. upto 20 storeys, earthquake force does not govern the design for zone I and zone II

ii) Earthquake force does not govern the design, upto six storeys in zone III, upto five storeys in zone IV and upto three storeys in zone V

iii) Design of buildings upto three storeys is not governed by earthquake forces, anywhere in India

iv) EarthQuake of following seismic coefficient value, is withstood by the building without damage i.e. with the building remaining elastic.

TABLE 2.4

No. of storeys	EQ of S.C. Value withstood
20 or more	0.04
9 to 19	0.05
7, ~~to~~ 8	0.06
6	0.07
5 or less	0.08 or more

CHAPTER - III

SEISMIC SAFETY IN THE INELASTIC RANGE

In this chapter, maximum earthquake force which the buildings can withstand with some damage but without total collapse, is investigated using results of inelastic response analysis. Computations for exact non-linear analysis being too tedious, the concept of reduction factor[5,6,8] is used here. Since linear systems are easily analysed, a relationship is established between linear and non-linear systems so that a non-linear system can be approximated by an equivalent linear system for the purposes of analysis and design. The method adopted here is to use a reduction factor by which an earthquake data should be toned down so that a linear analysis with the modified data indicates a seismic lateral load coefficient which corresponds to that of yield point of the nonlinear system. This means the design of a non-linear system capable of certain ductility can be based on the knowledge of an equivalent system which is linearly analysed for the toned down earthquake. The factor by which the force in linear structure must be reduced to obtain the force in the non-linear structure, is defined as 'Reduction Factor'.

For the type of structure and loading chosen for this study and for the Koyna earthquake, an average value of reduction factor is taken as 2.

Thus the stress in steel in beam is taken as 5200 Kg/cm^2 assuming a linear structure. For the column the corresponding stresses in concrete are taken as 190, 250 and 310 kg/cm^2 respectively for M150, M200 and M250 concrete. The actual stress in the materials in the associated non-linear structure will be much less than the above values.

TABLE 3.1

S.C. Resisted in Inelastic Range

No. of Storeys	Beam	Column
1	1.01	1.03
4	0.26	0.42
7	0.19	0.34
10	0.14	0.26
15	0.14	0.25
20	0.12	0.22

The graphs are plotted in fig 3.1 and fig 3.2 From the computations shown in Table 3.1 and from figures 3.1 and 3.2, it is seen that:

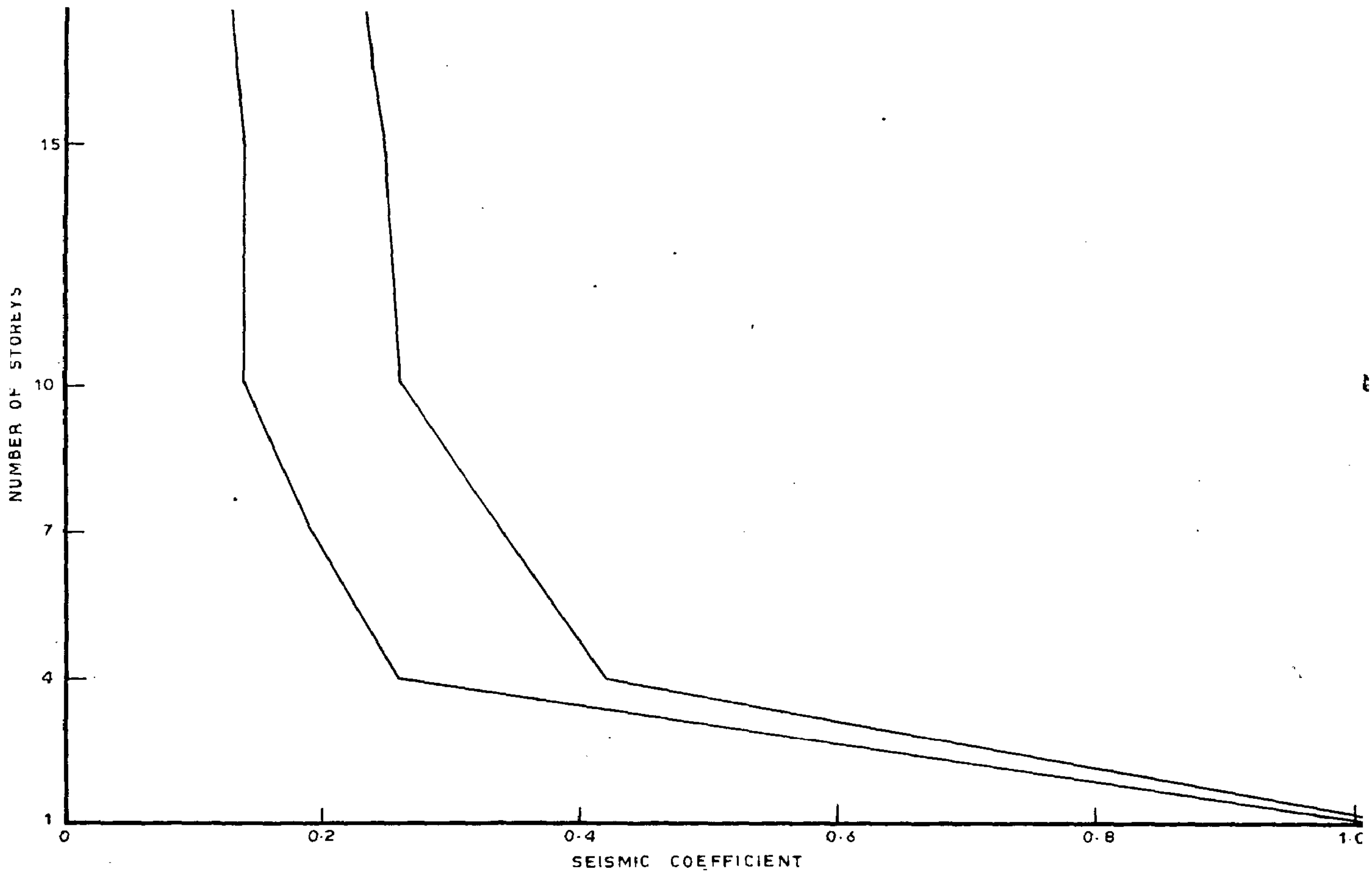


FIG. 3.1 _ NUMBER OF STOREY VS SEISMIC COEFFICIENT IN INELASTIC RANGE

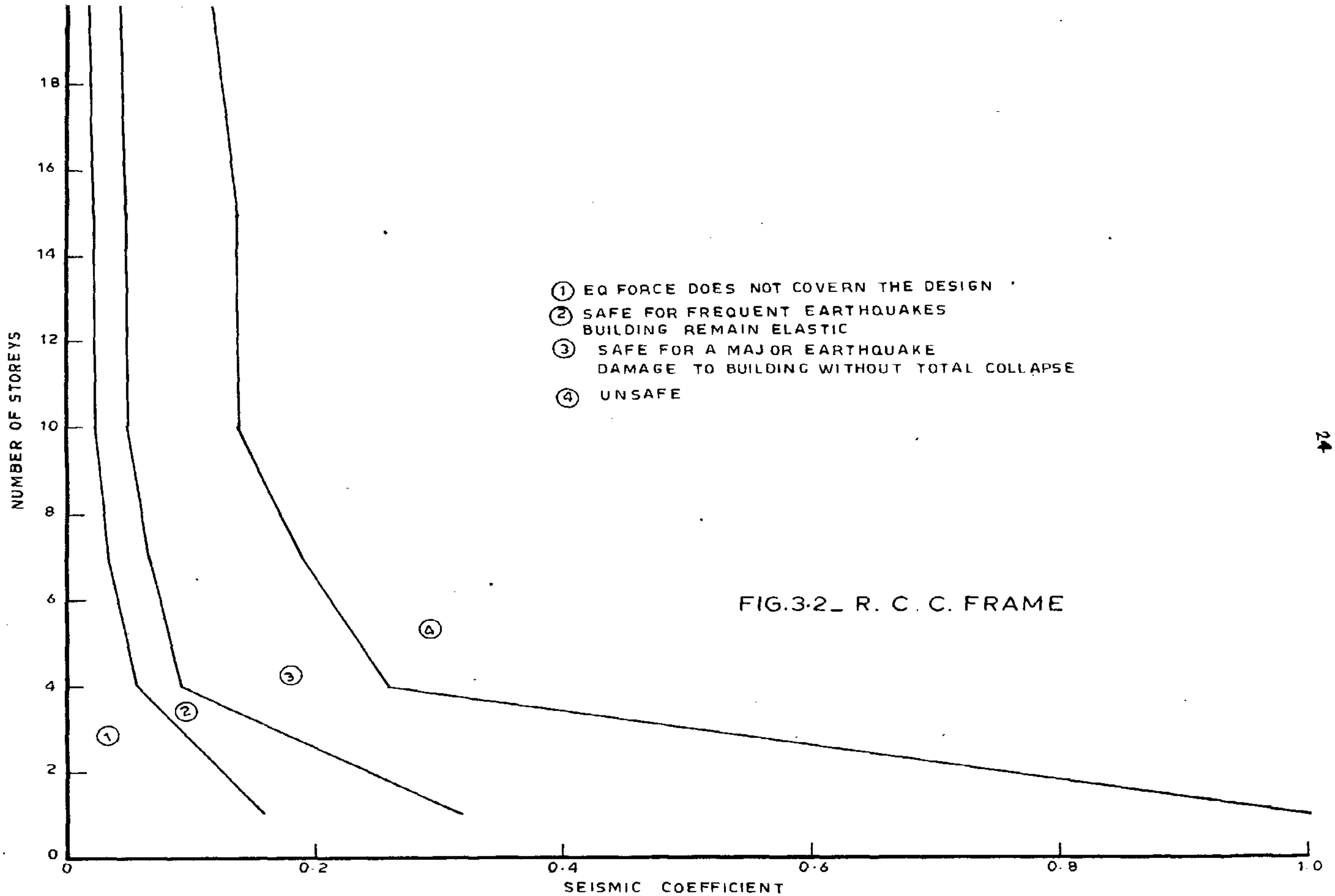


FIG.3·2_ R. C. C. FRAME

Buildings are safe for a major earthquake of seismic coefficient value given below, which may occur only once in the lifetime of the structure. The building enters the inelastic range hence there is some damage but total collapse does not take place.

