Effect of Inclusion of Soil-Structure Interaction on the Design of Multistoreyed Frames

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CERTIFICATE

This is to certify that the Thesis entitled

Effect of Inclusion of Soil-Structure Interaction

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Being Submitted by

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Bonafide Work Carried Out by him in this Department.

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Certified that the Candidate was examined by us in the

Project work Viva-voce examination held on.....

Internal Examiner

External Examiner

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ABSTRACT

This thesis work presents the collection of different theories proposed by various authors regarding the analysis of frameworks taking into account the soil-structure interaction.

A computer program for the analysis of space frames, incorporating soil-structure interaction has been developed in FORTRAN 77. Direct stiffness matrix method has been adopted for the analysis.

Four types of multibay multistorey r.c. frames have been analysed by three different approaches. In the first approach, the conventional three dimensional method is adopted. In the second approach, in addition to the three dimensional idealization, the soil-structure interaction is also taken into account. In the third approach, the so called approximate methods involving substitute frames for gravity loading and portal method for horizontal loading are adopted.

The stress resultants obtained by the three approaches are compared.

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CHAPTER I

INTRODUCTION

1.1. General:

The trend in increased construction of high-rise buildings has been caused by increase in population and limited space in the urban areas. Still research works are going on for the analysis and design of multistoreyed frames for various types of loading towards improving the rationality in the procedure of analysis and design. The inclusion of soil-structure interaction in the analysis and design of multibay-multistoreyed structure is an important step in achieving this rationality.

In the conventional design of frames, the designer calculates the moments, shearforces and axial forces in the members of the super structure, by assuming fixity of columns with foundation against all the relevant degrees of freedom. However, the foundation resting on deformable soil also undergoes deformation depending on the relative rigidities of the super structure, substructure and the soil. For a building with flexible frames, the distribution of load on the soil, will be practically unaffected by differential settlement, where as for a structure mostly consisting of walls a relatively uniform setilement will result over the plan area of the building accompanied by a significant redistribution of the load on the soil.

The foundation type and the flexibility of soil foundation system is an important criterion in the computation of response of buildings to earthquake ground motion. With reference to the past literature, it has been concluded that soil-structure interaction is a very important criterion in determining the dynamic response of a structure. It is possible that depending on the soil conditions, the foundation characteristics alone may suitably be changed to reduce earthquake force on a structure. Interactive analysis takes an important role both in static and dynamic analysis of the structure.

1.2 Significance of Interactive analysis:

Ultimate bearing pressure for continuous foundations straps and mats, is invariably determined by the settlement consideration of the supporting system. Work done by O.P.Jain et al (1) shows that, the stress resultants in a nine storeyed R.C building consisting of ten equal bays in longitudinal direction and two equal bays in the transverse direction, due to differential settlement are abnormally high unless reduced by rigidity of proper foundation system taking into account the soil-structure interaction. Only recently, the importance of soil-structure interaction on influencing differential settlements has been recognised. O.P.Jain et al had an occasion to design the foundation system for a multi-storeyed building, in which the column footings were interconnected by a diagrid of foundation beams and basement walls. It was found that the maximum total settlement and the differential settlement of soil would be of the order of 9.15 cm and 5.3 cm respectively inspite of nearly equal pressures below the footings. Change in size of footing did not help, as the compressible layers of soil were located fairly deep. Unequal settlements occured due to interaction of the loads of various footings on the deeper strata of soil. Since this will result in excessive deformation and very large moments in beams and columns of the frame, a reduction of differential settlements became necessary for an economic design as well as for proper serviceability of the structure. This was achieved by the use of stiff foundation beams connecting various columns and carrying out a soil-structure interaction analysis.

The plan of the building taken into consideration by O.P.Jain et al is shown in Fig 1.1. They followed an iterative method for interactive analysis. For this problem, three iterations were required for convergance.

The final results are summarised in Table 1.1. It is seen from the reported data that consideration of stiffness of the sub structure considerably evens out the vertical deflections. The maximum deflection of 9.15 cm under the most severely loaded column reduces to a value of 7.88 cm and the maximum differential settlement is only 3.36 cm as against the value of 5.30 cm. The most remarkable consequence of the interaction analysis is the reduction of the maximum slopes in the transverse and longitudinal directions from values of 1 in 155 and 1 in 218 to the values of 1 in 1859 and 1 in 685 respectively. This results in reducing the additional moment, due to settlement considerations.

Similarly the effect of soil-structure interaction on framed structure, resting on pile group is examined by D.N.Buragohain et al (1). They considered a frame, in which the columns are individually supported by groups of piles with rigid pile caps and connected by strap beams. The details of the frame and foundation are shown in Fig 1.2. Initially an independent analysis is carried out for the structure assuming the column bases to be fixed. In the interactive analysis, a three dimensional pile group analysis is carried out including the stiffnesses of the pile elements. In this analysis two loading cases are considered:

1. Dead load
2. Dead load with live load.

The results of the analyses are tabulated in Table 1.2. In case of vertical loads, the maximum and minimum settlements are 5.57 mm and 1.5 mm respectively with the maximum differential settlement along a panel being 2.44 mm corresponding to an angular deviation of 1 in 2880. Consequently, the internal columns are relieved of part of their axial load with the outer columns taking up more axial loads. The maximum change in axial forces is 8 percent, when compared to the independent analysis result. The lateral reactions at column bases increase by as much as 47 percent and the moments by as much as 60% because of the lateral movements of the column bases. The maximum change in super structure member end moment is around 30-35%.

Similarly incase of vertical plus lateral loads the maximum and minimum settlements are 5.57 mm and 1.36 mm respectively, with the maximum differential settlement along a panel being 2.58 mm corresponding to an angular deviation of 1 in 1900. The changes in axial reactions at column bases are similar to those under vertical loads, but the lateral reactions and moments at the base change drastically on account of the lateral loads. In the super structure, the maximum change in member end moment is around 45%.

It is realized from the works of D.K.Paul et al (1) that the building - foundation interaction plays an important role in determining the response of buildings subjected to dynamic loads. The survey done by Thornley and Albin (1) about the damage to buildings during Mexico earthquake of 1957, gives the following results:

1. Not a single building resting on concrete raft rigidly connected to concrete piles was damaged.
2. Eleven out of fifteen buildings resting on raft connected to sectional wooden piles were damaged.
3. All buildings resting on compensated raft were damaged.
4. Six out of several buildings resting on raft hung from piles by yokes and screws were damaged.
5. Most buildings resting on simple raft with no piles were damaged.

It is observed that the time period of a building increases with increasing flexibility of soil - foundation system. Thus the equivalent uniform seismic coefficients are varied. And the displacement of the base increase with increase in flexibilities of foundation in all cases.

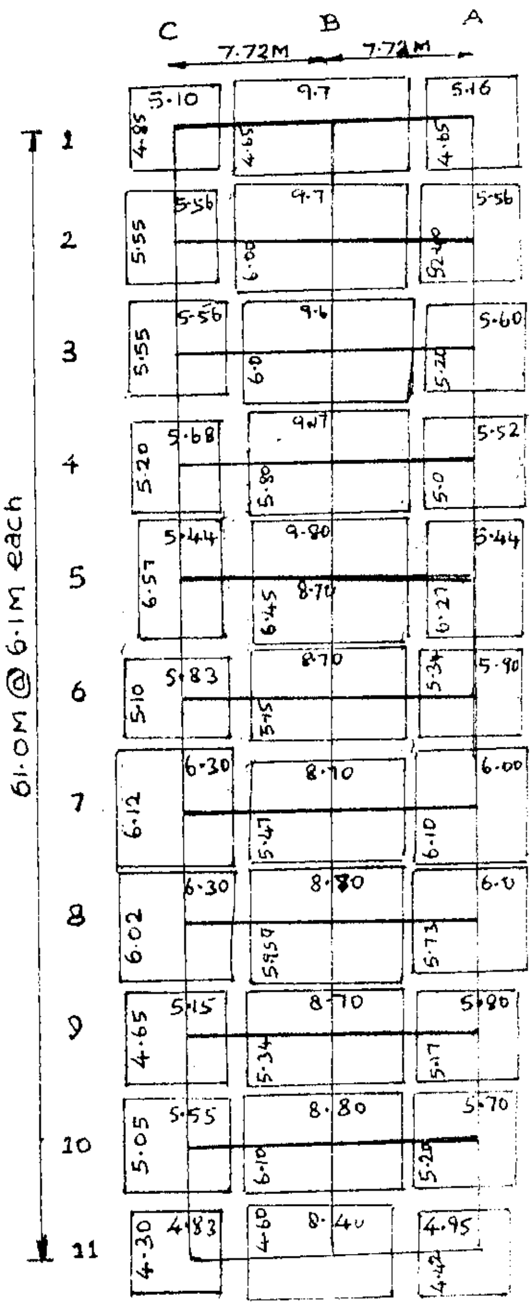
From the examples quoted above, the following conclusions are drawn:

From the consideration of economy and safety in the structural design, it is absolutely necessary to perform an interactive analysis of the foundation along with the super structure to redistribute the loads on the footings so that acceptable differential settlements are obtained, for static and dynamic load cases.

1.3 Aims of the present Investigation:

The aims of the present investigation may be briefly stated as follows.

1. To study the literature available regarding soil structure interactive analysis in the past.
2. To develop a computer program for the analysis of space frames incorporating soil - structure interaction.
3. To analyse some frames with and without soil structure interaction and by substitute frame method and to compare the three results.



ALL OUTER COLUMNS : 0.4 × 0.90

ALL INNER COLUMNS : 0.55 × 0.80

FIG 1.1 FOUNDATION PLAN WITH FOOTING AREAS

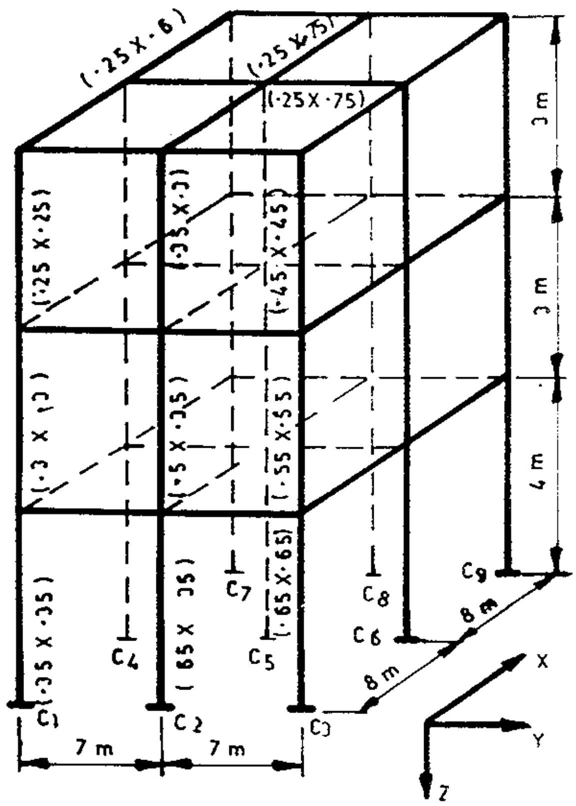


Fig1.2a SPACE FRAME FOR NUMERICAL EXAMPLE

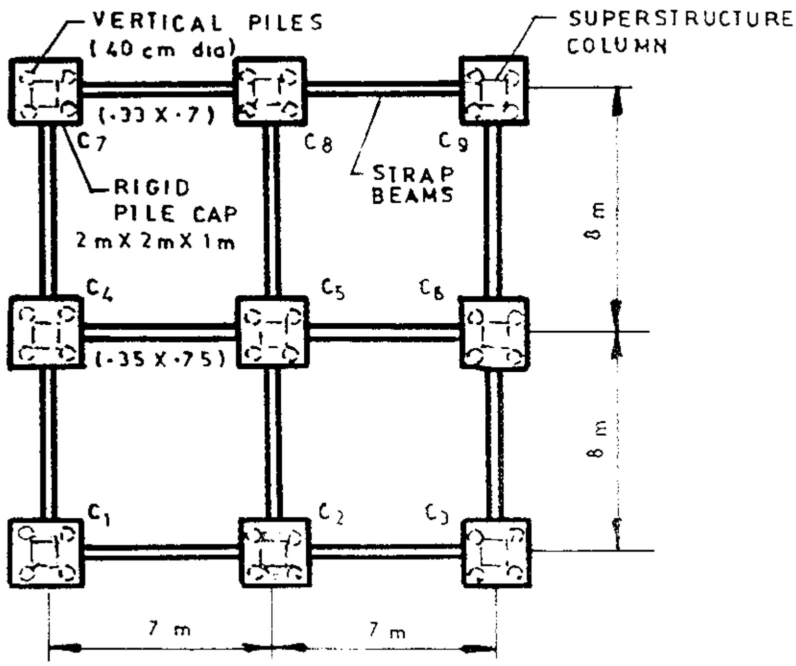


Fig1.2b DETAILS OF FOUNDATION

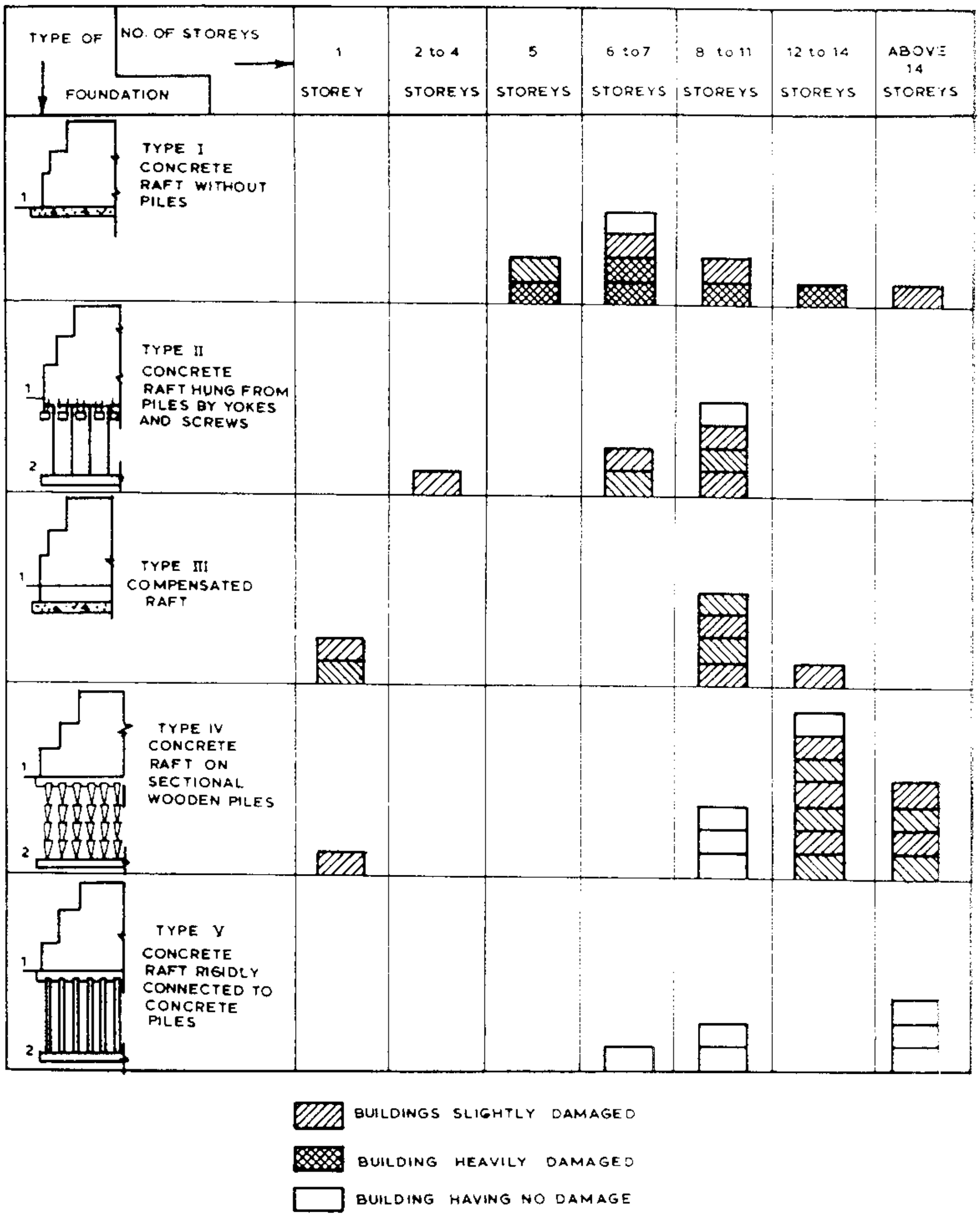


FIG. 1·3 _ DAMAGE DESTRIBUTION OF MEXICO CITY BUILDINGS ON BASIS OF FOUNDATION TYPE AND STORY HEIGHT DURING JULY 1957 MEXICO EARTHQUAKE