TABLE 3.2

No. of storeys	S. C. withstood
15-20	0.12
10-15	0.14
7-10	0.14
4-7	0.19
1-4	0.26

CHAPTER - IV

ULTIMATE LOAD ANALYSIS

It is possible to estimate the ultimate lateral load at various storey levels for a building initially designed for vertical load and say lateral load which is within that which causes a $33\frac{1}{3}\%$ increase in allowable stresses.

The ultimate load theory[26] as applicable to R.C.C. frames is still in a developing stage. The main hurdle is that while in a steel beam or column, the plastic hinge has a large capacity of rotation, it is not so for R.C.C. beam or column, where-in the plastic hinges have limited capacity for rotation. Further, in case of R.C.C., the first formed hinges rotate more when further hinging takes place and the first hinge may fail even when other hinges are still taking rotation. The exact method of analysis under the circumstances, is essentially a procedure of trial and error requiring an accurate M-Φ diagram of R.C.C. sections. Thus, even for a frame of moderate size, staggering number of computations are required.

Perhaps because of this, IS 456 : 1964 does not cover the methods of determining the collapse mechanism of an indeterminate structure for different conditions of ultimate loads and for the given properties of the structure and its members. Therefore the present practice[16] is to find the forces in members by the elastic theory, apply load factors to find ultimate forces and then to design the reinforced concrete section by ultimate load theory. A structure designed by this method is likely to give lesser value of seismic coefficient resisted than what was obtained by the reduction factor method. However to expound the utility of ultimate load theory for computing the seismic coefficients withstood at collapse, steel frames are analysed in this chapter by "the method of combining mechanisms".

For the computations shown in table 4.1, member sections, dimensions and loadings are taken as calculated in Appendix I for steel sections.

The seismic coefficients are as given in table 4.1 for an increase of $33\frac{1}{3}$% in allowable stress, at yield stress and in the inelastic range respectively for steel frames.

Statical checks have been applied for all the mechanisms found below and these have been found to give safe and statically admissible B.M. diagrams. In the computations done below the total uniformly distributed

TABLE 4.1

No. of Storeys	S.C. at 33 1/3 % increase in stress		S.C. at Yield		S.C. in the in-elastic range	
	Beam	Column	Beam	Column	Beam	Column
1	0.165	0.167	0.344	0.345	1.01	0.96
2	0.12	0.105	0.25	0.23	0.73	0.63
3	0.09	0.085	0.17	0.175	0.45	0.48
4	0.058	0.075	0.09	0.155	0.26	0.44
7	0.035	0.065	0.07	0.145	0.19	0.40

load coming on the beam affects the value of P, whereas the total lumped mass of the storey level seems not to affect the value of P. However, as the total lumped mass at the storey level controls the M_p value of columns, it indirectly affects the value of P. Hence the seismic coefficient is governed by both, viz. the load coming on the beam as well as the lumped mass at the storey level.

Single Storey : When the lateral load comes, sway occurs and the portal would fail in combined mechanism. Referring to Fig 4.1

$$P \times 2.2\,\theta + \frac{1}{2} \times 18.3 \times 3.75\,\theta = 6\,M_{p\theta}$$

$$\therefore 3.2\,P + 34.3 = 98.4$$

$$P = 20t$$

$$\therefore \text{Seismic coefficient} = \frac{20}{28.44} = 0.7$$

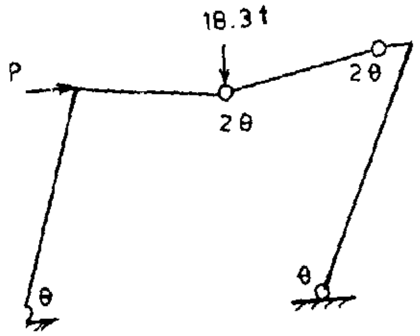


FIG. 4.1

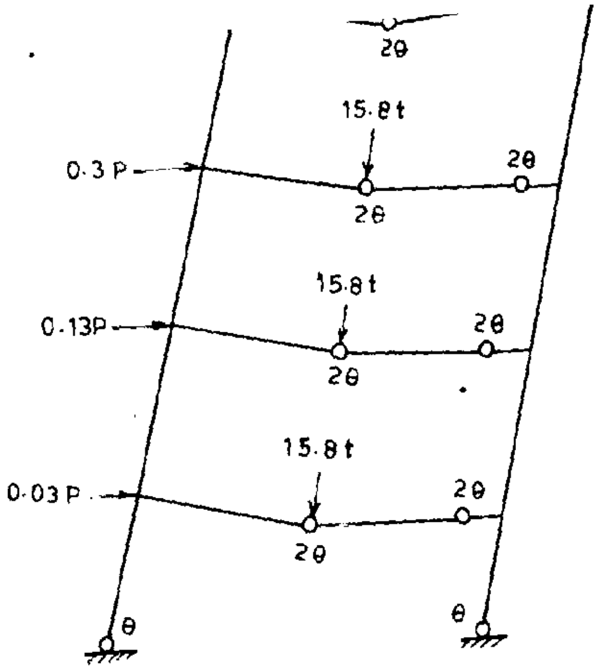


FIG. 4.4

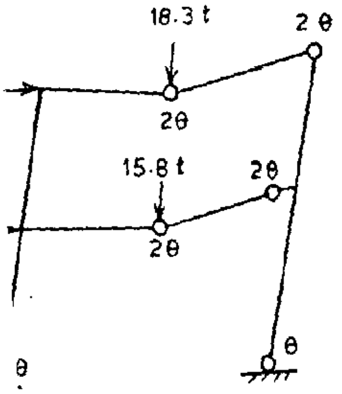


FIG. 4.2

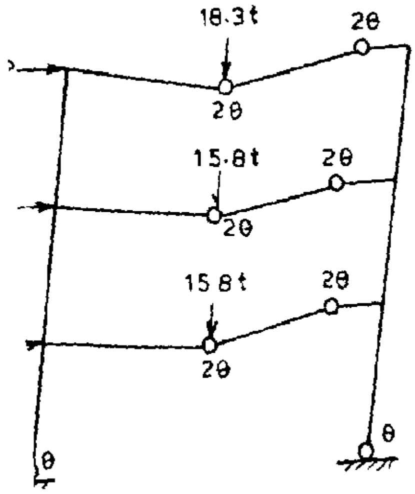


FIG. 4.3

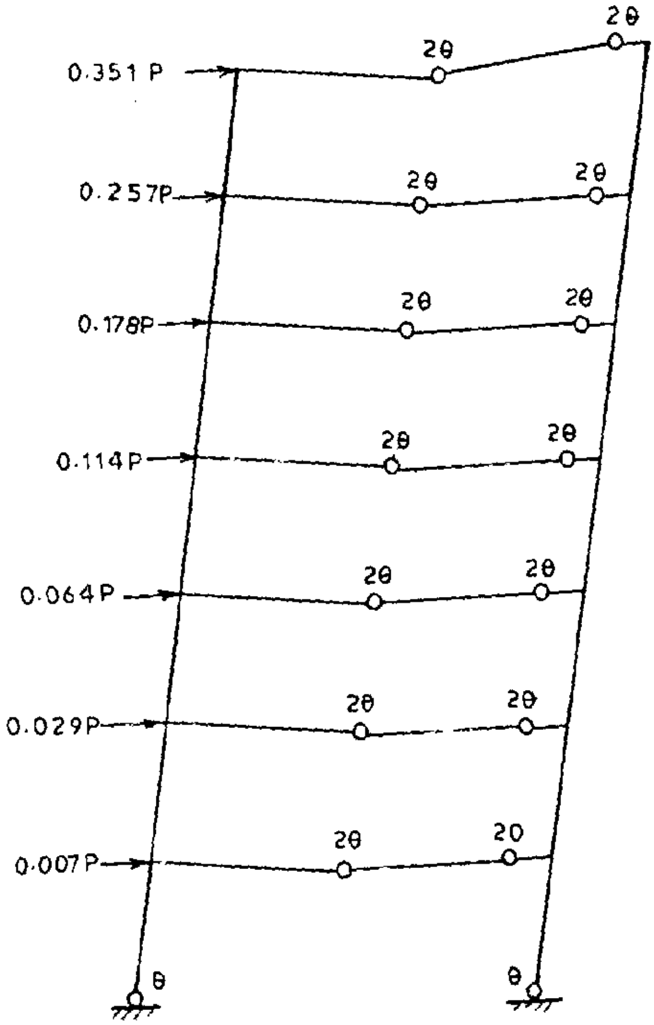


FIG. 4.5

COLLAPSE MECHANISMS

<u>Two Storey Portal</u> :

Referring to Fig 4.2

Let $V_B = P$

Parabolic distribution gives lateral load at second storey as $0.8\,P$ and at first storey as $0.2\,P$

$$0.8P \times 6.40 + 0.2P \times 3.20 + \frac{1}{2} \times 18.3 \times 3.75\theta + \frac{1}{2} \times 15.8 \times 3.75\theta$$

$$= 10.4\,M_{p\theta}$$

$$\therefore P = 18.6\ t$$

$$V_B = C\,\alpha_h\,\beta w$$

$$\therefore 18.6 = 0.85 \times \alpha_h \times 1 \times 57.06$$

$$\therefore \alpha_h = \frac{18.6}{0.85 \times 57.06} = 0.385$$

<u>Three Storeyed Portal</u>

Referring to Fig. 4.3

$$14.6\,M_{p\theta} = 18.3 \times \frac{1}{2} \times 3.75\theta + 2 \times 16.5 \times \frac{1}{2} \times 3.75\theta$$

$$+ 0.64P \times 9.60 + 0.29P \times 6.40 + 0.07P \times 3.20$$

$$\therefore P = 17.8\ t$$

$$17.8 = 0.75 \times \alpha_h \times 85.68$$

$$\therefore \alpha_h = 0.28$$

Four Storeyed Portal

Referring to Fig 4.4

$$19.2\, M_{p\theta} = 34.3\theta + 3 \times 29.65\,\theta + 0.54P \times 12.8\theta$$

$$+ 0.30P \times 9.6\theta + 0.03P \times 3.2\theta + 0.13P \times 6.4\theta$$

$$\therefore P = 17.85\ t$$

$$\therefore 17.85 = 0.7 \times \alpha_h \times 114.3$$

$$\therefore \alpha_h = 0.223$$

Seven Storeyed Portal

Referring to Fig 4.5

$$32\, M_{p\theta} = 34.3\,\theta + 6 \times 29.65\,\theta + 0.351P \times 22.4\theta$$

$$+ 0.257P \times 19.2\theta + 0.178P \times 16\,\theta + 0.114P$$

$$\times 12.8\theta + 0.064P \times 9.6\theta + 0.029P \times 6.4\theta$$

$$+ 0.007P \times 3.2\,\theta$$

$$\therefore 32\, M_p = 17.94\, P + 212.2$$

$$\therefore P = 17.45\ t$$

$$\therefore 17.45 = 0.564 \times \alpha_h \times 200.16$$

$$\therefore \alpha_h = 0.155$$

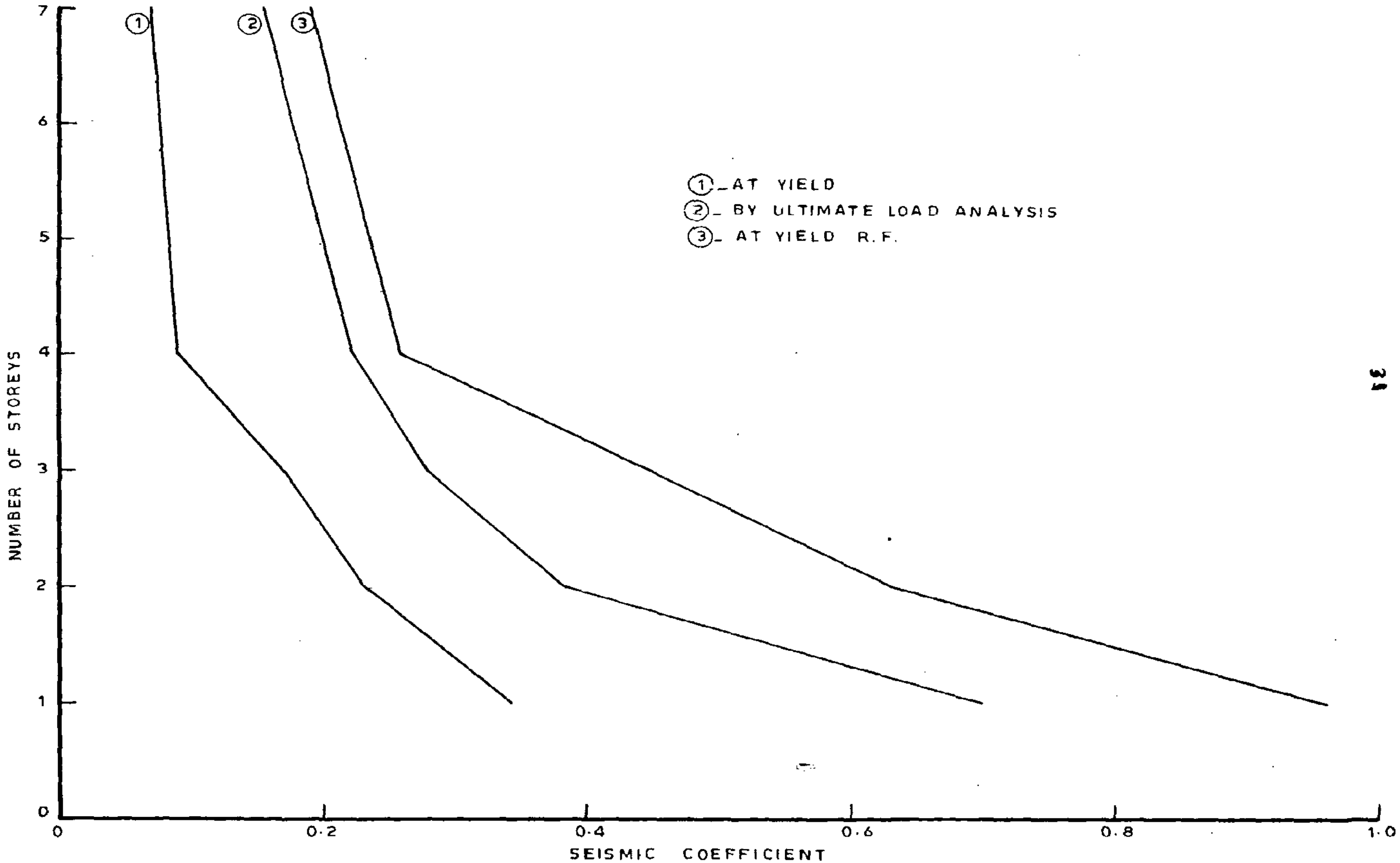


FIG.4·6_ STEEL FRAME

It is seen from fig 4.6, that the results obtained by ultimate load analysis are different from those obtained by the reduction factor method. In fact, the reduction factor method which does not assume total collapse, gives higher seismic coefficients than those obtained by ultimate load theory which assumes total collapse.

Therefore the actual seismic coefficient obtainable in ultimate stage must be higher than computed by the conventional theory. The reason for this is that the ultimate load theory assumes a gradual application of load, resulting in the formation of plastic hinges one after the other. Whereas the earthquake is a sudden reversible motion occuring for a very short duration. Further the earthquake force changes as soon as hinging action starts. Thus the actual earthquake load resisted by the frame may be higher than that computed by ultimate load theory.

CHAPTER - V

CRITICAL REVIEW OF CODE PROVISIONS

Structures designed elastically for a relatively smaller lateral load are able to withstand moderate earthquake shocks without much damage. During strong ground motions, the structure undergoes non-linear deformations dissipating good deal of energy. This results in reduction of forces in the members of a framed structure during earthquake shocks.

Linear analysis of structures is presently used for the earthquake resistant design. The code recommends seismic coefficients for elastic analysis of structures. A linear response analysis corresponding to recorded earthquakes, ~~analysis~~, indicates that the structures would be subjected to lateral forces much higher than those obtained from the design seismic coefficients given in the code. Recent investigation and experience from past earthquakes reveal that the linear analysis does not in most cases, represent the actual behaviour of the structure and, in fact, overestimates the seismic response. It is found that structures designed for small seismic forces have withstood major earthquake shocks without much damage.

Before the present code came into existence, the general practice was to construct reinforced concrete structures in the highest seismic zone, using an arbitrarily chosen value of seismic coefficient, generally about 0.1. Because such structures have escaped damage in past earthquakes, perhaps our original codal recommendations went by this experience.

However few R.C.C. structures were constructed in the highest seismic zone of Assam, at that time compared to the frequency of occurence of major earthquakes, the period over which these structures were exposed to earthquake is quite short. Therefore, there is a strong need to put the design seismic coefficients on a more rational basis.

About half of India's total area of 3.3 million sq. km is in seismic zones[3]. This is just about 11 times the seismic area of Japan and California put together. The seismic coefficients provided for in the seismic zones of Japan are however greater than those provided for in India and the values of the design seismic coefficients are determined within the range given below.

TABLE 5.1

Zone	Seismic Coefficient
I	0.096 to 0.240
II	0.108 to 0.270
III	0.120 to 0.300

The above coefficients obtained for a particular area, is modified depending upon ground condition and the type of construction of the building[1], by multiplying it with a factor α given in the Table 5.2.

TABLE 5.2

Kind of ground \ Type of construction	Steel frame	R.C.C. frame
I Rock	0.6	0.8
II Sandy Gravel	0.8	0.9
III Sand, Clay	1.0	1.0
IV Bad and Soft Clay	1.0	1.0

Thus the lowest seismic coefficient provided for in Japan is higher than the highest seismic coefficient given in our code, though the allowable unit stress in materials as well as the permissible increase in working stress when earthquake forces govern the design, in Japan are almost the same as those provided for in India.

In spite of the high value of design seismic coefficients, buildings have suffered damage in past earthquakes even in Japan. This is because the actual ground acceleration during an earthquake is very high, resulting in greater structural response during earthquake shocks.

The Koyna accelerogram is one of the strongest ever recorded from the point of view of peak ground acceleration. For the purpose of design of structures to withstand future earthquakes in this area, it is important to know the damage to which they were subjected due to ground motion rather than the details of the ground motion. The table No. 5.3[25] gives a correlation of Ground acceleration and structural damage based on a survey carried out after Koyna earthquake.

The peak ground acceleration recorded on Koyna accelerogram was 0.63 % g.

TABLE 5.3

Estimated Ground acceleration	Description of Structural Damage (R.C.C.)
0.2 to 0.3 g	Plaster cracks in 3 storey buildings
0.3 to 0.4 g	Hair cracks in a 2-storey building. Small extent of cracks at corners of some other buildings
0.4 to 0.5 g	Minor to Major cracking of cement plaster
0.5 to 0.6 g	Minor cracks
More than 0.6 g	Cracks indicative of severe shaking

The following observations were made about the behaviour of structures in Koyna earthquake[11].

1. Concrete Slab in front verandah of a school building withstood the earthquake while the brick walls suffered damage.
2. There was no continuity of reinforcement in a collapsed beam of an R.C.C. building in Koynanagar.
3. A reinforced cement concrete portal frame assembly hall (16.7 x 6 sq.m) attached with Maruti temple, Koynanagar, did not suffer any damage.
4. The hoist tower at right end of the Spillway of Koyna

Dam, was a framed R.C.C. structure with ring beams at floor levels. Even though the structure did not have aseismic provisions in its design, it withstood the violent shaking well due to its inherent strength because of monolithic action coupled with ductility.

5. New Koynanagar bridge which a reinforced concrete beam and slab type bridge, did not suffer any damage. R.C. slab bridge resting on masonry pillars over stilling basin also had no damage.

6. There was a complete collapse of a semi-circular masonry culvert on Karad-Chiplun road but half the portion of the culvert was of slabtype and did not suffer damage.

These instances show that though the actual earthquake forces were much higher than the design earthquake, yet most of the buildings withstood them with minor or no damage.

This also confirms the findings of the present study where it has been shown that R.C.C. and steel building frames can withstand fairly high seismic coefficients tho' originally they are designed only for vertical loads.