TABLE 1-1 : RESULTS OF ANALYSIS OF THE ILLUSTRATIVE EXAMPLE

Column	Deflections in cm.		slope in Transverse Direction x 10^5		slope in Longitudinal Direction x 10^5		Percentage change in column loads
	Initial	Final	Initial	Final	Initial	Final	
A 1	4.30	5.55	-466.0	- 11.80	+278.0	+121.90	+ 64.9
B 1	6.00	5.66	+ 12.9	- 0.59	+459.0	+110.18	- 9.7
C 1	4.25	5.56	+453.0	+ 10.43	+344.0	+131.65	+ 69.1
A 2	5.90	6.16	-634.0	- 49.35	+213.0	+110.30	+ 11.6
B 2	8.40	6.81	0.0	- 4.18	+196.0	+102.17	- 40.2
C 2	6.10	6.25	+595.0	+ 40.39	+213.0	+116.02	+ 6.7
A 3	6.40	7.02	-660.0	- 18.37	+ 32.7	+ 77.44	+ 26.1
B 3	9.00	7.18	- 12.9	- 6.31	+ 49.1	+ 84.84	- 46.2
C 3	6.50	7.12	+647.0	+ 5.90	+ 32.7	+ 77.85	+ 24.4
A 4	6.60	7.17	-595.0	- 46.84	+ 32.7	+ 46.86	+ 25.7
B 4	8.95	7.80	- 19.4	- 4.17	0.0	+ 65.83	- 30.6
C 4	6.70	7.25	+621.0	+ 41.65	+ 32.7	+ 44.65	+ 23.3
A 5	6.95	7.66	-544.0	- 14.69	+ 16.3	+ 19.98	+ 27.0
B 5	9.15	7.88	- 12.9	- 3.03	0.0	+ 49.59	- 31.0
C 5	7.00	7.77	+569.0	+ 8.94	0.0	+ 16.43	+ 25.9
A 6	6.80	7.46	-531.0	- 53.78	- 16.3	- 2.62	+ 26.8
B 6	8.90	8.28	0.0	- 1.95	+ 49.1	+ 37.58	- 16.1
C 6	6.80	7.50	+531.0	+ 49.64	- 32.7	- 6.99	+ 29.6
A 7	7.05	7.44	-466.0	- 47.13	+ 32.7	- 19.88	+ 14.3
B 7	8.90	8.15	- 12.9	- 1.69	- 65.5	- 90.27	- 21.4
C 7	7.15	7.48	+453.0	+ 45.67	+ 16.3	- 28.06	+ 11.2
A 8	6.80	7.39	-492.0	- 8.54	- 32.7	- 53.55	+ 23.0
B 8	8.85	7.45	- 12.9	+ 1.75	- 81.9	-102.23	- 37.3
C 8	6.90	7.36	+505.0	+ 12.39	- 65.5	- 66.99	+ 15.8
A 9	6.40	6.90	-518.0	+ 4.74	-114.0	- 97.92	+ 19.7
B 9	8.30	6.88	+ 25.9	+ 14.59	- 49.1	-114.11	- 42.5
C 9	5.90	6.67	+595.0	+ 23.88	-131.0	-108.11	+ 38.9
A10	5.75	5.92	-518.0	- 27.71	-196.0	-137.03	+ 7.6
B10	7.70	6.31	+ 12.9	+ 12.15	-163.0	-125.88	- 36.7
C10	5.40	5.68	+569.0	+ 50.78	-180.0	-134.25	+ 12.7
A11	4.00	5.08	-414.0	+ 1.41	-327.0	-146.02	+ 61.2
B11	5.50	5.11	+ 12.9	+ 10.70	-459.0	-131.68	- 13.3
C11	3.85	4.92	+388.0	+ 19.69	-393.0	-144.43	+ 63.3

Table 1.2: Reactions at column base of superstructure

Column	Load case	F_x in tonne: independent	F_x: interactive	F_x: difference as a percentage	M_x in tonne-metre: independent	M_x: interactive	M_x: difference as a percentage	M_y in tonne-metre: independent	M_y: interactive	M_y: difference as a percentage.
C_1	1	-24.35	-25.79	5.90	1.00	1.44	44.00	-0.74	-1.09	44.80
	2	-22.89	-24.10	5.30	-0.44	-0.04	-90.70	0.88	0.61	-30.70
C_2	1	-59.59	-59.82	0.40	0.0	0.0	0.0	-2.22	-3.64	64.40
	2	-57.89	-58.13	0.41	-2.67	-2.78	4.40	2.62	0.004	-99.80
C_3	1	-24.35	-25.79	5.90	-1.00	-1.44	44.00	-0.74	-1.09	44.80
	2	-24.37	-25.78	5.80	-2.44	-2.88	18.20	0.88	0.57	-34.60
C_4	1	-54.87	-56.65	3.64	3.07	4.95	37.90	0.0	0.0	0.0
	2	-53.18	-54.95	3.32	-0.97	1.85	-	3.00	3.17	5.60
C_5	1	-117.61	-107.90	-8.30	0.0	0.0	0.0	0.0	0.0	0.0
	2	-117.61	-107.90	-8.30	-7.58	-6.31	-16.70	9.05	7.32	-19.10
C_6	1	- 54.87	-56.65	3.64	-3.07	-4.95	37.90	0.0	0.0	0.0
	2	- 56.56	-58.34	3.20	-7.12	-8.04	12.95	3.00	3.17	5.60
C_7	1	- 24.35	-25.79	5.90	1.00	1.44	44.00	0.74	1.09	44.80
	2	- 24.33	-25.78	6.00	-0.44	-0.001	-99.70	2.37	2.53	6.90
C_8	1	- 59.59	-59.82	0.40	0.0	0.0	0.0	2.22	3.64	64.40
	2	- 61.29	-61.50	3.40	-2.67	-2.79	4.50	7.05	7.29	3.40
C_9	1	- 24.35	-25.79	5.90	-1.00	-1.44	44.00	0.74	1.09	44.80
	2	- 25.82	-27.47	6.42	-2.44	-2.93	19.80	2.36	2.76	16.80

Changes in top floor member end moments due to interaction

Beam	Percentage change in moments: Load case 1, end 1	Load case 1, end 2	Load case 2, end 1	Load case 2, end 2
C_1-C_2	20.4	- 9.6	19.9	- 8.9
C_4-C_5	33.6	-15.6	39.8	-15.4
C_1-C_4	-10.2	21.3	- 9.2	21.7
C_2-C_5	36.5	-17.0	-16.5	47.9
C_5-C_8	36.5	-17.0	28.8	-18.3
C_6-C_9	21.7	- 9.2	-11.3	21.1
C_8-C_9	- 9.6	20.4	20.3	-10.4
Column	**bottom end**	**top end**	**bottom end**	**top end**
C_1	22.1	22.2	22.7	22.7
C_2	0.0	0.0	11.3	12.4
C_4	32.1	32.3	37.6	37.6
C_5	0.0	0.0	- 7.0	- 9.5
C_6	32.1	32.3	27.7	28.1
C_8	0.0	0.0	12.4	11.5
C_9	22.1	22.2	21.8	21.6

CHAPTER II

REVIEW OF EXISTING LITERATURE

2.1 Interactive analysis for frames with Isolated footings:

2.1.1 Method proposed by O.P.Jain et al (1)

The procedure suggested by O.P.Jain et al, is an iteration method in which the frame is considered as a plane frame. The influence of the soil-structure interaction on the final displacement components in the longitudinal and transverse directions has been studied.

The initial estimate of vertical deflections and the slopes of the footings in the transverse and longitudinal directions is made for the known column loads and the settlement characteristics of the soil layers, using Boussinesq equations. With these initial displacement components known, the method uses the displacement approach to determine their revised values by considering interactionbetween the foundation and superstructure in the transverse direction. A similar interaction analysis in the longitudinal direction uses these revised deflections and original slopes as initial values and arrives at modified values. Those modified values, in turn, give initial deflections for the second iteration in the transverse direction. The procedure, thus continues so that at any stage in a given direction, the deflections from the recent iteration in the perpendicular direction are used as initial values. This method has been found to be quickly convergent and normally three iterations in both directions are sufficient. Knowing the final displacement components, stress resultants for different members are computed.

Although this method is simple, for a frame with more number of bays, the number of plane frames to be analysed will be more.

2.2 Interactive analysis for frames with Raft foundation:

2.2.1 Method proposed by G.J.W.King (1)

In practice a reinforced concrete raft will be neither rigid nor perfectly flexible. Also the super structure will have finite rigidity and the loads transmitted to the raft will depend on the displacements at the column bases. Thus the contact pressure distribution will be dependent on the super structure Raft soil interaction. The only direct way of finding the actual distribution is by performing a fully interactive analysis, considering super structure, raft and soil as a single unit.

In this method, the frames are considered as two dimensional. The beams and columns are considered as individual elements and the raft is divided up into a number of elements and in between the raft and soil, are number of friction elements.

The elements are considered to be jointed only at their corners which are referred to as nodes. The displacement at any point in an element can be related in terms of the nodal displacements, by shape functions. For this purpose the displacement function has to be assumed which approximates the variation of displacement within the element. For example translation in the x-direction can be considered as:

$$u = \alpha_1 + \alpha_2 x + \alpha_3 y + \alpha_4 xy + \alpha_5 x^2 + \ldots \text{etc}$$

With similar polynomials for the other degrees of freedom. These polynomials have to be satisfied by nodal displacements. The direct stiffness method is applied. In the two dimensional analysis, a 6x6 matrix is used for beam elements.

The friction element used is similar to the zero thickness element used by GOOD MAN, TAYLOR and BREKKE (1968) execept that allowance is made for the fact that the interface lies below the centroidal axis of the raft. Both normal and tangential displacements are considered to vary linearly along the length of the element. The Fig 2.1 gives the details of friction element. The relationship between the nodal forces and displacements, is:

$$\begin{Bmatrix} x_i \\ y_i \\ x_j \\ y_j \\ x_k \\ y_k \\ M_k \\ x_l \\ y_l \\ M_l \end{Bmatrix} = \frac{L}{6} \begin{bmatrix} 2K_s & 0 & K_s & 0 & -K_s & 0 & K_s\frac{h}{2} & -2K_s & 0 & K_s h \\ & 2K_n & 0 & K_n & 0 & -K_n & 0 & 0 & -2K_n & 0 \\ & & 2K_s & 0 & -2K_s & 0 & K_s h & -K_s & 0 & K_s\frac{h}{2} \\ & & & 2K_n & 0 & -2K_n & 0 & 0 & -K_n & 0 \\ & & & & 2K_s & 0 & -K_s h & K_s & 0 & -K_s\frac{h}{2} \\ & & & & & 2K_s & 0 & 0 & K_s & 0 \\ & & & & & & K_s\frac{h^2}{2} & K_s\frac{h}{2} & 0 & K_s\frac{h^2}{4} \\ & & \text{SYMMETRIC} & & & & & 2K_s & 0 & -K_s h \\ & & & & & & & & 2K_s & 0 \\ & & & & & & & & & K_s\frac{h^2}{2} \end{bmatrix} \times \begin{Bmatrix} u_i \\ v_i \\ u_j \\ v_j \\ u_k \\ v_k \\ \theta_k \\ u_l \\ v_l \\ \theta_l \end{Bmatrix}$$

Here M_k and M_l are the moments at the centroidal axis of the raft produced by the shear forces on its base, 'h' is the thickness of the raft, and k_n and k_s are coefficients which relate the normal and shear stresses, σ_n

and τ_s, with the relative normal and tangential displacements, Δ_n and Δ_s , so that

$$\sigma_n = K_n \cdot \Delta_n$$

and $$\tau_s = K_s \cdot \Delta_s$$

Here the value of K_n should be assigned an arbitrary large value and K_s has been assigned either a high or a low value to simulate a rough or smooth interface.

The elements used for the soil are rectangular continum elements with the displacement functions.

$$u = \alpha_1 + \alpha_2 \cdot x + \alpha_3 \cdot y + \alpha_4 xy$$

$$v = \alpha_5 + \alpha_6 x + \alpha_7 \cdot y + \alpha_8 xy$$

The stiffness matrix for this element depends on the elasticity matrix involved and is therefore evaluated during computation [Refer Fig 2.2].

The same concept can be extended for a three dimensional rafted frame.

Merits of this method:

(i) This is not an iterative method, like the method suggested previously for frame with isolated footings.

(ii) The variation in the soil strata can be accounted. Thus the results will be accurate.

Demerits:

(i) If a three dimensional interactive analysis is done, we have to consider 4 different types of elements like beam element, plate element for raft, friction element and three dimensional solid element for soil media. So, it will be very difficult for programming and preparing input data. In addition, the computer time will be more in this case.

2.2.2 Method suggested by L.Strat et al (1)

In this method, the interaction between super structure and substructure is considered. The effect of soil media is no included indirectly.

The vertical settlement at any point 'i' in the soil media, can be written as below

$$W_i = \sum_{j=1}^{m} \beta_{ij} \cdot x_j$$

Frequently a number of pressure values are considered as unknowns or as their resultants on a number of sub-domains equal to the number of displacements taken. In this way, the influence coefficients form a square matrix (β).

$$(\beta) = \begin{bmatrix} \beta_{11} & \beta_{12} & \cdots & \beta_{1n} \\ \beta_{21} & \beta_{22} & \cdots & \beta_{2n} \\ \vdots & \vdots & & \vdots \\ \beta_{n1} & \beta_{n2} & \cdots & \beta_{nn} \end{bmatrix}$$

If the ground is modelled by a field of independent springs, so that the deformations in a point depends only upon the pressure in that (Winkler hypothesis), then

$$\beta_{ij} = \frac{1}{\Omega_j \cdot K'_j} \; ; \; \beta_{ij} = 0 \quad \text{for } i \neq j$$

where K'_j = Rigidity or bed coefficient of soil

Ω_j = Area of the sobdomain.

If the soil media is assumed as a linearly deformable homogeneous and isotropic half space, the influence coefficients are determined using the following expression:

$$\beta_{ij} = \frac{1-\nu_0^2}{\pi E_0} \frac{1}{ab} \iint \frac{d\xi \cdot d\eta}{\sqrt{(x_{ij}+\xi)^2+(y_{ij}+\eta)^2}}$$

$$= \frac{1-\nu_0^2}{\pi E_0} \frac{1}{ab} \frac{\lambda_x \cdot \lambda_y}{9.0} [S]^T \cdot [\Psi_{rs}]$$

where $x_{ij} = x_j - x_i$

$y_{ij} = y_j - y_i$

λ_x and λ_y represent the division steps of the field in x and y directions (Fig 2.3).

[S] = The vector of the coefficients in Simpson's rule.

For a 25 point grid, the [S] vector is as below

$$[S]^T = \begin{bmatrix} 1 & 4 & 2 & 4 & 1 \\ 4 & 16 & 8 & 16 & 4 \\ 2 & 8 & 4 & 8 & 2 \\ 4 & 16 & 8 & 16 & 4 \\ 1 & 4 & 2 & 4 & 1 \end{bmatrix}$$

$$\Psi_{rs} = \frac{1}{\sqrt{\xi_r^2 + \eta_s^2}} \qquad \begin{array}{l} r = 1, 2, \ldots, 5 \\ s = 1, 2, \ldots, 5 \end{array}$$

The stiffness matrix of the soil (K_T) can be written as

$$(K_T) = (\beta)^{-1}$$

Now we know the stiffness matrix for all types of elements in the interaction problem. From these, we can develop the global stiffness matrix for the entire structure. And this stiffness matrix can be solved for unknown displacements and forces.

Otherwise, one can assume the structure detached from the ground, replacing its action on the structure by the equivalent forces (Fig. 2.4).

This procedure is complicated and input data preparation is also difficult.

2.2.3 Method proposed by G.J.W.King (1)

In this method, the super structure and raft are considered. Super structure is considered as a three dimensional frame, and the raft is discretised into a number of rectangular plate elements of size axb. A uniform contact pressure is assumed to act on a rectangular of size axb.

The following term f_{jk} of the flexibility matrix $[f]$ is given by the expression

$$f_{jk} = \sum_{r=1}^{n} \frac{1}{E_r} \cdot \left[\sigma_{zjkr} - 0{\cdot}5\, \sigma_{xjkr} - 0{\cdot}5\, \sigma_{yjkr}\right] \cdot H_r$$

Here n = number of layers

E_r = undrained young's modulus, in the vertical direction of r^{th} layer.

H_r = Thickness of r^{th} layer.

σ_{xjkr} , σ_{yjkr} and σ_{zjkr} represent the initial total stresses at the centre of the r^{th} layer below k due to unit load distributed over an area axb around j. These values can be obtained from these equation.

$$\sigma_{xjkr} = \int_{-\frac{b}{2}}^{b/2} \int_{-a/2}^{a/2} I'(x' - \xi)^2 \, d\xi \, d\eta$$

$$\sigma_{yjkr} = \int_{-\frac{b}{2}}^{b/2} \int_{-a/2}^{a/2} I'(y' - \eta)^2 \, d\xi \cdot d\eta$$

$$\sigma_{zjkr} = \int_{-\frac{b}{2}}^{b/2} \int_{-a/2}^{a/2} I' \cdot Z_r^2 \cdot d\xi \cdot d\eta$$

in which

$$I' = \frac{3 Z_r}{2ab} \left[(x' - \xi)^2 + (y' - \eta)^2 + Z_r^2 \right]^{-5/2}$$

Z_r = Depth of the centre of subdomain below the surface.

$x' = x_k - x_j$

$y' = y_k - y_j$

The stiffness matrix of the raft support for immediate behaviour is obtained by inverting the flexibility matrix $[\delta]$.

The flexibility term for long term behaviour (long term settlement due to consolidation of clay) is given by

$$\bar{\delta}_{jk} = \sum_{r=1}^{n} \left[(\sigma_{zjkr} - 0.5\, \sigma_{xjkr} - 0.5\, \sigma_{yjkr}) \frac{H_r}{E_r} + u_{jkr} \cdot m_{vr} \cdot H_r \right]$$

in which m_{vr} is the coefficient of volume decrease of the r^{th} layer and u_{jkr} is the pore pressure below node k at the centre of the r^{th} layer due to unit load distributed over an area axb at j. u_{jkr} can be obtained from the expression

$$u_{jkr} = \frac{\sigma_{xjkr} + \sigma_{yjkr} + \sigma_{zjkr}}{3}$$

The stiffness matrix of the raft support for long term behaviour is obtained by inverting [$\bar{\epsilon}$].

The settlements of soil at any point, below the raft can be obtained from soil mechanics principles. The settlements can be incorporated in the program and the effect of differential settlements over the structure can be studied.

In this work, they have solved an interaction problem for a rafted 4 storey frame with 5 bays in longitudinal direction and 3 bays in transverse direction. It has been shown that, in the conventional analysis the bending moments in the refts are under estimated by 14%.

Since this method, accounts for the long term behaviour of settlements and the variation of soil properties along depth, the results will be more accurate.

2.3 Interaction of frames with pile Foundations:

2.3.1 Method suggested by V.S. Chandrasekaran et a

In this method, initially an independent analysis is carried out for the structure assuming the column bases to be fixed. In the interactive analysis, a three-dimensional pile group analysis is carried out separately to evaluate the pile head stiffnesses from which the pile cap stiffnesses are computed. The foundation stiffnesses are then combined with the super structure stiffness matrix and the analysis is carried out in one step.

Pile group analysis:

For a group of arbitrarily oriented piles connected by a pile cap, a three dimensional matrix method as suggested by Bowles (2) is used. For a rigid pile cap, the pile cap joints have a constant relative position and the the axial, lateral and torsional behaviours are linearly independent to permit super position. Using the equilibrium equations and the conditions of compatability for rigid body movement of the pile cap, the forces and displacements at each pile head can be expressed interms of forces and displacements at the centriod of the pile cap. Thus,

$$P_i = A_i F_i \qquad (1)$$

where,

P_i = external force vector acting at the centroid of the pile cap.

F_i = vector of generalised forces acting at the head of the i^{th} pile.

A_i = Reaction transformation matrix for the i^{th} pile head.

Similarly

$$e_i = A_i^T x \qquad (2)$$

where e_i = vector of displacement of the i^{th} pile head.

x = vector of displacements at the centroid of the pile cap.

In addition,

$$F_i = S_i . e_i \qquad (3)$$

where S_i = pile head stiffness matrix.

Using the equations 1,2,3, one obtains

$$P_i = A_i . S_i . A_i^T . x \qquad (4)$$

Thus the total forces at the pile cap centroid can be related to the pile cap displacements as,

$$P_i = (A_i . S_i . A_i^T) x \qquad (5)$$

The assembled stiffness matrix of equation (5) is added to the corresponding column base stiffness matrix of the super structure for the complete interactive analysis in which the strap beams are included as frame members connected to the edges of the pile cap.

For obtaining the A_i matrix, three sets of coordinate systems are used - (i) A global coordinate system (x , y , z) at the centroid of the pile cap (this system can be easily related to super structure global axes x, y, x) (ii) A local cap coordinate system (x', y', z')

at each pile head, parallel to the (x_p, y_p, z_p) system, (iii) A pile local coordinate system (U_p, V_p, W_p) as shown in Fig 2.7.

U_p is along the pile axis and W_p is in the x'y' plane such that the positive x'-component of W_p is always in the positive x' direction. Thus, with l_u, m_u, n_u as as the direction cosines of U_p axis with respect to x'y'z' etc., $n_w = o$ and,

$$[A_i] = \begin{bmatrix} l_u & m_u & n_u & 0 & 0 & 0 \\ l_v & m_v & n_v & 0 & 0 & 0 \\ l_w & m_w & 0 & 0 & 0 & 0 \\ (-z\cdot l_v + y\cdot l_w) & (-z\cdot m_v + y\cdot m_w) & -z\cdot n_v & l_u & m_u & n_u \\ (z\cdot l_u - z\cdot l_w) & (z\, m_u - x\, m_w) & z\cdot n_u & l_v & m_v & n_v \\ (-y\cdot l_u + x\cdot l_v) & (-y\cdot m_u + x\cdot m_v) & (-y\cdot n_u + x\cdot n_v) & l_w & m_w & 0 \end{bmatrix}_{6\times 6}$$

in which x,y,z are the coordinates of the pile head in the (x_p, y_p, z_p) system. The direction Cosines (l,m,n) are obtained easily as in a space frame.

•

$$(F_i)^T = [\ F_u\ F_v\ F_w\ M_u\ M_v\ M_w\ ]$$

$$(P_i)^T = [\ F_x\ F_y\ F_z\ M_x\ M_y\ M_z\ ]$$

$$(e_i)^T = [\ u\ v\ w\ \theta_u\ \theta_v\ \theta_w\ ]$$

The pile head stiffness matrix (S) is derived by considering four modes of translation and rotation of the pile head, as shown in Fig 2.8.

$$S_i = \begin{bmatrix} C_5 & 0 & 0 & 0 & 0 & 0 \\ 0 & C_1 & 0 & 0 & 0 & C_2 \\ 0 & 0 & C'_1 & 0 & -C'_2 & 0 \\ 0 & 0 & 0 & C_6 & 0 & 0 \\ 0 & 0 & -C'_3 & 0 & C'_4 & 0 \\ 0 & C_3 & 0 & 0 & 0 & C_4 \end{bmatrix}$$

C_1, C_2, C_3 and C_4 are response modulii. They are obtained by considering the first two modes in the lateral pile analysis. Defining C_1 and C_3 as the force response (in V direction) and moment response (about W_p axis, respectively for unit displacement of pile head in V_p direction without rotation, and C_2 and C_4 as similar responses for unit rotation of the pile head about W_p axis without displacement, a series of forces P_{app} are applied to compute the displacement δ_t and the derived moment M_{der} for no rotation, from which the force displacement curves which are generally non-linear are obtained as in fig (2.8). Similarly, applying a series of moments M_{app} and P_{der} for no displacement, the curves of fig 2.8 are obtained. A similar set of constants C_1', C_2', C_3' and C_4' will be obtained by considering pile head displacement in the W_p direction and rotation about V_p axis.

The torsional constant (C_6) = GJ/L'

where G = shear modulus of pile head material

J = polar moment of Inertia

L' = effective pile length.

The compression constant (C_5) = $2A\,E/L$

where A = C/s area of pile

E = Young's modulus of pile material

L = Length of pile.

This stiffness matrix should be added with that of super structure and solved for unknown forces and displacements.

The following results are obtained, from the example problem

(i) In the analysis for vertical loads alone, the lateral reactions at column bases increase by as much as 47%

and the moments by as much as 60% because of the lateral movement of the column bases. The maximum change in super structure member end moments is around 30-35%.

(ii) The maximum and minimum settlements in the case of vertical plus lateral loads are 5.57 mm and 1.36 mm respectively, with the maximum differential settlement along a panel being 2.58 mm corresponding to angular deviation of 1 in 1900. In the super structure, the maxim maximum change in member and moments is around 45%.

2.3.2 Method suggested by F.Miyahara and J.G.Ergatoudis (13)

In this method, the foundation is assumed to be of the Winkler type but it offers resistance not only to normal forces but to shear and torsional forces. The stiffness matrix of the foundation element is derived by assuming the displacement function such that there is a complete compatability between beam and foundation elements.

The soil is assumed to adhere to the beam but simple non linear problems allowing separation between beam and soil when tension develops have been solved by recycling the solution.

Stiffness matrix of foundation element:

The foundation medium in the formulation presented here is assumed to be of the Winkler type, (ie) the beam rests on distributed springs as in Fig 2.9. However the beam may be embedded in the elastic medium as in the case of a pile and in addition the foundation may offer resistance to shear and torsion as well as normal reactions. The elasticity matrix (E) is given in terms of the modulus of subgrades as

$$[\bar{E}_f] = \begin{bmatrix} \bar{K}_x & o & o & o \\ o & \bar{K}_y & o & o \\ o & o & \bar{K}_s & o \\ o & o & o & \bar{K}_t \end{bmatrix}$$

$\bar{K}_x$, $\bar{K}_y$, $\bar{K}_s$ and $\bar{K}_t$ are modulus of subgrade values in x-direction, y-direction, in shear and in torsion respectively, for a beam completely buried in soil.

But we know the values of modulus of sugrades only for beam resting on the soil surface. So, these values should be modified, for a beam buried in soil. Thus, the relation is made as below

$$[\bar{E}_f] = \begin{bmatrix} \xi_x D_y K_x & 0 & 0 & 0 \\ 0 & \xi_y D_x . K_y & 0 & 0 \\ 0 & 0 & \xi_z . D_x . K_y & 0 \\ 0 & 0 & 0 & \xi_t . C . K_t \end{bmatrix}$$

Here ξ_x, ξ_y, ξ_z and ξ_t are non dimensional coefficients, which are independent of material properties, and dependent on the shape of the section. D and C are the width and perimeter of the beam cross section respectively. The modified reaction moduli (moduli of subgrade) express the reaction at a point on the beam per unit length of beam due to unit displacement at that point.

The values of for circular and square cross sections, are given in Table 2.1.

The reaction at any point along the length of the beam element if is given by $[\bar{E}_f]$. [u]. dz, in which (u) is the displacement matrix.

Therefore strain energy stored in the element is

$$U_f = \frac{1}{2}\int_0^L (u)^T \cdot (\bar{E}_f) \cdot (u)\, dz$$

The displacement at any point in the beam, can be expressed in terms of nodal displacement vector (q) by shape function (N), as below

$$(u) = (N) \ (q)$$

$$U_f = \frac{1}{2} \int_0^L (q)^T \ (N)^T \ (\bar{E}_f) \ (q) \ dz$$

From this, the stiffness matrix of the foundation element (K_f) can be written as

$$[K_f] = (N)^T \ (\bar{E}_f) \ (N) \ dz$$

By choosing proper cubic polynomials for the displacements in the x and y direction and linear polynomials for the axial displacement along the z axis and for rotation about the z axis, the stiffness matrix is derived as in Table 2.2.

This method has been found to be easier to follow and the results are satisfactory when compared with other methods.

2.4 Approximate methods of frame analysis:

The analysis of a frame can be done exactly or approximately. Since we have to compare the results of exact and approximate analysis, some problems are analysed by both methods. An approximate method used for analysing such reinforced concrete members is to use a substitute frame model, as recommended by IS 456-1978. A substitute frame consists of beams at a given floor level together with columns above and below with their far ends fixed. Usually a different combination of live load in various spans are considered, inorder to get the maximum effect in members and joints. But, in this thesis all the frames are analysed for all spans loaded condition. Substitute frame model is applicable only to gravity loads. The substitute frames have been analysed by direct stiffness method.

For the lateral load analysis, either Portal method or cantilever method can be followed. The Portal method is supposed to be satisfactory for most buildings upto 25 storeys, whereas, the cantilever method is good enough for about 35 storeys. In this thesis work, Portal method has been follwed.

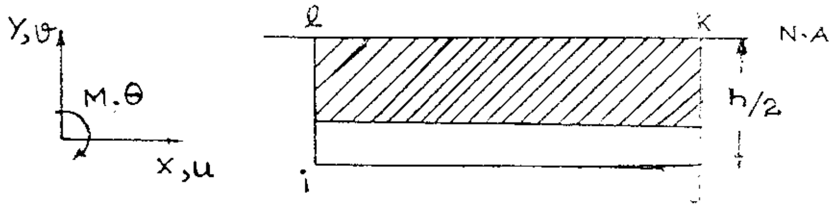
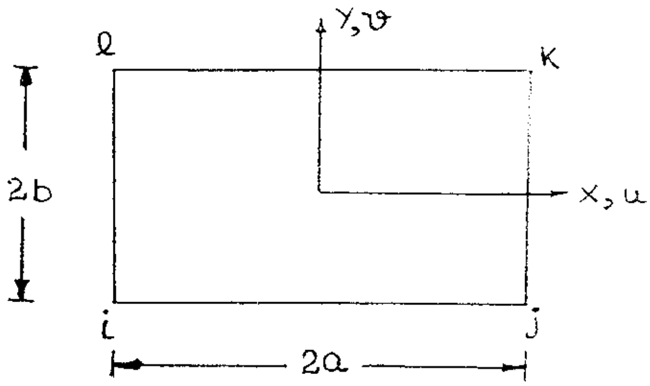
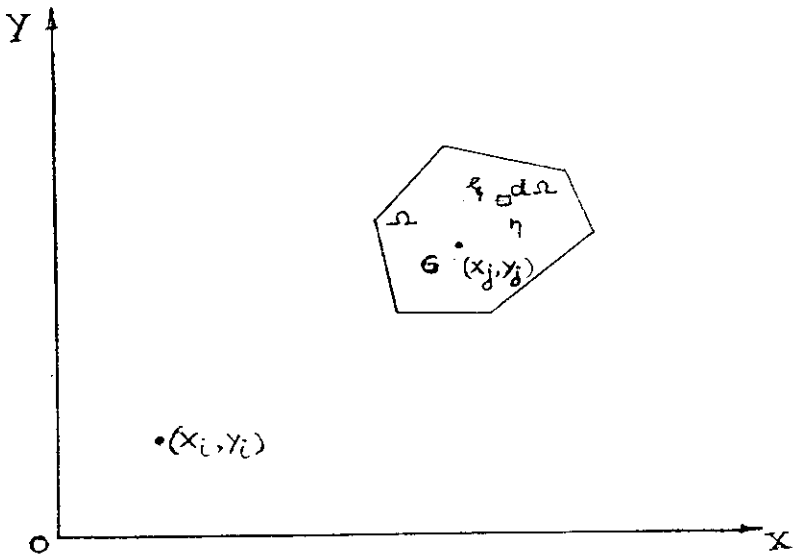


FIG 2.1 MODIFIED FRICTION ELEMENT.



$$u = \alpha_1 + \alpha_2 x + \alpha_3 y + \alpha_4 xy$$
$$v = \alpha_5 + \alpha_6 x + \alpha_7 y + \alpha_8 xy.$$

FIG 2.2 RECTANGULAR CONTINUUM ELEMENT.