Earthquake zoning is largely a matter of prob and statistics. According to IS 1893:1970 §06 : These limits of intensity (the intensity referred here is the value of seismic coefficients provided) have been recommended for the purpose of design but these limits need not necessarily be always the highest intensity that would occur anywhere within the zone. It is possible in some cases that earthquake of much high intensity may occur at any particular spot which is unpredictable. The probabilities, however, are that a structure designed on the assumption that intensity indicated for each zone is about the maximum that is likely to occur, would ensure a reasonable amount of safety.

In the present study (in chapter II, III and IV), it has been established that fairly high seismic coefficients can be withstood by buildings, at the yield, in the inelastic range and at ultimate load respectively. Compared to these, the design seismic coefficients as given in the code are very small. However, as and when a major earthquake occurs, upgrading of zones becomes necessary.

CHAPTER - VI

CONCLUSIONS, RECOMMENDATIONS AND FUTURE SCOPE OF THE STUDY

1. CONCLUSIONS

i) For framed building in R.C.C. or steel upto 20 storeys, earthquake force may not be considered for design for Zone I and Zone II.

ii) Earthquake force need not be considered for design, upto six storeys in Zone III, upto five storeys in Zone IV and upto 3 storeys in Zone V.

iii) R.C.C. and steel buildings upto three storeys need not be designed for earthquake.

iv) Frequent earthquake of seismic coefficient value given in Table 2.4 can be withstood by the buildings without damage, with the buildings remaining elastic

v) Buildings are safe for a major earthquake of seismic coefficient value given in Table 3.1 which may occur only once in the life span of the structure. The buildings will enter the inelastic range, hence there will be damage, but total collapse will not occur.

vi) Ultimate load analysis gives result lower than that obtained by the reduction factor method.

vii) Behaviour of R.C.C. buildings in the Koyna earthquake confirm the finding of the present study that R.C.C.

frames originally designed only for vertical loads, can sustain fairly high seismic coefficients.

2. RECOMMENDATIONS

The design seismic coefficients given in the code should be increased and put on a more rational basis. Though increased seismic coefficients would result in increase of cost of building, nevertheless, the percentage increase in cost of building will not be very high. For short buildings, the design will be governed by dead and live loads. Also beyond a certain height the design may be governed by wind forces and the seismic force may not affect the design. For buildings of intermediate heights, the increase in cost will be approximately as follows.

TABLE 6.1

S.C.	% increase in cost	
	R.C.C.[28]	Steel[15]
0.5	2 %	-
0.10	7 %	3 %
0.15	12 %	8 %
0.20	20 %	14 %
0.25	27 %	19 %

In the above computations, of course, the consideration of wind forces is taken into account.

3. FUTURE SCOPE OF THE STUDY

The study was restricted to seismic coefficient method. The findings can be checked by modal analysis based on the concept of average spectra or by detailed dynamic analysis based on actual spectra computed from expected ground motion. Non-linear analysis may be employed for the above purpose. Further the study can be extended to timber, brick and similar other construction.

REFERENCES

1. Architectural Institute of Japan, Edited by, "Design Essentials in EarthQuake Resistant Building", Architectural Institute of Japan, Tokyo and Elsevier Publishing Co., Amsterdam, 1970.

2. Arya, A.S. and Ajmani, J.L., "Design of Steel Structures", N.C.B., Roorkee, U.P., 1965.

3. Arya, A.S. and Brijesh Chandra", Can we Build Earthquake proof Structures", Science Today, May, 1974.

5. Berg, G.V., Das, Y.C., Gokhale, K.V.G.K., and Setlur, A.V., "The Koyna, India, EarthQuakes", Proceedings of the Fourth World Conference on Earthquake Engineering, Vol. III, Santiago De Chile, 1969.

5. Bertero, V. and Bresler, B., "Seismic Behaviour of Reinforced Concrete Framed Structures", Proceedings of the Fourth World Conference on Earthquake Engineering, Vol. I, Santiago De Chile, 1969.

6. Blume, I.A., Newmark, N.M. and Corning, L.M., "Design of Multi-storey Reinforced Concrete Building for Earthquake Motions", Portland Cement Association, 1961.

Boguslavsky
7. ~~[illegible]~~, B.W., "Design of Reinforced Concrete", The Macmillan Company, New York, 1956.

8. Chandra, B., "Study of Inelastic Response of Multistorey Frames During Earthquakes", Ph.D. Thesis, Deptt. of Civil Engg., University of Roorkee, Roorkee, 1971.

9. Chandra, R. and Narayanan, R., "Reinforced Concrete Design Handbook, working Stress Method", S.E.R.C., Roorkee, C.S.I.R., Today and Tommorrow's Book Agency, New Delhi, 5. 1967.

10. Chandrasekaran, A.R., and Saini, S.S., "Dynamic Response of Non-linear Systems", Journal of the I.E. (India) Vol XLVIII No.3, Pt CI2, Nov. 67 (Special).

11. Chandrasekaran, A.R. and Saini, S.S., "Spectral Analysis of Koyna Earthquake of December 11, 1967", Earthquake Engineering Studies, Feb'68, S.R.T.E.E., U.O.R., Roorkee, U.P.

12. Chandrasekaran, A.R., Srivastava, L.S. and Arya, A.S., "Behaviour of Structures in Koyna Earthquake of December 11, 1967", Earthquake Engineering Studies, Feb.'68, S.R.T.E.E., U.O.R., Roorkee, U.P.

13. Desai, J.P., "Design of a framed Building at Delhi", M.E. Special problem, Jan'74, S.R.T.E.E., U.O.R., Roorkee, U.P.

14. Desai, J.P., "Lateral Load Analysis of Multistorey Buildings", M.E. Seminar, Sept. '73, S.R.T.E.E., U.O.R., Roorkee, U.P.

15. Fenves, S.J. and Newmark, N.M., "Seismic forces and overturning moments in Building, Towers and Chimneys", Proceedings of the Fourth World Conference on Earthquake Engineering, Vol. III, Santiago De Chile, 1969.

16. Gadi, S.N., "Vibration Characteristics of a multistorey frame and the effects on its cost due to earthquake allowance", M.E. Thesis 1960 Civil Engg. Deptt., U.O.R., Roorkee, U.P.

17. Ghanekar, V.K., Chandra, R. and Sarkar, S., "Handbook for Ultimate Strength Design of Reinforced Concrete Members", S.E.R.C., Roorkee.

18. Goel, S.K., "Computer-aided Processing of Structural Design Specifications", Ph.D. Thesis, Graduate College of the University of Illinois, Urbana, Illinois, 1970.

19. Goyal, B.K., and Sharma, S.P., "Computer Analysis of Multistorey Frames", I.C.J., Oct.'64.

20. I.S. 456:1964, "Code of Practice for Plain and Reinforced Concrete", (Second Revision), I.S.I., New Delhi.

21. I.S. 800:1962, "Use of Structural Steel in General Building Construction", I.S.I., New Delhi.

22. I.S. 875:1964, "Code of Practice for Structural Safety of Buildings : Loading Standards" (Revised), I.S.I., New Delhi.

23. I.S. 1893:1962, "Recommendations for Earthquake Resistant Design of Structures", I.S.I., New Delhi.

24. I.S. 1893:1970, "Criteria for Earthquake Resistant Design of Structures" (Second Revision), I.S.I., New Delhi.

25. I.S. Draft, "Criteria for the Design of Multistoreyed Buildings", S.E.R.C., Roorkee, U.P., 1973.

26. Jai Krishna, Arya, A.S. and Krishen Kumar, "Distribution of the Maximum Intensity of Force in the Koyna Earthquake of December 11, 1967", Earthquake Engineering Studies, Aug.'69, S.R.T.E.E., U.O.R., Roorkee, U.P.

27. Jai Krishna and Jain, O.P., "Plain and Reinforced Concrete", Vol. II, N.C.B. Roorkee, 1968.

28. Japan Society of Civil Engineering, Compiled by, "Earthquake Resistant Design for Civil Engineering Structures, Earth Structures and Foundations in Japan", 1968.

29. Mukherjee, D., "Earthquake Effects on Multistoreyed Concrete Frames", M.E. Thesis, 1961, C.E. Deptt., U.O.R., Roorkee, U.P.

30. Neal, B.G., "The Plastic Methods of Structural Analysis", Chapman and Hall Ltd. and Science Paper books, 1965.

31. Newmark, N.M., "Torsion in Symmetrical Buildings", Proc. of the Fourth World Conf. on Earthquake Engg., Santiago, Chile, Vol. II, pp. 29, 1969.

32. Nigam, N.C. and Housner, G.W., "Elastic and Inelastic Response of Framed Sea Structures During Earthquakes", Proceedings of the Fourth World Fonf. on Earthquake Engineering, Vol. II, Santiago De Chile, 1969.

Appendix -II

ANALYSIS AND DESIGN OF A SEVEN-STOREYED BUILDING

The framed building chosen for illustration has the following data:

i) Ground Floor Plan (Fig A.1.1)

ii) Number of Storeys above ground level = 7

iii) Height of each storey = 3.2 m

iv) Superimposed dead loads :

a) Longitudinal external walls : 20 cm thick B.W.

b) Internal long walls : 10 cm thick B.W.

c) Internal partition walls : 10 cm thick light weight concrete at rate 500 kg/cm²

d) Roof Finish = 340 Kg/m² Floor finish = 130 kg/m²

v) Parapet at roof level = 20 cm thick 75 cm high

vi) Concrete mix : Beam and Slab M150

Columns M 250, M200, M150 as necessary

From the preliminary design[19,24] the following sizes of members will be found sufficient.