Y
Ω
dΩ
η
G
(x_j, y_j)
(x_i, y_i)
O
X

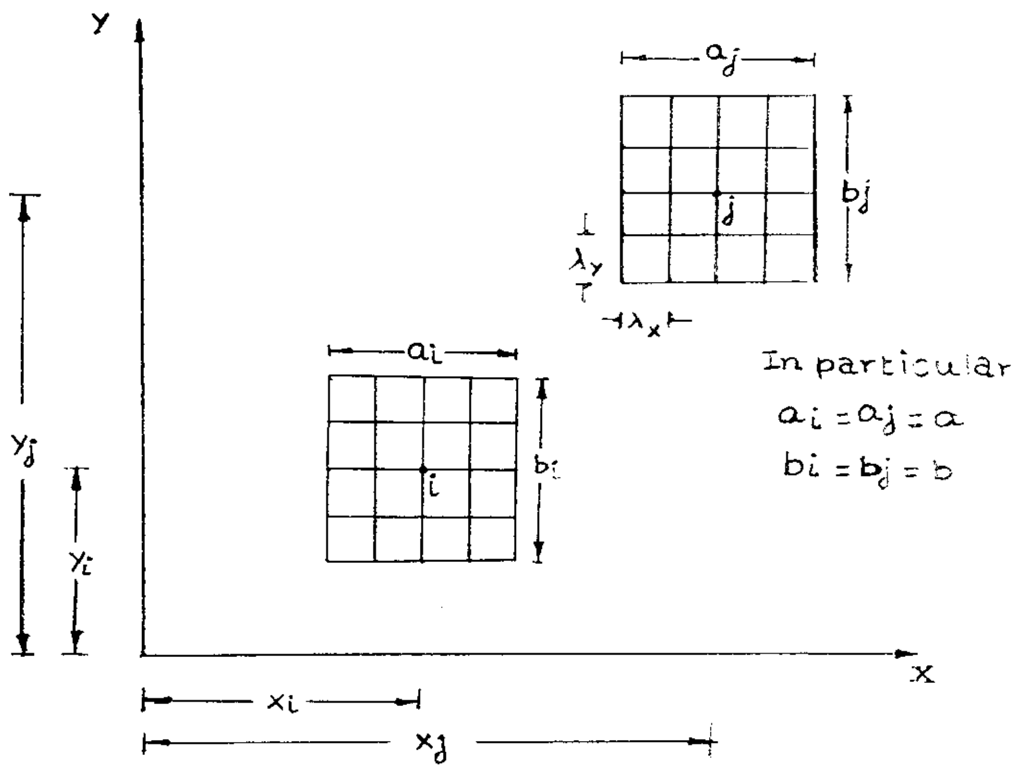

Y
a_j
b_j
j
λ_y
λ_x
In particular
$a_i = a_j = a$
$b_i = b_j = b$
a_i
b_i
i
y_j
y_i
X
x_i
x_j

FIG 2.3

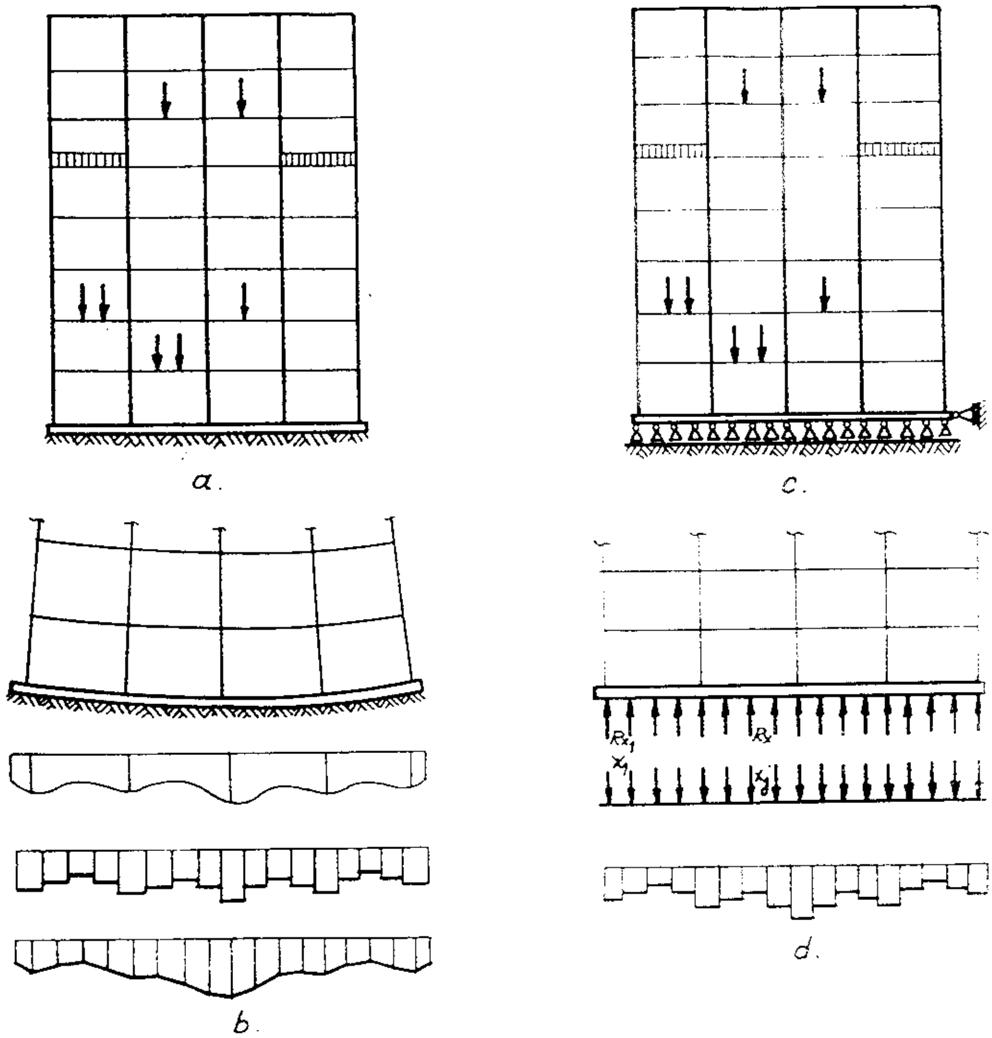
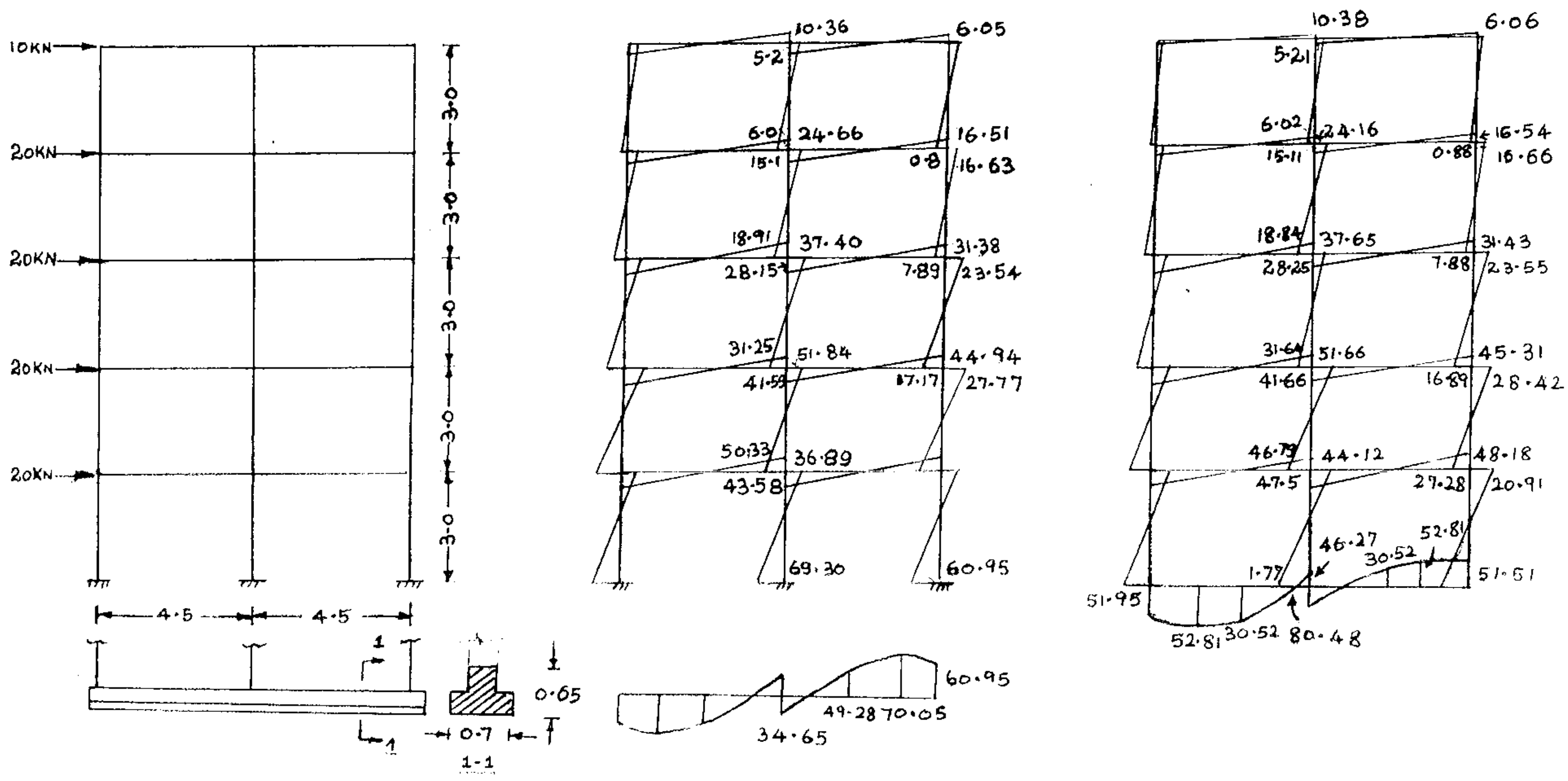


FIG 2-4 EXPLANATORY FIGURE FOR L·STRAT'S PROCEDURE.



a. ELEVATION OF FRAME   b. FRAME WITHOUT INTERACTION   c. FRAME WITH INTERACTION

FIG 2·5

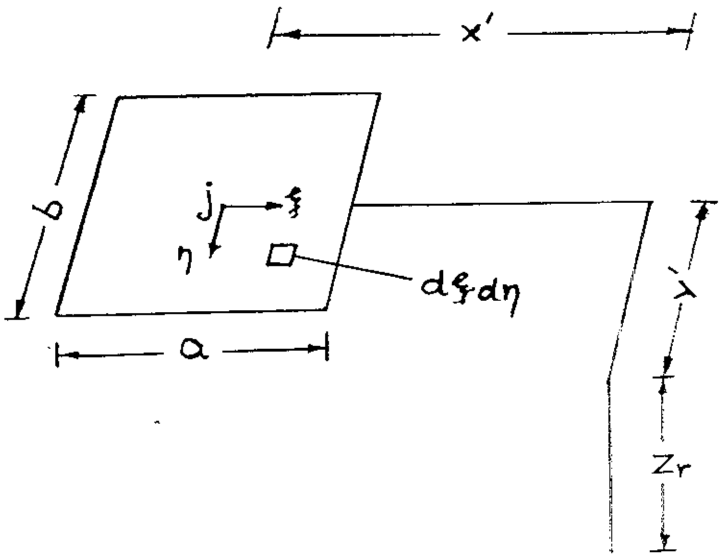


FIG 2.6 DERIVATION OF STRESSES

C/S SHAPE	ξ_x	ξ_y	ξ_z	ξ_t
CIRCLE	2.2	2.2	1.0	1.0
SQUARE	2.3	2.3	1.0	1.0

TABLE 2.1 VALUES OF ξ FOR PILE SECTIONS.

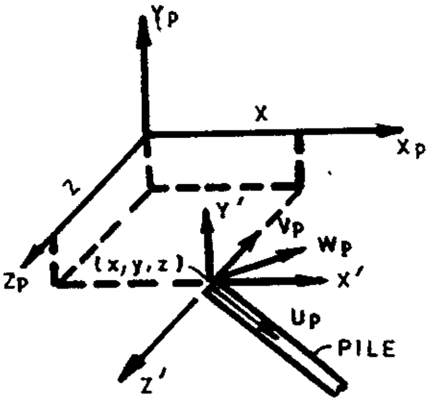


Fig.2.7 PILE GROUP COORDINATE SYSTEMS

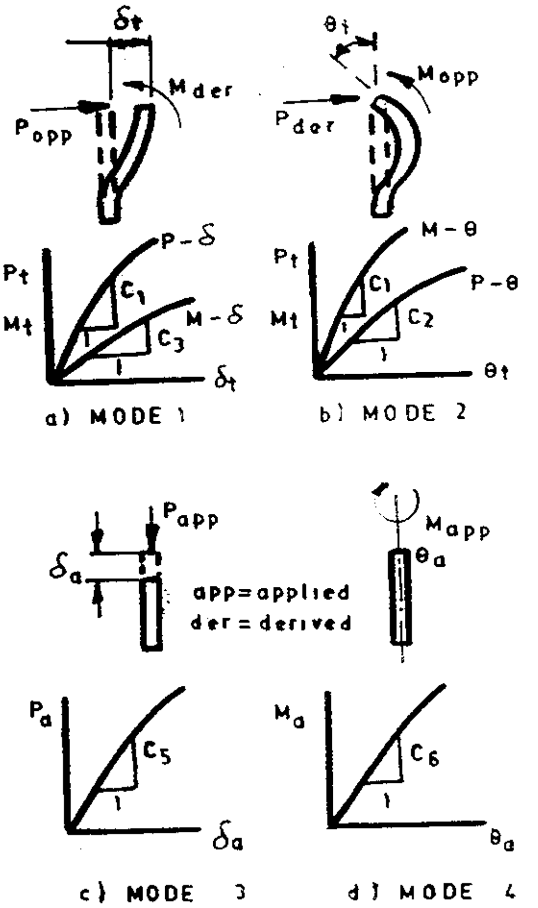
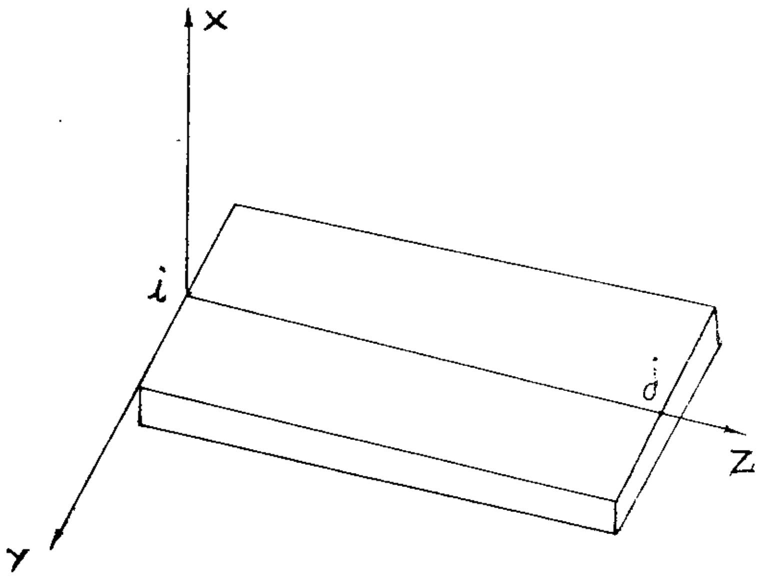
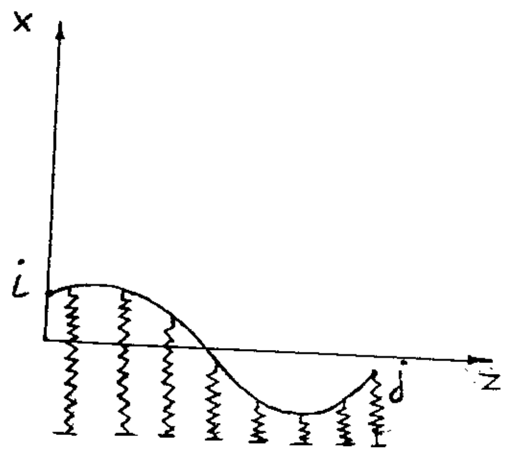


Fig.2.8 PILE RESPONSE MODULI



FOUNDATION ELEMENT.