Thickness of slab = 15 cm

Longitudinal beams = 20 x 35 cm

Transverse beams = 40 x 60 cm

Columns External = 40 x 50 cm

Internal = 45 x 45 cm

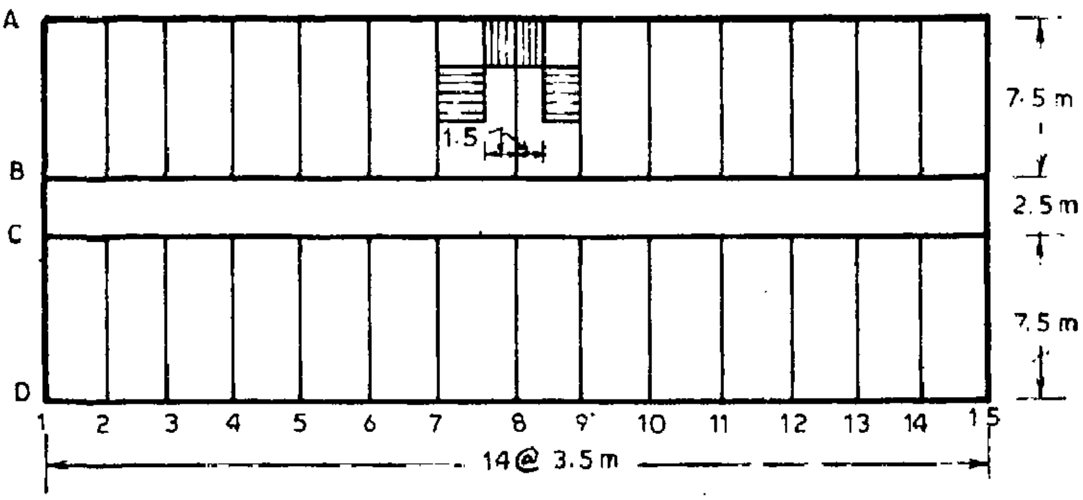


FIG. A.1.1_GROUND FLOOR PLAN

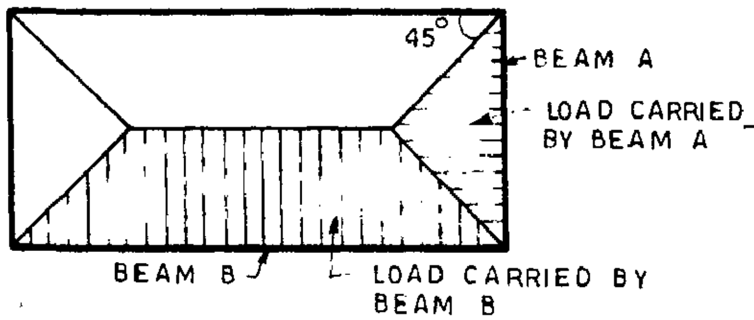


FIG. A.1.2_ LOAD DISTRIBUTION ON BEAMS

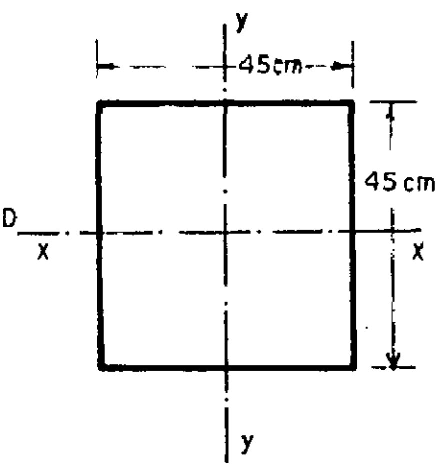


FIG. A.1.6 _ COLUMN SECTION

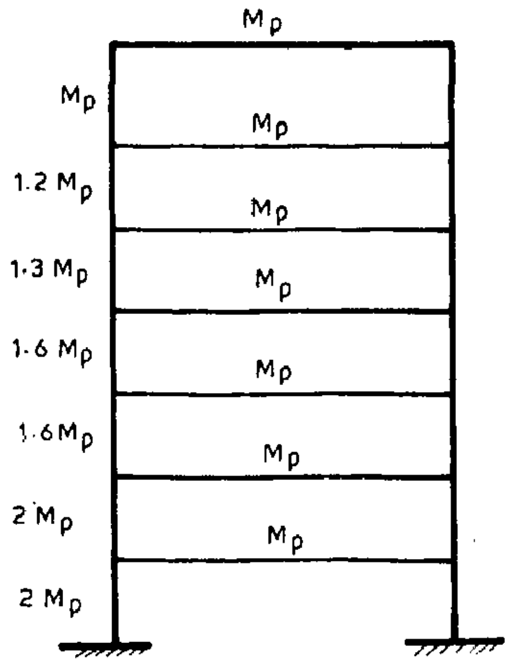


FIG. A.1.7_ STEEL FRAME

Loading on Floors

On terrace floor, the dead load will be 700 kg/sqm and live load[21] will be 150 Kg/sqm. On typical floors, the dead load will be 490 Kg/sqm. and live load will be 250 Kg/sq. m.

Loads on beams from slab panels (Kg/m)

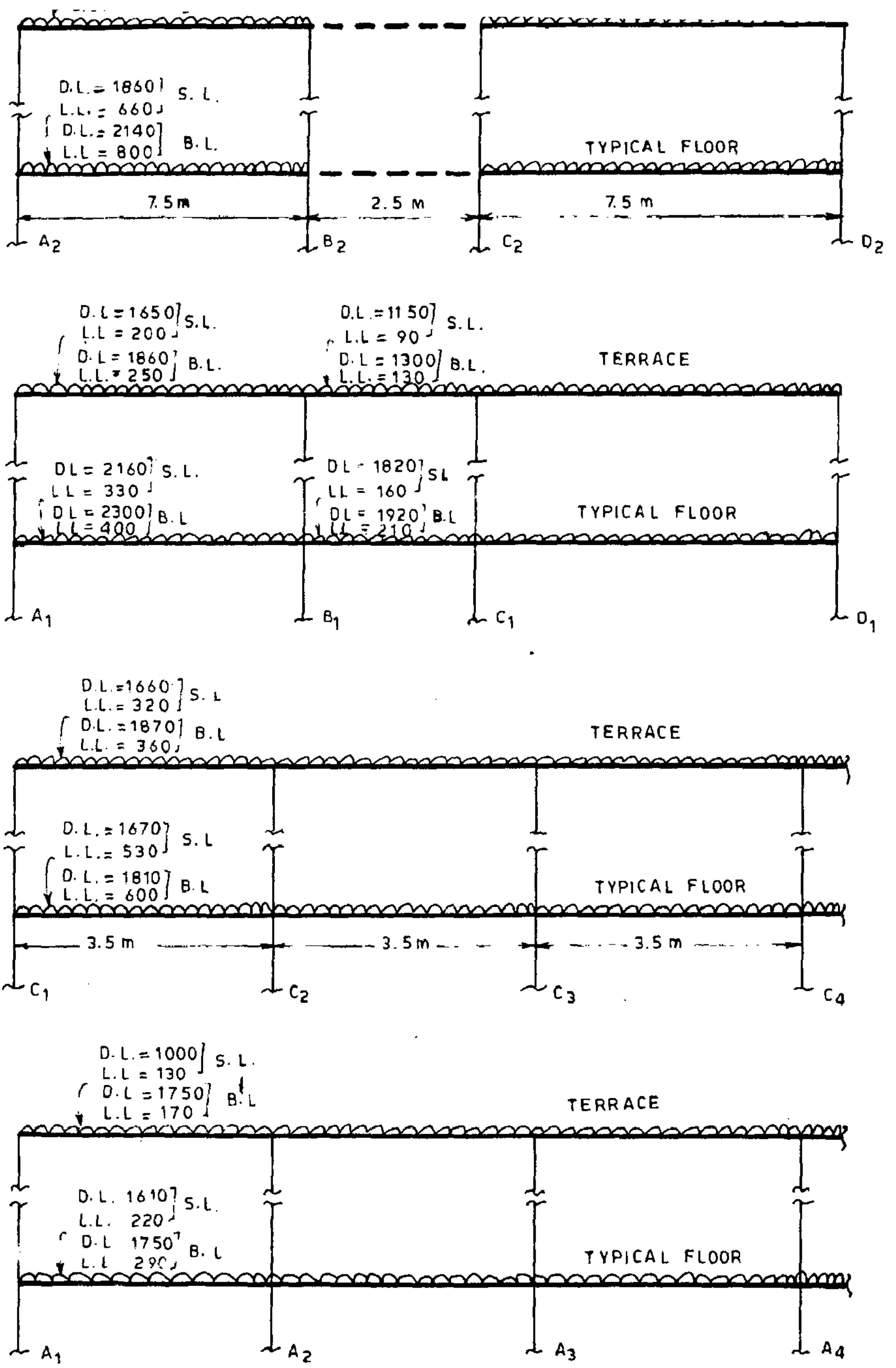
The load distribution is assumed as in Fig A. 1. 2. where $m = \frac{Ly}{Lx}$ and w = u. d. L. on slab in Kg/m². The loading on supporting beams (Kg/m) will work out as shown in Fig A. 1. 3.

TABLE A. 1. 1

MEMBER	Equivalent u. d. L. /m run for computing S. F. on beams and reaction on columns	Equivalent u. d. L/m run for computing B. M. in beams
For Beam A (Short Span Beam)	$\frac{w.Lx}{4}$	$\frac{w.Lx}{3}$
For Beam B (Long Span Beam)	$\left(\frac{w.Lx}{2} . Ly - \frac{w.Lx^2}{4}\right) \times \frac{1}{Ly}$	$\frac{w.Lx}{3} \times \frac{(3-m^2)}{2}$

Calculations for F. E. M. S for beams

1. All beams are loaded with u. d. L. only.
2. All typical floors (Floor 1st to 6th) have similar



S.L. _ SHEAR LOAD.
B.L. _ BENDING LOAD

FIG. A.1.3_SHEAR AND BENDING LOAD ON FRAMES loading. Therefore F.E.M.S. need be calculated for (i) Terrace floor beams and (ii) Typical floor beams. In Fig A.1.4 all moments are given in kgm.

Analysis of Frames for vertical loads

The frames shall be analysed for vertical loads by Kani's method. Kani's method[13] is a form of iterative method which solves modified form of slope deflection equations by Gauss-Seidel method of iteration.

A computer programme[18] for computer analysis of frames by Kani's method is enclosed in Appendix II. The programme can be used for analysis of frames for vertical loads, lateral loads or their combinations.

Floorwise loading on columns is as follows for four typical columns (i) Col. D_1 (ii) Col. C_1 (iii) Col. D_2 (iv) Col. C_2. REFER TABLE A.1.3

To allow for various combination of live loads with the dead load, in order to produce maximum and minimum moments in beams and columns, the two cycle method[7] of moment distribution has been employed.