DEFORMED SHAPE AS PER ASSUMED SHAPE FUNCTION

FIG 2·9

$[K_f] = \dfrac{L}{420} \times$ (12 × 12)

u_i	v_i	w_i	θ_{xi}	θ_{yi}	θ_{zi}	u_j	v_j	w_j	θ_{xj}	θ_{yj}	θ_{zj}
$156\bar{K}_x$											
0	$156\bar{K}_y$										
0	0	$140\bar{K}_s$						SYMMETRIC			
0	$-22\bar{K}_y \cdot L$	0	$4\bar{K}_y \cdot L^2$								
$22\bar{K}_x \cdot L$	0	0	0	$4\bar{K}_x \cdot L^2$							
0	0	0	0	0	$140\bar{K}_t$						
$54\bar{K}_x$	0	0	0	$13\bar{K}_x \cdot L$	0	$156\bar{K}_x$					
0	$54\bar{K}_y$	0	$-13\bar{K}_y \cdot L$	0	0	0	$156\bar{K}_y$				
0	0	$70\bar{K}_s$	0	0	0	0	0	$140\bar{K}_s$			
0	$13\bar{K}_y \cdot L$	0	$-3\bar{K}_y L^2$	0	0	0	$22\bar{K}_y \cdot L$	0	$4\bar{K}_y \cdot L^2$		
$-13\bar{K}_x \cdot L$	0	0	0	$-3\bar{K}_x \cdot L^2$	0	$-22\bar{K}_x \cdot L$	0	0	0	$4\bar{K}_x \cdot L^2$	
0	0	0	0	0	$70\bar{K}_t$	0	0	0	0	0	$140\bar{K}_t$

TABLE 2.2 STIFFNESS MATRIX FOR FOUNDATION ELEMENT.

CHAPTER III

THREE DIMENSIONAL ANALYSIS OF FRAMES

3.1 Explanation for the computer program:

This program analyses the space frame (with interaction) by direct stiffness method. The input data are stored in a file named SPIN.DAT. The instructions for input data preparation are given at the end of this chapter. In the main part of the program, number of nodes, MODEX value, number of materials, number of loading conditions are read. The nodal data is read by the subroutine INPUT. This subroutine is programed so that it can develop the nodal data for intermediate nodes, if the data for 1st and last node in a row, is given. Then the member sectional properties and member load data are read by the subroutine BEAM. The same subroutine reads the member number, its material group, i,j and kth node, load set number and a kode for its connection with the spring.

The member stiffness matrix and transformation matrix are developed by the subroutines STIFF and TRANS respectively. Then the transformed stiffness matrix is assembled in the global stiffness matrix by the subroutine ASSEMBL. The fixed end moments of the member loads are calculated by the subroutine FIXED. We can consider any type of member loading like point load, UDL, axial load, trapezoidal load and triangular load. The nodal loads like the wind load are read by the subroutine NODLOD.

The skyline storage method is used for storing the global stiffness matrix. The addresses of the diagonal members (matrix members) are stored in a file NCSAR. The stiffness matrix is splitted into [L] [D] [L] form, and solved by Gaussian Elimination method.

In case of columns connected with springs, (for interactive analysis) the ith node should be that point which is connected with the spring. In addition a kode 1 should be given with the element data. If the column is not connected with spring a blank for the kode is left. If two or more number of elements are connected with one spring (in case of interactive analysis for a rafted frame), it is enough to give the kode 1 for any one of the members.

3.2 Input Data :

1. Heading (20A4)

   which contains the title of the problem.

2. Problem control data (4I5)

   NSN, NMP, NLC, MODEX

   NSN = number of structure nodes

   NMP = number of materials used

   NLC = number of loading condition

MODEX = a control factor

= 0 for execution

= other than 0, for input data check.

3. Option for printing nodal and load data in output file (I5)

IPR = 0 for printing nodal and load data

= other than 0 for not printing the data.

4. Nodal data (7I5, 3F10.3, I5)

NN, (JF(I), I =1,6), X(NN), Y(NN), Z(NN), NI

in one line for each node.

NN = node number

JF(I) = 0, if node is free in the corresponding D.O.F

= 1, if that D.O.F is arrested.

X(NN), Y(NN), Z(NN) = X,Y and Z coordinates of the node

NI = nodal increment for automatic development of nodal data.

5. Material properties (I3, E10.3, 2F7.3)

I,E(I), PR(I), WD(I)

in one line for each material (NMP)

E = Young's modulus of the frame material

PR = Poisson's ratio

WD = weight density

6. Number of members, sectional properties and load sets (3I5)

NEL, NGP, NLS

NEL = number of elements

NGP = number of sectional property group

NLS = number of load sets.

7. Sectional property data (I10, 4F10.3)

I, BX(I), BIX(I), BIY(I), BIZ(I)

in one line for each type.

BX = C/S area of the member

Bix = Torsional constant

BIY = moment of intertia of the section about the local Y-axis

BIZ = moment of inertia about loacl Z-axis

8. Element load data (I3, 2(F7.3, F4.2), 3F7.3, 2(F7.3, F4.2))

I, ALIY(I), ADIY(I), ALIZ(I), ADIZ(I), ALJY(I), ALJZ(I), ALAX(I), ALTY(I), ALDY(I), ALTZ(I), ALDZ(I)

in one line for each load set.

ALIY = point load along the local Y-axis

ADIY = distance of point load (Y-Drn) from ith node

ALTZ = point load along local Z-axis

ADIZ = distance of point load (Z-Drn) from ith node

ALJY = UDL along Y-axis

ALJZ = UDL along Z-axis

ALAX = axial force

ALTY = trapezoidal force along Y-axis

ALDY = distance 'a'

ALTZ = trapezoidal force along Z-axis

9. Element data (8I4, 2E10.3)

LNUM, MG, (MNC(J), J=1,3), NG, LSN(LNUM), MODE, STIF, SET

in one line for each element

LNUM = element number

MG = material group number

MNC arry = i,j and kth node for the element

NG = sectional property group number

LSN = load set number

MODE = 0 for the members, not connected with spring at ith node.

= 1 for members connected with spring at ith node

STIF = stiffness value of the spring

SET = specified displacement at spring

10. Self weight factors (3F5.2)

SWT(I), I = 1,3

self weight factors along X,Y and Z directions respectively.

11. Element load multiplying factor (F5.2)

ELM = element load multiplying factor.

12. Number of loaded nodes (I5)

NLN = number of loaded nodes in the structure.

13. Nodal load data (I10, 6F10.4)

NODE, (CNL(I), I = 1,6)

in one line for each loaded node.

NODE = node number

CNL arry = magnitude of the load in the corresponding D.O.F

The output data is stored in a file named SPOUT.DAT

In this work, four multistoreyed frames of differnt configurations like long structure, unsymmetrical in plan, unsymmetrical in elevation and a symmetrical frame almost square in elevation are considered. The frames are analysed for a live load of 2KN/m^2 and wind load 1 KN/m^2.

In order to arrive at the size of the sections, an approximate analysis has been carried out, in which the beams are considered as fixed at ends and the section is designed for the maximum bending moment and shear force. Similarly the axial load in the columns are arrived from the beam reactions. Then the column should be designed as a plain concrete member, for the calculated axial load alone. The sectional properties are calculated by assuming the members, as plain concrete members.