Combinations and Design

Out of the four different types of frames analysed previously, it is seen that the frames of the type A_2-B_2

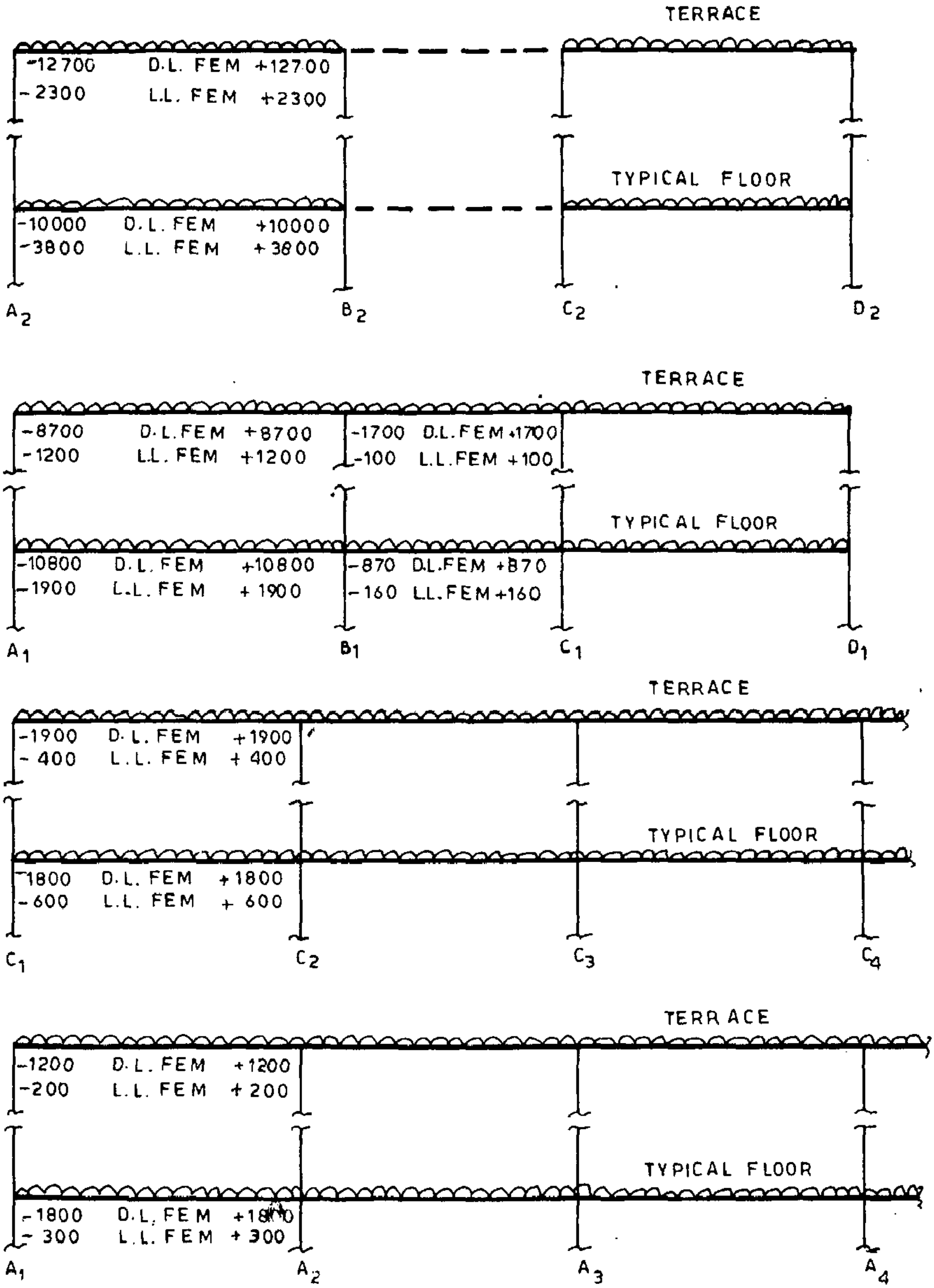


FIG. A.1.4. _ F.E.M.S FOR BEAMS FOR DIFFERENT FRAMES

TABLE A.1.2

CALCULATION OF RELATIVE STIFFNESS OF MEMBERS

Frame	Member	size cm x cm	M.I. of conc. sec. = I cm^4	Length L cm	Stiffness = I/L cm^3	Relative Stiffness
1. Frame A_1-B_1-C_1-D_1	Beam A_1-B_1	40 x 60 T-beam	116×10^4	750	16×10^2	16
	Beam B_1-C_1	40 x 60 L- beam	72×10^4	250	29×10^2	29
	Col. A_1 (Mix M150)	40 x 50 (External)	42×10^4	320	13×10^2	13
	-do- (Mix M200)	"		"	$\frac{13 \times 10^2 \times 70}{50} = 18 \times 10^2$	18
	-do- (Mix M250)	"		"	$\frac{13 \times 10^2 \times 85}{50} = 22 \times 10^2$	22
2. Frame A_2-B_2-C_2-D_2	Col. B_2 (Mix M150)	45 x 45 (Internal)	37.4×10^4	320	12×10^2	12
	Col. B_2 (Mix M200)	"		"	$\frac{12 \times 10^2 \times 70}{50} = 16 \times 10^2$	16
	Col. B_2 (Mix M250)	"		"	$\frac{12 \times 10^2 \times 85}{50} = 20 \times 10^2$	20
	Beam A_2-B_2 and C_2-D_2	40 x 60 T-beam	142×10^4	750	19×10^2	19

Col. A_2 and D_2 : Stiffnesses as in frame A_1-B_1-C_1-D_1 for Col. A_1 (External)

Table --- Contd.

Table A.1.2 (Continued)

3. Frame C_1-C_2-C_3	Beam C_1-C_2 etc.	20 x 35 T-beam	14.4×10^4	350	4×10^8	4
COL. STIFFNESSES AS IN FRAME A_1-A_2-A_3-A_4 ... for external columns AND AS IN FRAME A_2-B_2-C_2-D_2 for internal columns.						
4. Frame A_1-A_2-A_3	Beam A_1-A_2 etc.	20 x 35 L-Beam	11.3×10^4	350	3×10^8	3
	Col. A_1-A_2 etc. (Mix M150)	50 x 40 (External)	29.3×10^4	320	9×10^8	9
	-do- (Mix M200)	"		"	$\frac{9 \times 10^8 \times 70}{50} = 13 \times 10^8$	13
	-do- (Mix 250)	"		"	$\frac{9 \times 10^8 \times 85}{50} = 15 \times 10^8$	15

TABLE A.1.3

FLOORWISE LOAD ON COLUMNS (TONNES)

Floor Level	Col. D_1	Col. C_1	Col. D_2	Col C_2
Terrace	10.5	13.2	15.4	18.3
6th	24.3	29.8	32.0	36.0
5th	37.9	46.0	47.9	52.9
4th	51.1	61.6	63.2	68.9
3rd	63.9	76.8	77.9	84.0
2nd	77.4	92.8	93.7	100.7
1st	90.9	108.9	109.5	117.3

Max Moments in beams (Vertical Loads)

Frame A_2-B_2 Ist Floor level

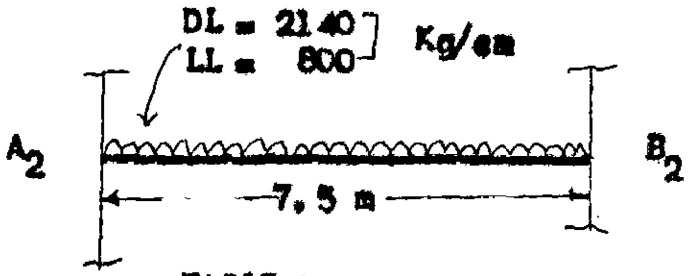
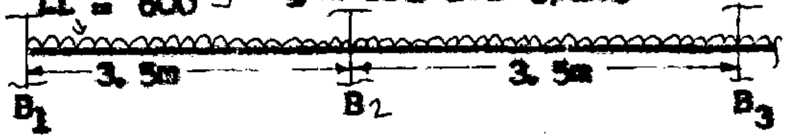


TABLE A.1.4

Joint Number	A_2		B_2
Span (m)		7.5	
Dist. Factors	0.30		0.32
Mid point Factors QL		- 0.65	
QR		+ 0.66	
1. D. L. F. E. M (Kgm)	-10,000		+10,000
2. T. L. F. E. M. (Kgm)	-13,800	+ 6900	+ 13800
3. Distribute and C.O		+ 1370	
(T.L. on span and	-2200		+ 2100
D. L. on adjacent Span)		+ 1430	
4. Add (2) and (3)	-16,000		+ 15,900
5. Distribute	+4800		- 5100
6. Add (4) and (5) (Max. B. M.)	-11200	+9700	+10800
D. L. B. M.	-8100	+7000	+7800
L. L. B. M. (by proportioning)	-3100	+2700	+3000

Max. Moments in beams (Vertical loads) **Frame B_1-B_2 ... B_{15} 1st Floor level**

DL = 1810, LL = 600 Kg/m for all spans

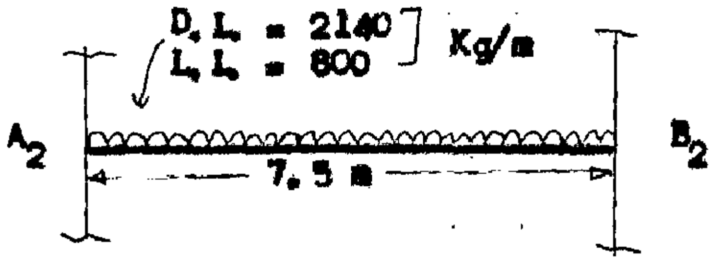


Joint Number	B_1			B_2			B_3		
Span (m)		3.5			3.5			3.5	
Distribution Factor	0.10		0.09	0.09		0.09	0.09		0.09
Midpoint Factors QL		-0.55			-0.55			-0.55	
QR		+0.55			+0.55			+0.55	
1. D.L.F.E.M. (Kgm)	-1800		+1800	-1800		+1800	-1800		+1800
2. T.L.F.E.M. (Kgm)	-2400	+1200	+2400	-2400	+1200	+2400	-2400	+1200	+2400
3. Distribute and CO		+20			+20			+20	
(T.L. on span and DL	-30		+120	-30		+30	-30		+30
on adjacent span)		+110			+20			+20	
4. Add(2) and (3)	-2430		+2520	-2430		+2430	-2430		+2430
5. Distribute	+240		-10	-10		-	-		-
6. Add(4) and(5) to get Max. B.M.	-2190	+1330	+2510	-2440	+1240	+2430	-2430	+1240	+2430

TABLE CONTD......