The results of the frame analyses are tabulated.

```
$large
$debug
$float calls
C       SPACE FRAME ANALYSIS
        INTEGER CHT
        COMMON/DIM/N1,N2,N3,N4,N5,N6,N7,N8,N9,N10,N11,N12,N13,N14
        COMMON/PAR/IND,NSN,NMP,NEQ,NSKY,NEQ1,LCOUNT
        COMMON/TAPES/STRESS,ARRAY,IPR
        COMMON/MULT/ELMN
        COMMON/ELM/NDEL(20)
        COMMON/ELOAD/M1,M2,AA1,AA2,AA3,AA4,NSLC,NUM
        COMMON STIF,SET,MODE
        DIMENSION NLN(10),TITLE(20),ELM(10)
        DIMENSION A(100000)
        OPEN (5,FILE='SPIN.DAT',STATUS='OLD')
        OPEN(6,FILE='SPOUT.DAT',STATUS='NEW')
        OPEN(1,FILE='STRESS',STATUS='NEW')
        OPEN(2,FILE='ARRAY ',STATUS='NEW')
           OPEN (8,FILE='NDSAR',STATUS='NEW')
        OPEN(3,FILE='STIF',STATUS='NEW')
        MTOT=100000
C       STRESS=FILE STORING STRESS DISPLACEMENT MATRIX
C       ARRAY=FILE STORING ND ARRAY
C       READ AND PRINT STRUCTURE DATA
        READ(5,55) (TITLE(I),I=1,20)
        READ(5,11)NSN,NMP,NLC,MODEX
        IF(NSN.EQ.0)STOP
        READ(5,12)IPR
        WRITE(6,66)(TITLE(I),I=1,20)
        WRITE(6,22)NSN,NMP,NLC
C       READ,GENERATE AND PRINT NODAL DATA
C
        N1=1
        N2=N1+6*NSN
        N3=N2+NSN
        N4=N3+NSN
        N5=N4+NSN
        IF(N5.GT.MTOT) CALL ERROR(N5-MTOT,1)
        IF(N5.GT.MTOT) STOP
        N6=N5+NMP
        N7=N6+NMP
        N8=N7+NMP
        CALL INPUT (A(N2),A(N3),A(N4),A(N5),A(N6),A(N7),A(N1),NSN,
      * NMP,NEQ)
C       INITIALIZE COLUMN HEIGHTS TO ZERO
        N9=N8+NEQ
```

```
      DO 10 I=N8,N9
   10 A(I)=0.0
C     READ AND GENERATE ELEMENT DATA
      WRITE(6,33)
      NEQ1=NEQ+1
      DO 175 J=1,NLC
      IND=1
      LCOUNT=0
      CALL FRAME(A,MTOT)
C     ADDRESS THE DIOGONAL ELEMENTS OF GLOBAL
C     STIFFNESS MATRIX FOR ASSEMBLY
      N11=N10+NEQ1
      CALL CADNUM(A(N8),A(N10),NEQ,NEQ1,NSKY,MBAND)
C     WRITE DATA OF SOLUTION PHASE
      WRITE(6,88)NEQ,NSKY
      WRITE(*,88)NEQ,NSKY
C     ALLOT SPACE FOR STIFFNESS MATRIX AND LOAD VECTOR
      N12=N11+NSKY
      N13=N12+NEQ
      N14=N13+NEQ
      IF(N14.GT.MTOT)CALL ERROR(N14-MTOT,3)
      IF(N14.GT.MTOT)STOP
C     INITIALIZE GLOBAL STIFFNESS MATRIX,LOAD VECTOR
C     AND DISPLACEMENT VECTOR
      NSIZE=NSKY+NEQ+NEQ
      CALL INISKP(A(N11),NSIZE)
      IF(MODEX.NE.0) GO TO 222
C
C     CALCUATES LOAD VECTOR & ELEMENT STIFNESSES AND
C     ASSEMBLES THE STRUCTURE STIFFNESS MATRIX
      IND=2
      LCOUNT=0
      REWIND 2
      REWIND 1
      CALL FRAME(A,MTOT)
C     TRIANGULARIZE STIFFNESS MATRIX
      KTR=1
      CALL SOLVER(A(N11),A(N13),A(N10),NEQ,NEQ1,NSKY,KTR)
  222 CONTINUE
C     READ AND PRINT ELEMENT LOAD MULTIPLIERS
C
      READ(5,77)(ELM(J),J=1,NLC)
      WRITE(6,78)
      WRITE(6,79) (L,L=1,NLC)
      WRITE(6,81)(ELM(J),J=1,NLC)
      READ(5,44)(NLN(J),J=1,NLC)
```

```
      NN1=N13-1
      DO 80 L=1,NLC
      ELMN=ELM(L)
      DO 144 I=N12,NN1
  144 A(I+NEQ)=A(I)*ELMN
      WRITE(6,99) L
      WRITE(6,122)
      NC=NLN(L)
         IF(NC.EQ.0) WRITE(6,1000)
      IF(NC.EQ.0) GO TO 30
C
C     ADD CONCENTRATED LOAD TO CALCULATED LOAD VECTOR
C
      CALL NODLOD (A(N13),A(N1),NC,NSN,NEQ)
C
   30 CONTINUE
      IF(MODEX.NE.0) GO TO 80
C     CALCULATION OF DISPLACEMENTS
      KTR=2
      CALL SOLVER(A(N11),A(N13),A(N10),NEQ,NEQ1,NSKY,KTR
C
C     PRINT THE NODAL DISPLACEMENTS
      WRITE(6,111)
      CALL DISP(A(N13),A(N1),NSN,NEQ)
C
C
C     CALCULATE STRESSES AT THE SELECTED POINTS OF
C     EACH ELEMENT
C
      REWIND 2
      REWIND 1
      IND=3
      LCOUNT=0
      CALL FRAME(A,MTOT)
   80 CONTINUE
  175 CONTINUE
   11 FORMAT(4I5)
   12 FORMAT(I5)
   13 FORMAT(5I3)
   22 FORMAT(5x,'NUMBER OF STRUCTURE NODES',I4/5X,
     *'NUMBER OF MATERIAL GROUPS....',I4/5X,
     *'NUMBER OF LOADING CONDITIONS...',I3//)
   33  FORMAT(//25X,'ELEMENT DATA'/)
   44  FORMAT(10I5)
   55  FORMAT(20A4)
   66  FORMAT(70('*')/25X,20A4/70('*')/25X,'STRUCTURE DATA'
```

```
  77   FORMAT(10F5.2)
  78   FORMAT(//5X,'ELEMENT LOAD MULTIPLIERS'//)
  79   FORMAT(5X,'LOAD CASE NO.',I4,9I8)
  81   FORMAT(15X,10(2X,F6.2))
  88   FORMAT(/20X,'DATA OF SOLUTION PHASE'/20X,22('*')//
     * 10X,'NUMBER OF EQUATIONS....(NEQ)=',I7//10X,
     *'NUMBER OF ELEMENTS IN SK....(NSKY)=',I9//)
  99   FORMAT(//5X,'LOAD CASE NO....',I4/)
 111   FORMAT(///25X,'NODAL DISPLACEMENTS'//4X,'NODE',2X,
     *'X-DISP',4X,'Y-DISP',4X,
     *'Z-DISP',4X,'ROTA-X',4X,
     *'ROTA-Y',4X,'ROTA-Z'/)
 122   FORMAT(/25X,'NODAL LOAD DATA'//)
1000   FORMAT(5X,'NUMBER OF LOADED NODES = 0 ')
C
          CLOSE(1,STATUS='KEEP')
          CLOSE(2,STATUS='KEEP')
          CLOSE(8,STATUS='KEEP')
          CLOSE(6,STATUS='KEEP')
          CLOSE(3,STATUS='KEEP')
       STOP
       END
C
       SUBROUTINE ERROR(N,I)
       GO TO (1,2,3),I
   1   WRITE(6,11)
       GO TO 10
   2   WRITE(6,22)
       GO TO 10
   3   WRITE(6,33)
  10   WRITE(6,44)N
  11   FORMAT(7X,'NOT ENOUGH STORAGE FOR READ-IN OF
     * NDF ARRAY AND NODAL POINT COORDINATES')
  22   FORMAT(//5X,'NOT ENOUGH STORAGE FOR ELTL NODE
     * COORDINATES')
  33   FORMAT(//'NOT ENOUGH STORAGE FOR ASSEEMBLAGE OF ',
     *'GLOBAL STRUCTURE STIFFNESS AND DISPLACEMENTS ',
     *'IN STRESS SOLUTION PHASE '/)
  44   FORMAT(////10X,'***ERROR***STORAGE EXCEEDED BY',I7)
       RETURN
       END
C      SUBROUTINE INPUT  READS NODAL AND MATERIAL DATA
       SUBROUTINE INPUT (X,Y,Z,E,PR,WD,NDF,NSN,NMP,NEQ)
       COMMON/TAPES/STRESS,ARRAY,IPR
       DIMENSION X(NSN),Y(NSN),Z(NSN),E(NMP),PR(NMP)
       DIMENSION NDF(6,NSN),JF(6),WD(NMP)
```

```
      DIMENSION NCYLT(100)
      MN=0
100   READ(5,33)NN,(JF(I),I=1,6),X(NN),Y(NN),Z(NN),NI
      write(*,33)nn,(jf(i),i=1,6),x(nn),y(nn),z(nn),ni
      N=MN+NI
      MN=MN+1
110   DO 120 I=1,6
      NDF(I,NN)=JF(I)
120   CONTINUE
      IF (NI.EQ.0) GO TO 130
      IF (NN-MN)130,125,140
125   CONTINUE
      IF(NSN-NN)170,170,100
130   MN=NN
      GO TO 125
C
C
C     AUTOMATIC GENERATION OF NODAL DATA
C
140   NX=(NN-N+NI)/NI
      XD=(X(NN)-X(N-NI))/FLOAT(NX)
      YD=(Y(NN)-Y(N-NI))/FLOAT(NX)
      ZD=(Z(NN)-Z(N-NI))/FLOAT(NX)
      MN=NN
150   X(N)=X(N-NI)+XD
      Y(N)=Y(N-NI)+YD
      Z(N)=Z(N-NI)+ZD
      DO 160 I=1,6
      NDF(I,N)=JF(I)
160   CONTINUE
      N=N+NI
      IF(N.LT.NN) GO TO 150
      IF(NSN-NN)170,170,100
170   CONTINUE
      IF(IPR.NE.0) GO TO 235
      WRITE(6,44)(I,(NDF(J,I),J=1,6),X(I),Y(I),Z(I),
     * I=1,NSN)
      write(*,44)(i,(ndf(j,i),j=1,6),x(i),y(i),z(i),i=1,nsn
C     CONVERT ZERO AND ONE OF NDF ARRAY TO EQUATION
C     NUMBERS AND ZEROS
235   NEQ=0
      DO 30 N=1,NSN
      DO 30 I=1,6
      IF(NDF(I,N))10,20,10
 20   NEQ=NEQ+1
      NDF(I,N)=NEQ
```

```
      GO TO 30
   10 NDF(I,N)=0
   30 CONTINUE
      IF(IPR.EQ.0)WRITE(6,77)(I,(NDF(J,I),J=1,6),I=1,NSN)
C     READ AND PRINT MATERIAL PROPERTIES
      DO 1 J=1,NMP
    1 READ(5,55)I,E(I),PR(I),wd(I)
      WRITE(6,66)(I,E(I).PR(I),wd(i),i=1,nmp)
C
   33 FORMAT(7I5,3F10.3,I5)
   44 FORMAT(//23X,'NODAL POINT DATA'//7X,'NODE',3X,
     *'NODAL D.O.F.',5X,'X-COORD.',5X,'Y-COORD',5X,
     *'Z-COORD.'//
     *(5X,I4,3X,6I2,3X,F10.4,3X,F10.4,3X,F10.4))
   55 FORMAT(I3,E10.3,2f7.3)
   66 FORMAT(//20X,'MATERIAL PROPERTIES'//5X,
     *'GROUP NO',5X,'YOUNGS MODULUS',5X,'POISSON RATIO',
     *5x,'DENSITY'//
     *(5X,I3,8x,E10.3,10x,f7.3,11x,f7.3))
   77 FORMAT(/5X,'NODE',11X,'EQUATION NUMBERS'//
     *(5X,I4,3X,6I5/))
      RETURN
      END
C     SUBROUTINE INISKP INITIALIZES THE GLOBAL STIFFNESS
C        MATRIX
      SUBROUTINE INISKP(SK,NSIZE)
      DIMENSION SK(NSIZE)
      DO 10 I=1,NSIZE
   10 SK(I)=0.0
      RETURN
      END
      SUBROUTINE COLUMH(CHT,ND,NED,NEQ)
      INTEGER CHT
      DIMENSION CHT(NEQ),ND(NED)
C     CALCULATES THE COLUMN HEIGHTS OF EACH COLUMN
C     IN THE GLOBAL STIFFNESS MATRIX
C
      LS=300000
      DO 30 K=1,NED
      IF(ND(K)) 10,30,10
   10 IF(ND(K)-LS)20,30,30
  20  LS=ND(K)
   30 CONTINUE
      DO 40 K=1,NED
      II=ND(K)
      IF(II.EQ.0) GO TO 40
```

```
      ME=II-LS
      IF(ME.GT.CHT(II))CHT(II)=ME
   40 CONTINUE
      RETURN
      END
      SUBROUTINE CADNUM(CHT,NDS,NEQ,NEQ1,NSKY,MBAND)
      INTEGER CHT(NEQ),NDS(NEQ1)
C     CALCULATES ADDRESSES OF DIOGONAL ELEMENTS IN
C     BANDED MATRIX WHOSE COLUMN HEIGHTS ARE KNOWN;
C
      DO 10 I=1,NEQ1
   10 NDS(I)=0
      NDS(1)=1
      NDS(2)=2
      MBAND=0
      IF(NEQ.EQ.1) GO TO 30
      DO 20 I=2,NEQ
      IF(CHT(I).GT.MBAND) MBAND=CHT(I)
   20 NDS(I+1)=NDS(I)+CHT(I)+1
   30 MBAND=MBAND+1
      NSKY=NDS(NEQ1)-1
         WRITE(8,*)(NDS(I),I=1,NEQ1)
      RETURN
      END
C        SUBROUTINE ASSEMB ASSEMBLES ELEMENT STIFF MATRIX
C        IN GLOBAL STIFF MATRIX
      SUBROUTINE ASSEMB (SK,EK,STIF,SET,NDS,ND,NED,NEQ1,NSKY,NUED,
     *MODE)
      DIMENSION SK(NSKY),NDS(NEQ1),ND(NED),EK(NUED,NUED)
      READ(3,*)LNUM,MODE,STIF,SET
      DO 70 I=1,NED
      II=ND(I)
      IF(II)70,70,30
   30 CONTINUE
      DO 60 J=1,NED
      JJ=ND(J)
      IF(JJ)60,60,40
   40 CONTINUE
      MI=NDS(JJ)
      IJ=JJ-II
      IF(IJ)60,50,50
   50 KK=MI+IJ
      SK(KK)=SK(KK)+EK(I,J)
   60 CONTINUE
   70 CONTINUE
      IF(MODE.NE.1) RETURN
```

```
      DO 35 K=1,3
      KK=ND(K)
      IF(KK.EQ.0)GO TO 35
      MI=NDS(KK)
      SK(MI)=SK(MI)+STIF
   35 CONTINUE
      RETURN
      END
C
C
      SUBROUTINE SOLVER (SK,P,NDS,NN,NEQ1,NSKY,KKK)
      DIMENSION SK(NSKY),P(NN),NDS(NEQ1)
      IF(KKK-2) 40,150,150
   40 DO 140 N=1,NN
      KN=NDS(N)
      KL=KN+1
      KU=NDS(N+1)-1
      KH=KU-KL
      IF(KH) 110,90,50
   50 K=N-KH
      IC=0
      KLT=KU
      DO 80 J=1,KH
      IC=IC+1
      KLT=KLT-1
      KI=NDS(K)
      ND=NDS(K+1)-KI-1
      IF(ND) 80,80,60
   60 IF(IC.LT.ND) GO TO 51
      KK=ND
      GO TO 61
   51 KK=IC
   61 C=0.0
      DO 70 L=1,KK
   70 C=C+SK(KI+L)*SK(KLT+L)
      SK(KLT)=SK(KLT)-C
   80 K=K+1
   90 K=N
      B=0.0
      DO 100 KK=KL,KU
      K=K-1
      KI=NDS(K)
      C=SK(KK)/SK(KI)
      B=B+C*SK(KK)
      SK(KK)=C
  100 CONTINUE
```

```
      SK(KN)=SK(KN)-B
  110 IF(SK(KN)) 120,120,140
  120 WRITE(6,222)N,SK(KN)
      STOP
  140 CONTINUE
      RETURN
C     REDUCE RIGHT HAND SIDE LOAD VECTOR
C
  150 DO 180 N=1,NN
      KL=NDS(N)+1
      KU=NDS(N+1)-1
      IF(KU-KL) 180,160,160
  160 K=N
      C=0.0
      DO 170 KK=KL,KU
      K=K-1
  170 C=C+SK(KK)*P(K)
      P(N)=P(N)-C
  180 CONTINUE
C     BACK SUSTITUTION
C
      DO 200 N=1,NN
      K=NDS(N)
  200 P(N)=P(N)/SK(K)
      IF(NN.EQ.1) RETURN
      N=NN
      DO 230 L=2,NN
      KL=NDS(N)+1
      KU=NDS(N+1)-1
      IF(KU-KL) 230,210,210
  210 K=N
      DO 220 KK=KL,KU
      K=K-1
  220 P(K)=P(K)-SK(KK)*P(N)
  230 N=N-1
      RETURN
  222 FORMAT(//5X,'STOP-STIFFNESS MATRIX NOT'
     *'POSITIVE DEFINITE'/5X,'NON POSITIVE PIVOT FOR ',
     *'EQUATION',I4//10X,'PIVOT =',E20.12)
      END
C
      SUBROUTINE NODLOD(P,NDF,NC,NSN,NEQ)
      COMMON/TAPES/STRESS,ARRAY,IPR
      DIMENSION P(NEQ),NDF(6,NSN)
      DIMENSION CNL(6)
      IF(IPR.EQ.0) WRITE(6,30)
```

```
      DO 20 J=1,NC
      READ(5,11) NODE,(CNL(I),I=1,6)
      WRITE(6,40) NODE,(CNL(I),I=1,6)
      DO 20 I=1,6
      II=NDF(I,NODE)
      IF(II) 20,20,10
   10 P(II)=P(II)+CNL(I)
   20 CONTINUE
      RETURN
   11 FORMAT(I10,6F10.4)
   30 FORMAT(3X,'NODE',5X,'X-FOR',5X,'Y-FOR',5X,'Z-FOR',
     * 5X,'X-MOM',5X,'Y-MOM',5X,'Z-MOM'/)
   40 FORMAT(2X,I4,6(3X,F7.3))
      END
C
      SUBROUTINE DISP(D,NDF,NSN,NEQ)
      DIMENSION D(NEQ),NDF(6,NSN),DISPV(6)
      DO 30 J=1,NSN
      DO 10 I=1,6
   10 DISPV(I)=0.0
      DO 20 I=1,6
      KK=NDF(I,J)
   20 IF(KK.NE.0) DISPV(I)=D(KK)
   30 WRITE(6,22) J,(DISPV(I),I=1,6)
   22 FORMAT(2X,I3,2X,E10.3,2E10.3,1X,E10.3,2E10.3/)
      RETURN
      END
C
C
      SUBROUTINE FRAME(A,MTOT)
      COMMON/DIM/N1,N2,N3,N4,N5,N6,N7,N8,N9,N10,N11,
     * N12,N13,N14
      COMMON/PAR/IND,NSN,NMP,NEQ,NSKY,NEQ1,LCOUNT
      COMMON/TAPES/STRESS,ARRAY,IPR
      COMMON/MULT/ELMN
      COMMON STIF,SET
      COMMON MODE,NDS
      DIMENSION A(MTOT)
      NED=12
      IF(IND.NE.1) GO TO 30
      N10=N9+NED
      N11=N10+1
      N12=N11+1
      N13=N12+1
      NSKY=1
   30 CALL BEAM  (A(N2),A(N3),A(N4),A(N5),A(N6),A(N7),
```

```
     * A(N11),A(N12),A(N1),A(N8),A(N9),A(N10),A(N13),NED)
       RETURN
       END
       SUBROUTINE BEAM(X,Y,Z,E,PR,WD,SK,PQ,NDF,CHT,ND,
     * NDS,P,NED)
       COMMON/PAR/IND,NSN,NMP,NEQ,NSKY,NEQ1,LCOUNT
       COMMON/TAPES/STRESS,ARRAY,IPR
       COMMON/MULT/ELMN
       COMMON STIF,SET,MODE,NDEL(20)
       DIMENSION X(NSN),Y(NSN),Z(NSN),E(NMP),PR(NMP),wd(nmp)
       DIMENSION BX(10),BIX(10),BIY(10),BIZ(10),B(12,12)
       DIMENSION ALIY(20),ADIY(20),ALIZ(20),ALJY(20),ADIZ(20)
       DIMENSION ALJZ(20),ALAX(20),ALTY(20),ALDY(20)
       DIMENSION LSN(100),ALTZ(20),ALDZ(20)
       INTEGER CHT(NEQ)
       DIMENSION Q(12),XL(3),YL(3),ZL(3),LNC(3),MNC(3)
       DIMENSION EK(12,12),T(12,12)
       DIMENSION FIX(12),KODE(3),SWF(3)
       DIMENSION SK(NSKY),P(NEQ),NDF(6,NSN),NDS(NEQ1),
     * ND(NED),PQ(NEQ)
       GO TO(310,350,400),IND
 310   CONTINUE
       READ(5,11) NEL,NGP,NLS
       LCOUNT=LCOUNT+1
       NDEL(LCOUNT)=NEL
       WRITE(6,22)NEL
       WRITE(*,22)NEL
       DO 1 I=1,NGP
   1   READ(5,55)I,BX(I),BIX(I),BIY(I),BIZ(I)
       WRITE(6,77)
       WRITE(6,88)(I,BX(I),BIX(I),BIY(I),BIZ(I),I=1,NGP)
C
       IF(NLS.EQ.0) GO TO 7
       DO 4 I=1,NLS
   4   READ(5,56)I,ALIY(I),ADIY(I),ALIZ(I),ADIZ(I),
     * ALJY(I),ALJZ(I),ALAX(I),ALTY(I),ALDY(I),ALTZ(I),
     * ALDZ(I)
       WRITE(6,31)
       WRITE(6,34)
       WRITE(6,35)
       WRITE(6,36)
       WRITE(6,37)
       WRITE(6,38)
       WRITE(6,39)
       WRITE(6,57)
       WRITE(6,58)
```

```
      WRITE(6,59)
31    FORMAT(/9x,'P-Y =POINT LOAD ALONG LOCAL Y-AXIS'/)
34    FORMAT(9X,'P-Z =POINT LOAD ALONG LOCAL Z-AXIS'/)
35    FORMAT (9X,'DYI =DISTANCE OF PT LOAD FROM NODE-I'/)
36    FORMAT (9X,'DZI =DISTANCE OF Z PT LOAD FROM NODE-I'/)
37    FORMAT (9X,'UDLY=UDL ALONG Y-AXIS'/)
38    FORMAT (9X,'UDLZ=UDL ALONG Z-AXIS'/)
39    FORMAT (9X,'AXIAL=AXIAL FORCE IN MEMBER'/)
57    FORMAT (9X,'TR-Y =TRAPEZOIDAL LOAD ALONG Y-AXIS'/)
58    FORMAT (9X,'TR-Z =TRAPEZOIDAL LOAD ALONG Z-AXIS'/)
59    FORMAT (9X,'DA.I =DISTANCE 'a' FROM NODE I'/)
      WRITE(6,89)
      WRITE(6,91) (I,ALIY(I),ADIY(I),ALIZ(I),ADIZ(I),
     * ALJY(I),ALJZ(I),I=1,NLS)
      WRITE(6,73)
      WRITE(6,74) (I,ALAX(I),ALTY(I),ALDY(I),ALTZ(I),
     * ALDZ(I),I=1,NLS)
   7  CONTINUE
      IF(IPR.EQ.0) WRITE(6,32)
      IF(IPR.EQ.0) WRITE(6,33)
      KK=0
      DO 340 I=1,NEL
      ki=0
      READ(5,44)LNUM,MG,(MNC(J),J=1,3),NG,LSN(LNUM),mode,stif,set
      write(*,*)LNUM,MG,(MNC(J),J=1,3),NG,LSN(LNUM),mode,stif,set
      WRITE(3,*)LNUM,MODE,STIF,SET
      IF(KI.GT.0) GO TO 110
      DO 120 J=1,3
120   LNC(J)=MNC(J)
      MM=LNC(3)
      XL(3)=X(MM)
      YL(3)=Y(MM)
      ZL(3)=Z(MM)
110   KK=KK+1
      IF(LNUM-KK)2,2,3
   3  CONTINUE
      DO 5 J=1,2
   5  LNC(J)=LNC(J)+KI
   2  JJ=0
      DO 330 J=1,2
      K=LNC(J)
      XL(J)=X(K)
      YL(J)=Y(K)
      ZL(J)=Z(K)
      AX=BX(NG)
      AIX=BIX(NG)
```

```
      AIY=BIY(NG)
      AIZ=BIZ(NG)
      DO 320 L=1,6
      JJ=JJ+1
 320  ND(JJ)=NDF(L,K)
 330  CONTINUE
      write(2,*)(nd(ll),ll=1,12),(xl(nn),yl(nn),zl(nn),
     * lnc(nn),nn=1,3),mg,ax,aix,aiy,aiz
      CALL COLUMH(CHT,ND,NED,NEQ)
      LSN(KK)=LSN(LNUM)
      IF(IPR.EQ.0) WRITE(6,66) KK,MG,(LNC(J),J=1,3),NG,LSN(KK)
 340  CONTINUE
      REWIND 3
         READ(5,92)(SWF(I),I=1,3)
  92  FORMAT(3F5.2)
      RETURN
 350  CONTINUE
C
      LCOUNT=LCOUNT+1
      NEL=NDEL(LCOUNT)
         write(*,*)'in ind=2,nel=',nel
      DO 200 LNUM=1,NEL
      READ(2,*)(ND(LL),LL=1,12),(XL(NN),YL(NN),ZL(NN),
     * LNC(NN),NN=1,3),MG,AX,AIX,AIY,AIZ
      DO 10 J=1,12
      DO 10 I=1,12
      T(I,J)=0.0
      EK(I,J)=0.0
  10  Q(J)=0.0
C
      CX=XL(2)-XL(1)
      CY=YL(2)-YL(1)
      CZ=ZL(2)-ZL(1)
      AL=SQRT(CX*CX+CY*CY+CZ*CZ)
      XP=XL(3)-XL(1)
      YP=YL(3)-YL(1)
      ZP=ZL(3)-ZL(1)
C
C     SUBROUTINE 'TRANS' COMPUTES ROTATION
C     TRANSFORMATION MATRIX
C
      CALL TRANS(T,CX,CY,CZ,XP,YP,ZP,AL)
C     SUBROUTINE 'STIFF'COMPUTES THE ELEMENT
C     STIFFNESS MATRIX IN LOCAL CO-ORDINATE SYSTEM
      F=E(MG)
      AMU=PR(MG)
```

```
      CALL STIFF(EK,F,AMU,AL,AX,AIX,AIY,AIZ)
C
C     TRANSFORM STIFFNESS MATRIX IN GLOBAL
C     COORDINATES AND STORE THE K*T PRODUCT ON
C     THE FILE 'STRESS'
      DO 80 I=1,12
      DO 80 J=1,12
      B(I,J)=0.0
      DO 80 K=1,12
   80 B(I,J)=B(I,J)+EK(I,K)*T(K,J)
      WRITE(1,*)((B(I,J),I=1,12),J=1,12)
      DO 90 I=1,12
      DO 90 J=1,12
      EK(I,J)=0.0
      DO 90 K=1,12
   90 EK(I,J)=EK(I,J)+T(K,I)*B(K,J)
      NUED=NED
C     ASSEMBLE ELEMENT STIFFNESS INTO COMPACTED
C     GLOBAL STIFFNESS
      REWIND 8
      READ(8,*)(NDS(I),I=1,NEQ1)
      CALL ASSEMB(SK,EK,stif,set,NDS,ND,NED,NEQ1,NSKY,NUED,mode)
      CALL SELFWT(Q,NED,AL,AX,MG,WD,T,NMP,swf)
C     CALCULATION OF THE LOAD VECTOR DUE TO
C     ELEMENT FORCES
      CALL FIXED(Q,NED,AL,LNUM,LSN,ALIY,ADIY,ALIZ,ADIZ,
     * ALJY,ALJZ,ALAX,ALTY,ALDY,ALTZ,ALDZ)
C     TRANSFORM THE LOAD VECTOR IN THE GLOBAL
C     COORDINATE SYSTEM
C     ADD THE LOAD VECTOR TO THE 'PQ' VECTOR
      WRITE(1,*)(Q(I),I=1,12)
      DO 160 K=1,12
      KK=ND(K)
      IF(KK.EQ.0) GO TO 160
      DO 180 L=1,12
      PQ(KK)=PQ(KK)+T(L,K)*Q(L)
  180 CONTINUE
  160 CONTINUE
      DO 357 I=1,3
      IF(ND(I))357,357,358
  358 Q(I)=STIF*SET
  357 CONTINUE
      write(*,*)'  elm',lnum,' over'
      IF (MODE.NE.1) GO TO 200
      DO 356 K=1,3
      KK=ND(K)
```