Joint Number	B_4			B_5			B_6	... ...
Span(m)		3.5			3.5			
Distribution Factors	0.09		0.09	0.09		0.09	0.09	... ...
Midpoint Factors QL		-0.55			-0.55			
QR		+0.55			+0.55			
1. D. L. F. E. M. (Kgm)	-1800		+1800	-1800		+1800		
2. T. L. F. E. M. (Kgm)	-2400	+1200	+2400	-2400	+1200	+2400		
3. Distribute and C.O.		+20			+20			
(T. L. on span and DL	-30		+30	-30		+30		
on adjacent span		+20			+20			
4. Add (2) and (3)	-2430		+2430	-2430		+2430		
5. Distribution								
6. Add (4) and (5) to get Max. B. M.	-2430	+1240	+2430	-2430	+1240	+2430		

Max. Column Moments (Vertical Loads)



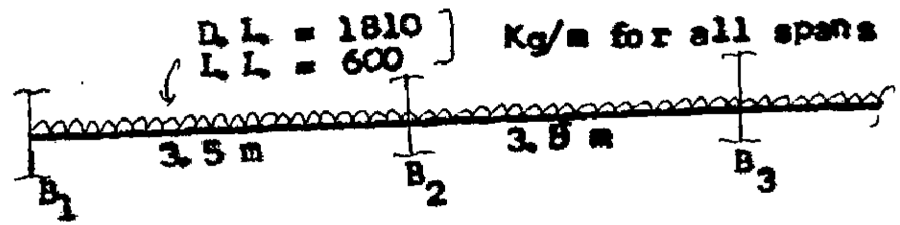
Frame A_2-B_2 at 1st Floor Level

D. F. for beams		0.30	0.32
Joint No.		A_2	B_2
Dist. Factors	Top Column	0.35	0.34
	Bottom Column	0.35	T. L. 0.34
1. T. L. or D. L. F. E. M.		-13800	+ 13800
2. Dist. and C. O.		-2200	+ 2100
3. Add (1) and (2)		-16000	+ 15900
4. Distribute	Top Col.	5600	5400
	Bottom Col.	5600	5400
(ii) N. L. L. B. M.	Top Col	1500	1500
	Bottom Col	1500	1500
(iii) 0.25 L. L. B. M.	Top Col.	400	400
	Bottom Col.	400	400
(iv) D. L. B. M.	Top Col	4100	3900
	Bottom Col.	4100	3900

Note : B. M. S. are in Kgm

MAX COL MOMENTS

Frame B_1-B_2-B_3 ... B_{15} 1st Fl level



Joint No.	B_1		B_2		B_3		B_4
Dist. Factor — Top Col.	0.45		0.41		0.41		0.41
Dist. Factor — Bot. Col	0.45	T.L	0.41	D.L	0.41	T.L.	0.41
1. T.L. of D.L. FEM	-2400	+2400	-1800	+1800	-2400	+2400	-1800
2. Dist. and C.O.	-30	+120	+30	-30	-30	+30	+30
3. Add (1) and (2)	-2430	+2520	-1770	+1770	-2430	+2430	-1770
4. Distribute — Top Col.	1000	320		270		270	
4. Distribute — Bot. Col.	1000	320		270		270	

D.L. + 0.25 L.L. B.M. Top Column (B_2) 70

Bottom Column (B_2) 70

Note : All moments are in Kgm

suffer the maximum amount of lateral load moments. Hence it is proposed to restrict the study to this frame only.

Further, out of all beams at various floor levels, the beam at second floor level carries the maximum moments under vertical and lateral loads. Hence this beam only will be examined for various combinations of loading.

Further, out of the four different types of columns analysed previously, it is seen that the column of the type B_2 carries maximum axial load, while type A_2 takes maximum moments due to vertical and lateral loads. However, as the difference for the two columns for the two values being marginal, the maximum values will be taken. This column only will be examined for the various combinations of loading.

The design of this beam A_2-B_2 at second floor level and the column A_2, B_2 from ground floor to first floor level will be based on the working stress method[9]. The design of first floor beam will be taken same as the second floor beam. This beam and the column will be designed for zone zero of IS 1893:1962 and without wind loads.

Design moments computed under various and conditions of loading are shown in table below.

Maximum moment for design will be 11.8 tm.

For the size of the beam provided singly reinforced section will be sufficient.

Condition of Loading	Beam MA	Moment (tm) Mc Centre	MB
1. Dead Load (D. L.)	-8.1	+7.0	-7.8
2. Normal Live Load (N. L. L.)	-3.1	+2.7	-3.0
3. D. L. + N. L. L.	-11.2	+9.7	-11.8

$$A_t = \frac{1180000}{1400 \times 0.865 \times 56} = 17.4 \text{ Sq. cm.}$$

Provide five bars of 22 mm dia. to give

$$A_t = 19 \text{ cm}^2$$

Curtail tensile steel as shown in fig A.1.5

Minimum compression steel $= \frac{0.35 \times 40 \times 56}{100} = 7.9$ cm²

Provide 2 bars of 22 mm dia. to give $A_t = 7.6$ cm²

which may be considered to satisfy the above requirement.

Steel at mid span $A_t = \frac{970000}{1400 \times 0.865 \times 56} = 14.3$ Sq cm

Provide 4 bars of 22 mm dia.

Check for Shear

Shear force = 9.45 t

Max. Shear Stress $= \frac{S}{b.\,jd} = \frac{9450}{40 \times 51.5} = 4.6$ Kg/cm² < 5

O. K.

∴ Shear reinforcement is not necessary.

∴ Only nominal stirrups shall be provided.

∴ Provide 8 mm dia. 2L stps at 28 cm c/c throughout.

Check for bond $S_b = \frac{S}{jd \sum O} = \frac{9450}{5 \times 51.5 \times \pi \times 2.2} = 5.3 \text{ kg/cm}^2$

< 6 O.K.

The bars shall be anchored by 94 cm into column.

Design of Column B_2 from G.F. to F.F. Level (Ref. Fig A.1.6)

Size 45 x 45 cm Mix M250

Moments and Direct Loads On the Column: These are as follows for Dead load + Normal live load (D.L. + N.L.L.)

Moment (tm)		Direct load (P)
M_{XX}	M_{YY}	(t)
5.6	0.32	117.3

M_{YY} is small, it may be neglected and design based on M_{XX} only (Uniaxial bending).

Providing equal steel on all four sides. Steel required is found as 1.75%

$$A_s = 1.75 \times 10^{-2} \times 45 \times 45 = 35.4 \text{ cm}^2$$

Provide 8 bars of 25 mm dia to give

$$A_s = 39.27 \text{ cm}^2$$

The design part is thus completed. In the Chapter II the capacity of these sections to resist lateral loads for increased stresses has been investigated.

Steel Sections[2,20]

Beams Span = 7.5 m M = 1319000 kgm $F = \frac{2710 \times 7.5}{2} = 10{,}200$ kg

F_b = 1650 kg/cm² F_s = 945 kg/cm²

Design for bending stress : Z reqd. $= \frac{1319000}{1650} = 800$ cm³

~~f_s = 10200~~ Adopt IS WB 350 at rate 56.9 kg/m

Check for Shear tw = 8 mm A_w = 0.8 x 35.0 = 28 cm²

$f_s = \frac{10200}{28} = 364$ kg/cm² < 945 Kg/cm² O.K.

Check for Declection $\Delta = \frac{WL^3}{384EI}$ $\therefore \Delta = \frac{20.4 \times (7.5 \times 100)^3}{384 \times 2110 \times 15521.7}$

= 0.68 cm

$\frac{\Delta}{L} = \frac{0.68}{7.5 \times 100} = \frac{1}{1100} < \frac{1}{325}$ $\therefore$ Safe

Adopt same section for all beams Z_p = 995.5 cm³ say 1000 cm³

$\therefore M_p$ = 1640000 Kg cm = 16.4 tm.

Columns (1) M = 5.6 tm P = 105.6t (Gr. Flr to 1st Flr)

Try ISHB 450 @ 92.5 kg/m

$$\frac{L}{r_{yy}} = \frac{320}{5.08} = 63 \quad F_a = 1113.5 \text{ kg/cm}^2$$

$$J = \frac{1}{3}\left[2\times25 \times 1.37^3 + 42.26 \times 1.13^3\right] = 63 \text{ cm}^3$$

$$C_s = \left[10,100,000 \times \frac{3045 \times 45}{1793.3\times320^2}\sqrt{1+0.162 \times \frac{63\times(320)^2}{3045\times45^2}}\right] \times 1.20$$

$$= \left[7540\sqrt{1 + 0.17}\right] \times 1.2 = 9800 \text{ Kg/cm}^2$$

$$F_b = 1522 \text{ kg/cm}^2$$

$$f_a = \frac{105600}{117.89} = 900 \text{ Kg/cm}^2$$

$$f_b = \frac{560000}{1793.3} = 312 \text{ kg/cm}^2$$

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = \frac{900}{1113.5} + \frac{312}{1522} = 0.794 + 0.205 = 0.999 < 1 \quad \text{O.K.}$$

This section gives $Z_p = 2030.9$

M_p value $= 2.04\ M_p = 2\ M_p$ Say.