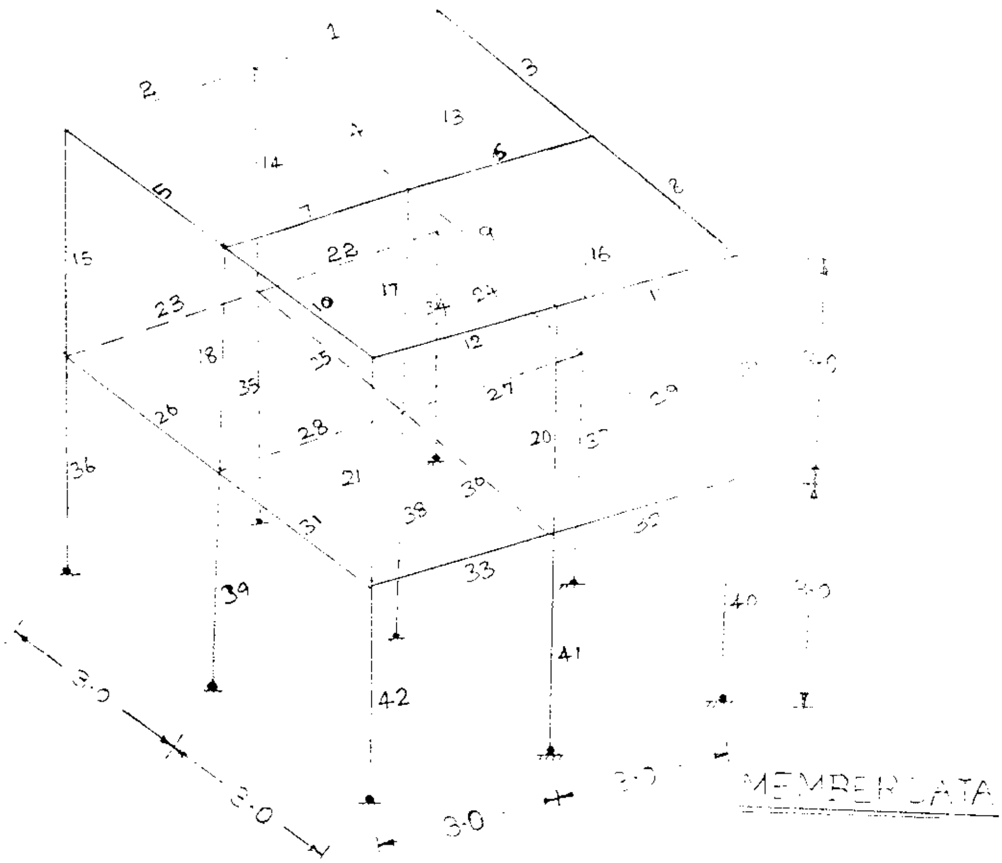
```
      IF(KK.EQ.0)GO TO 356
      PQ(KK)=PQ(KK)+Q(K)
  356 CONTINUE
  200 CONTINUE
      RETURN
  400 CONTINUE
      WRITE(6,98)
      WRITE(6,99)
      LCOUNT=LCOUNT+1
      NEL=NDEL(LCOUNT)
      DO 150 LNUM=1,NEL
      DO 130 I=1,12
  130 Q(I)=0.0
      READ(2,*)(ND(LL),LL=1,12),(XL(NN),YL(NN),ZL(NN),
     *LNC(NN),NN=1,3),MG,AX,AIX,AIY,AIZ
      READ(1,*)((B(I,J),I=1,12),J=1,12)
      READ(1,*)(FIX(I),I=1,12)
      DO 140 M=1,12
      K=ND(M)
      IF(K.EQ.0) GO TO 140
      DO 141 L=1,12
  141 Q(L)=Q(L)+B(L,M)*P(K)
  140 CONTINUE
C
      DO 170 I=1,12
      FIX(I)=FIX(I)*ELMN
  170 Q(I)=Q(I)-FIX(I)
      WRITE(6,132)LNUM,(Q(I),I=1,6)
      WRITE(6,154)(Q(I),I=7,12)
  150 CONTINUE
   11 FORMAT(3I5)
   22 FORMAT(/20X,'NUMBER OF BEAM ELEMENTS...=',I5)
   32 FORMAT(/15X,'ELEMENT CONNECTIVITY DATA'//)
   33 FORMAT(2X,'ELEM.NO',2X,'MAT.GROUP',2X,'NODE-I',2X,'NODE-J',
     *2X,'NODE-K',2X,'GEO.GROUP',2X,'LOAD GROUP'//)
   44 FORMAT(8I4,2f15.4)
   55 FORMAT(I10,4F15.7)
   56 FORMAT(I3,F7.3,F4.2,F7.3,F4.2,F7.3,F7.3,F7.3,F7.3,F4.2,
     *F7.3,F4.2)
   66 FORMAT(3X,I3,7X,I3,3(5X,I4),6X,I3,7X,I3)
   77 FORMAT(/20X,'GEOMETRIC PROPERTY TABLE'/2X,
     *'GROUP NO',2X,'AREA OF C/S',3X,'TORSIONAL M.I',
     *2X,'INERTIA',2X,'INERTIA'/16X,'(A)',9X,'(J)',9X,
     *'(IYY)',7X,'(IZZ)'/)
   88 FORMAT(5X,I4,4F10.3)
   89 FORMAT(/25X,'ELEMENT LOAD TABLE'//3X,'NO',6X,
```

```
     *'P-Y',7X,'DYI',7X,'P-Z',6X,'DZI',6X,'UDL-Y',7X,'UDL-Z'//)
  73 FORMAT(3X,'NO',5X,'AXIAL',
     *5X,'TR-Y',5X,'DA.I',5X,'TR-Z',5X,'DA.I'//)
 74   FORMAT(2X,I3,2X,F8.3,2X,F8.3,2X,F4.2,2X,F8.3,2X,F4.2)
  91 FORMAT(2X,I3,4X,F8.3,4X,F4.2,4X,F8.3,4X,F4.2,4X,F8.3,
     *4X,F8.3)
 98   FORMAT(//20X,'***BEAM FORCES AND MOMENTS***'//)
 99   FORMAT(1X,'ELEM',1X,'NODE',5X,'AXIAL',5X,'SH-Y',5X,'SH-Z',
     *5X,'TWIST',5X,'BM-Y',5X,'BM-Z'//)
 132  FORMAT(1X,I4,1X,'I',1X,6e10.3)
 154  FORMAT(6X,'J',1X,6e10.3/)
      RETURN
      END
c
      SUBROUTINE SELFWT(Q,NED,AL,AX,MG,WD,T,NMP,SWF)
      DIMENSION Q(NED),WD(NMP)
      DIMENSION T(12,12),GQ(12),SWF(3)
      SWT=AL*AX*WD(MG)
      DO 5 J=1,12
  5   GQ(J)=0.0
      DO 10 I=1,3
      GQ(I)=0.5*SWT*SWF(I)
 10   GQ(I+6)=GQ(I)
      DO 15 I=1,12
      DO 15 J=1,12
 15   Q(I)=Q(I)+T(I,J)*GQ(J)
      Q(6)=Q(2)*AL/6.0
      Q(12)=-Q(6)
      RETURN
      END
C
C
      SUBROUTINE FIXED(Q,NED,AL,LNUM,LSN,ALIY,ADIY,
     *ALIZ,ADIZ,ALJY,ALJZ,ALAX,ALTY,ALDY,ALTZ,ALDZ)
      REAL*4 LIY,LIZ,LJY,LJZ,LAX,LTY,LDY,LTZ,LDZ
      DIMENSION Q(NED)
      DIMENSION ALIY(20),ADIY(20),ALIZ(20),ADIZ(20),ALJY(20)
      DIMENSION ALAX(20),ALJZ(20)
      DIMENSION LSN(100)
      DIMENSION ALTY(20),ALDY(20),ALTZ(20),ALDZ(20)
      L=LSN(LNUM)
      LIY=ALIY(L)
      DIY=ADIY(L)
      LIZ=ALIZ(L)
      DIZ=ADIZ(L)
      LJY=ALJY(L)
```

```
      LJZ=ALJZ(L)
      LAX=ALAX(L)
      LTY=ALTY(L)
      LDY=ALDY(L)
      LTZ=ALTZ(L)
      LDZ=ALDZ(L)
      IF(LIY.EQ.0) GO TO 20
      CIY=AL-DIY
      AA=LIY*CIY*CIY*(AL+2*DIY)/AL**3
      BB=LIY-AA
      CC=LIY*DIY*CIY*CIY/AL**2
      DD=-LIY*DIY*DIY*CIY/AL**2
      Q(2)=Q(2)+AA
      Q(6)=Q(6)+CC
      Q(8)=Q(8)+BB
      Q(12)=Q(12)+DD
   20 IF(LIZ.EQ.0) GO TO 30
      CIZ=AL-DIZ
      AA=LIZ*CIZ*CIZ*(3*DIZ+CIZ)/AL**3
      BB=LIZ-AA
      CC=-LIZ*DIZ*CIZ*CIZ/AL**2
      DD=LIZ*DIZ*DIZ*CIZ/AL**2
      Q(3)=Q(3)+AA
      Q(9)=Q(9)+BB
      Q(5)=Q(5)+CC
      Q(11)=Q(11)+DD
   30 IF(LJY.EQ.0) GO TO 40
      AA=LJY*AL*0.5
      BB=LJY*AL*AL/12.0
      Q(2)=Q(2)+AA
      Q(8)=Q(8)+AA
      Q(6)=Q(6)+BB
      Q(12)=Q(12)-BB
   40 IF(LJZ.EQ.0) GO TO 50
      AA=LJZ*AL*0.5
      BB=LJZ*AL*AL/12.0
      Q(3)=Q(3)+AA
      Q(9)=Q(9)+AA
      Q(5)=Q(5)-BB
      Q(11)=Q(11)+BB
   50 Q(1)=Q(1)+LAX*0.5
      Q(7)=Q(7)+LAX*0.5
      IF(LTY.EQ.0.0) GO TO 13
      AA=LTY*(AL-LDY)/2.0
      Q(2)=Q(2)+AA
      Q(8)=Q(8)+AA
```

```
      CC=LDY*LDY*((2.0*AL)-LDY)
      BB=LTY*((AL**3)-CC)/(12.0*AL)
      Q(6)=Q(6)+BB
      Q(12)=Q(12)-BB
   13 IF(LTZ.EQ.0.0) GO TO 17
      AA=LTZ*(AL-LDZ)/2.0
      Q(3)=Q(3)+AA
      Q(9)=Q(9)+AA
      CC=LDZ*LDZ*((2.0*AL)-LDZ)
      BB=LTZ*((AL**3)-CC)/(12.0*AL)
      Q(5)=Q(5)-BB
      Q(11)=Q(11)+BB
   17 RETURN
      END
C
C
      SUBROUTINE TRANS(T,CX,CY,CZ,XP,YP,ZP,AL)
      DIMENSION T(12,12)
      CX=CX/AL
      CY=CY/AL
      CZ=CZ/AL
      IF(CX.EQ.0.0.AND.CZ.EQ.0.0) GO TO 20
      DEN=SQRT(CX**2+CZ**2)
      YPY=(-1.0*CX*CY*XP/DEN)+(DEN*YP)-(CY*CZ*ZP/DEN)
      ZPY=(CX*ZP/DEN)-(CZ*XP/DEN)
      DIV=SQRT(YPY*YPY+ZPY*ZPY)
      SI=ZPY/DIV
      CO=YPY/DIV
      T(1,1)=CX
      T(1,2)=CY
      T(1,3)=CZ
      T(2,1)=(-CX*CY*CO-CZ*SI)/DEN
      T(2,2)=DEN*CO
      T(2,3)=(CX*SI-CY*CZ*CO)/DEN
      T(3,1)=(CX*CY*SI-CZ*CO)/DEN
      T(3,2)=(-DEN*SI)
      T(3,3)=(CY*CZ*SI+CX*CO)/DEN
      GO TO 30
   20 DIV=SQRT(XP*XP+ZP*ZP)
      SI=ZP/DIV
      CO=-XP*CY/DIV
      T(1,2)=CY
      T(2,1)=-CY*CO
      T(2,3)=SI
      T(3,1)=CY*SI
      T(3,3)=CO
```

```
   30  CONTINUE
C      COMPLETE THE TRANSFORMATION MATRIX
       DO 40 I=1,9
       K=I+2
       DO 40 J=I,K
       IF(J.GT.9) GO TO 40
       T(I+3,J+3)=T(I,J)
       T(J+3,I+3)=T(J,I)
   40  CONTINUE
       RETURN
       END
C
       SUBROUTINE STIFF(EK,F,AMU,AL,AX,X,Y,Z)
       DIMENSION EK(12,12)
       G=0.5*F/(1.0+AMU)
       EK(1,1)=F*AX/AL
       EK(2,2)=12.0*F*Z/AL**3
       EK(3,3)=12.0*F*Y/AL**3
       EK(4,4)=G*X/AL
       EK(5,5)=4.0*F*Y/AL
       EK(6,6)=4.0*F*Z/AL
       DO 10 I=1,6
       EK(I+6,I+6)=EK(I,I)
   10  EK(I,I+6)=-EK(I,I)
       EK(5,11)=-0.5*EK(5,11)
       EK(6,12)=-0.5*EK(6,12)
       EK(2,6)=6.0*F*Z/AL**2
       EK(3,5)=-6.0*F*Y/AL**2
       EK(2,12)=EK(2,6)
       EK(3,11)=EK(3,5)
       EK(5,9)=-EK(3,5)
       EK(9,11)=-EK(3,5)
       EK(8,12)=-EK(2,6)
       EK(6,8)=-EK(2,6)
       DO 20 J=1,9
       M=J+1
       DO 20 I=M,12
   20  EK(I,J)=EK(J,I)
       RETURN
       END
```