(2) $P = 16.5$ t $\quad M = 7.2$ tm

Try ISHB 300 @ 63 Kg/m

$$\frac{L}{r_{yy}} = \frac{320}{5.29} = 60.4 \qquad F_a = 1128 \text{ Kg/cm}^2$$

$$J = \frac{1}{3}\Sigma bt^3 = \frac{1}{3}\left[2\times25\times1.06^3 + 27.88\times0.94^3\right] = 27.4 \text{ cm}^3$$

$$C_s = \left[10{,}100{,}000 \times \frac{2246.7 \times 30}{863.3 \times (320)^2} \sqrt{1 + 0.162 \times \frac{27.4 \times (320)^2}{2246.7 \times 1301^2}}\right] \times 1.2$$

$$= \left[7600\sqrt{1 + 0.225}\right] \times 1.2 = 10{,}000 \ \mathrm{Kg/cm^2}$$

$F_b = 1529 \ \mathrm{kg/cm^2}$

$$f_a = \frac{16500}{80.25} = 205.5 \ \mathrm{Kg/cm^2}$$

$$f_b = \frac{720000}{863.3} = 835 \ \mathrm{Kg/cm^2}$$

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = \frac{205.5}{1128.0} + \frac{835}{1529} = 0.1825 + 0541 = 0.7285 < 1 \quad \text{O.K.}$$

$Z_p = 962.2 \quad \therefore \quad M_p$ of col $= M_p$ of Beam.

The arrangement shown in Fig A.1.7 may then be taken with

$M_p = 16.4$ tm

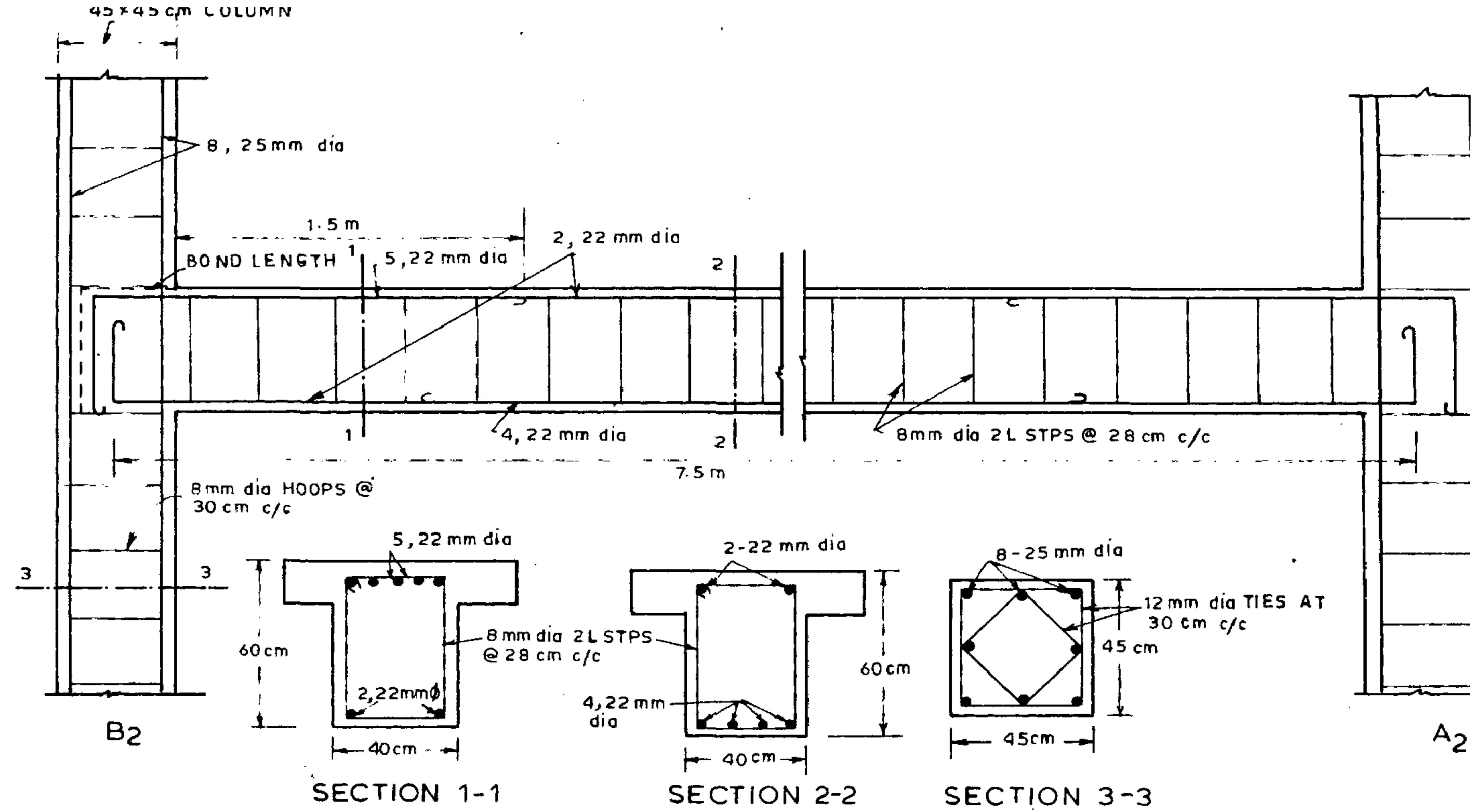


FIG.A1.5_ DETAILS OF FRAME $A_2 - B_2$ AT FIRST FLOOR LEVEL

Appendix II

COMPUTER PROGRAMME

```
C,C     FRAME ANALYSIS BY KANI,S METHOD
        DIMENSION A(1250),B(300),STM(50),SF(600),S(1250)
1000    READ 200, M,N,NC
 200    FORMAT (3I3)
        MN = M*N
        J = MN+MN+M
        READ 201,(S(I),I=1,J)
 201    FORMAT (14F5.3)
        DO 11 L=1,M
        SJ=0
        NL=N*L
        NLS=NL-N+1
        DO 10 J=NLS,NL
  10    SJ=S(J)+SJ
        SJ=-1.5/SJ
        DO 11 J=NLS,NL
  11    B(J)=SJ*S(J)
        NLS=MN+1
        J=1
        DO 24 L=1,M
        NL=N*L
        MF=NL+N
  20    J1=J+N
```

```
      L1=NLS+1
      I=MF+J
      K = NLS + NLS
      IF (L-M) 22,21,21
21    SJ = S(NLS) + S(L1) + S(J)
      SJ = -0.5/ SJ
      GO TO 23
22    SJ = S(NLS) + S(L1) + S(J) + S(J 1)
22    SJ = - .5/SJ
      A( I) = SJ*S(J I)
23    L2 = L-N
      A (L2) = SJ*S(J)
      A (K) = SJ*S(NLS)
      A(K +1) = SJ*S(L1)
      NLS = NLS +1
      J = J+1
      IF (J-NL) 20,20,24
24    NLS = NLS+1
      J = MN + LN
      READ 204,(SF( I) , I=1,J) ,(SJM( I) , I =1,M)
202   FORMAT (9F8.1)
      J = J-1
      DO 30 I =1,J,2
30    SF( I) = SF( I) + SF( I +1)
      J = 4*MN + M + M
      DO 40 I = 1,J
```

```
40    S(I) = 0
      DO 8 NK = 1,NC
      I = 1
      J1 = N + 1
      NLS = N+N
      SJ1 = STM(1)
      DO 50 J = J1, NLS
50    SJ1 = SJ1 + S(J)
      J = MN + MN+1
      L = 1
 7    SJ2 = STM (L+1)
      NL = N*L
      L1 = NL + NL + 1
      J1 = (NL + N)   2
      DO 51 K = L1,J1
51    SJ2 = SJ2 + S(K)
70    NL = N * L
      L1 = NL + N + N + I
      J1 = L1 -3*N
      K  = J + 1
      NLS = J + 2
      MF = L+I -1
      SJ = SF(MF) +S(J1)+ S(J) + S(J +3)+SJ1* B(I)
      IF (L -M) 60, 61, 61
60    L2 = I +N
      SJ = SJ + SJ2*B(L2) + S(L1)
      L1 = L1-N
```

```
      S(L1) = SJ*A(L1)
61    J1 = J1 + N
      S(J1) = SJ* A(J1)
      S(K) = SJ *A(K)
      S(NLS) = SJ *A(NLS)
      J = J +2
      I = I + 1
      IF ( I-NL) 70,70,71
71    J = J +2
      L = L +1
      SJ1 = SJ2
      IF(L-M) 7,70,8
 8    CONTINUE
      DO 81 L = 1,M
      SJ = STM (L)
      NL = N *L
      J1 = NL + NL-N
      L1 = J1 - N+1
      DO 80 I = L1, J1
      L2 = I +N
80    SJ = SJ + S(I) + S(L2)
      NLS = NL -N + 1
      DO 81 I = NLS, NL
      K =NL + I
      L1 = K-N
```

```
      SJ1 = S(L1) + S(K) + SJ*B(I)
      S(L1) = SJ1 + S(L1)
81    S(K) = SJ1 + S(K)
      I = 1
      J = MN + MN + 2
      DO 84 L =1, M
      NL = N * L *2 -1
82    K = J + I
      SJ = S(K) + S(K +1)
      S(K) = SJ + SF (I+1) + S(K)
      S(K +1) = SJ + SF (I + 2) - SF(I + 3) + S(K +1)
      I = I +2
      IF (I -NL) 82, 83, 83
83    J = J + 2
84    I = I + 2
      J = MN + MN + 2
      J1 = N + N-1
      NK = J1 + 1
      NLS = MN + MN
      K = J
      L1 = K + J1
      DO 100 I = 1, NLS, NK
      S(K) = SK(I) -SF (I +1)
      L2 = I + J1
      S (L1) = SF(L2)
```

```
      K = K + NK + 2
100   L1 =K + J1
      PUNCH 205
      PUNCH 203 (S( I), I =1,N)
205   FORMAT (19H COLUMN BASE MOMENTS)
203   FORMAT (4E16.4)
204   FORMAT(3E16.4)
206   FORMAT(14H JOINT NUMBER   , I3)
      L = 1
      DO 93 L = 1,M
      NL = N*L
      K = NL -N +1
      DO 90 I = K,NL
      J1 = NL + I
      L1 = J1 +N
      PUNCH 206, I
      IF(C -M) 91,92,92
91    PUNCH 203, S(J), S(L1), S(J+1), S(J1)
      GO TO 90
92    PUNCH 204, S(J), S(J+1), S(J1)
90    J = J +2
93    J = J +2
      GO TO 1000
      END
```