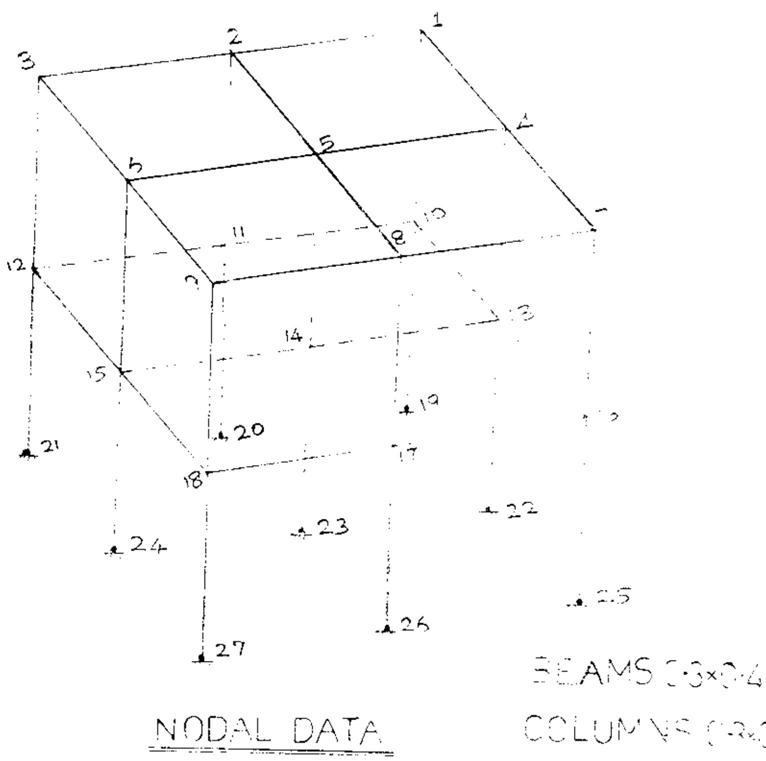
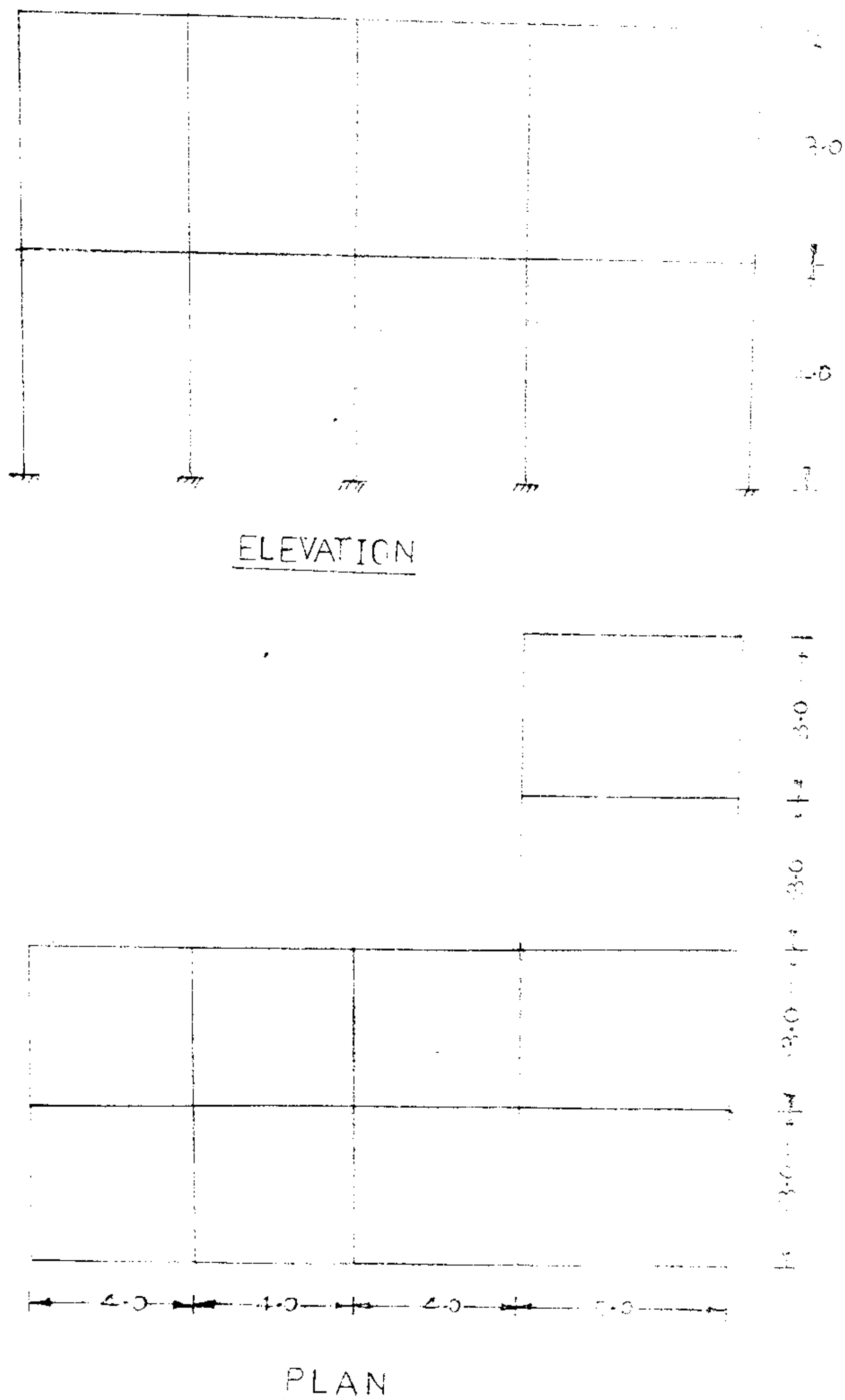


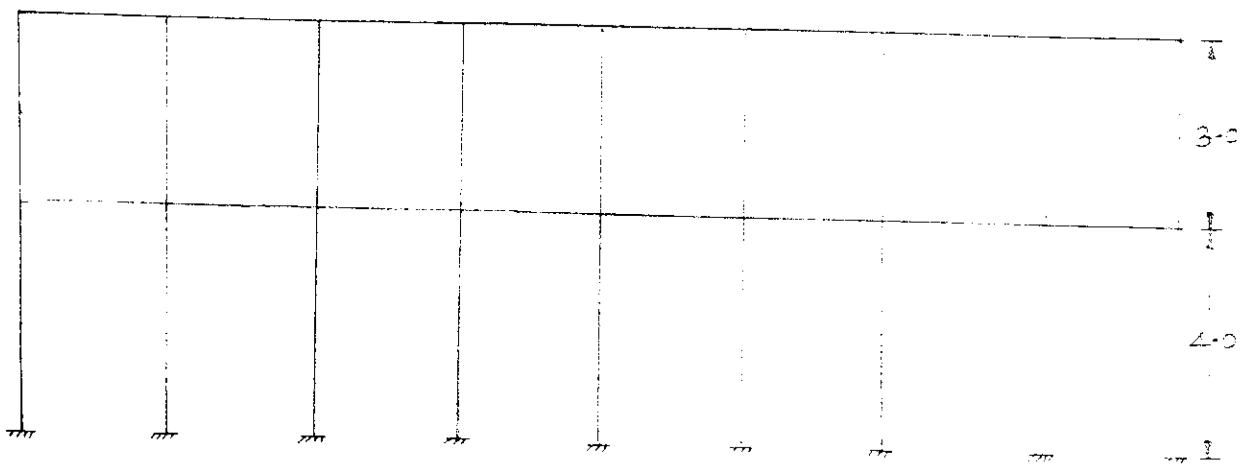
FIG 3·1 FRAME 1



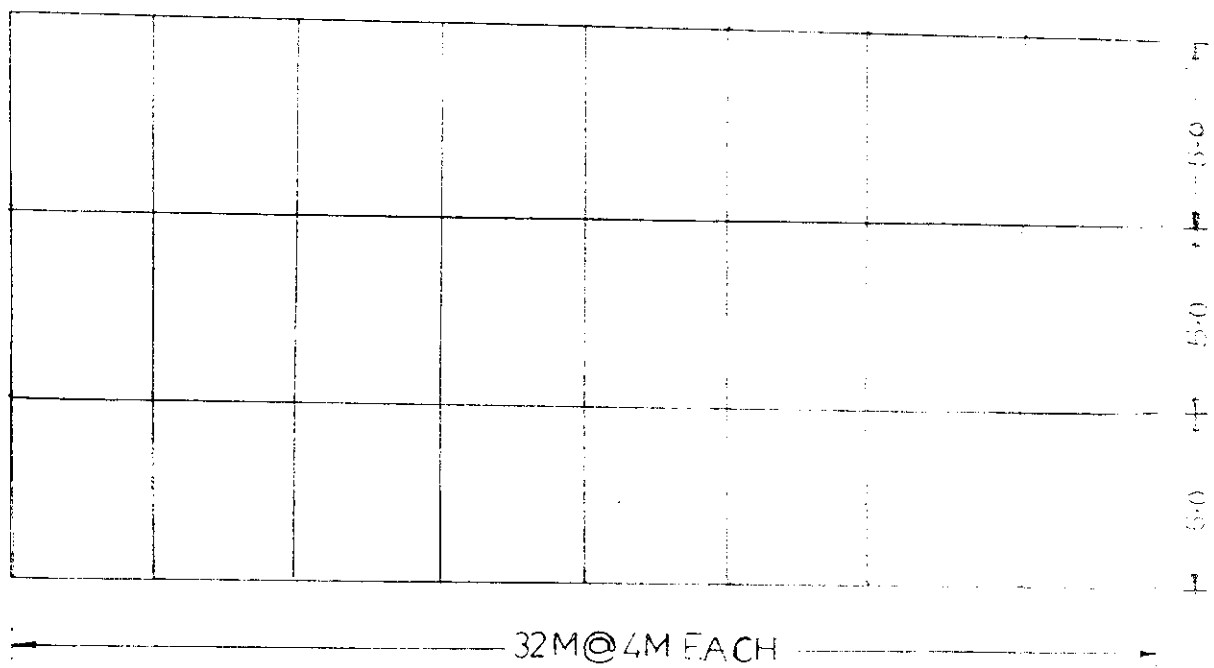
BEAMS 0·3×0·5

COLUMNS 0·3×0·4

FIG 3·2 FRAME-2



ELEVATION



PLAN

BEAMS 0·3×0·6
COLUMNS 0·3×0·6

FIG 3·3 FRAME 3

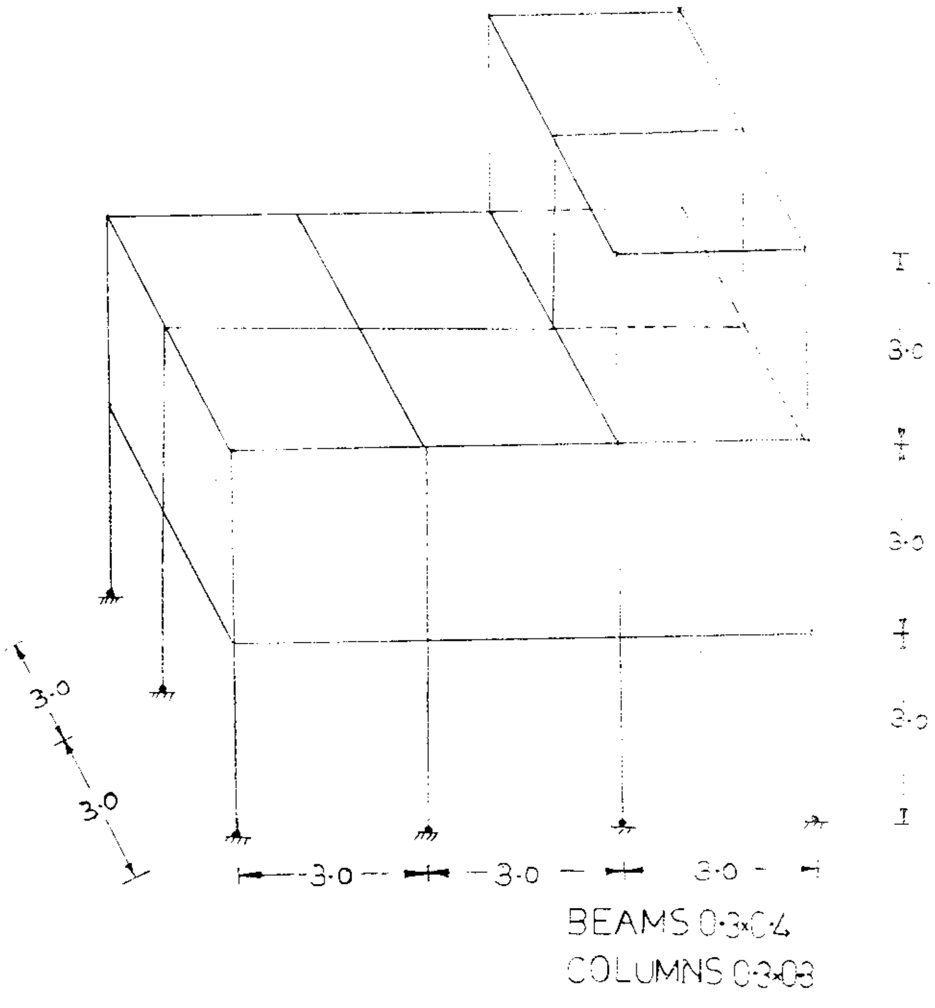


FIG 3·4 FRAME 4

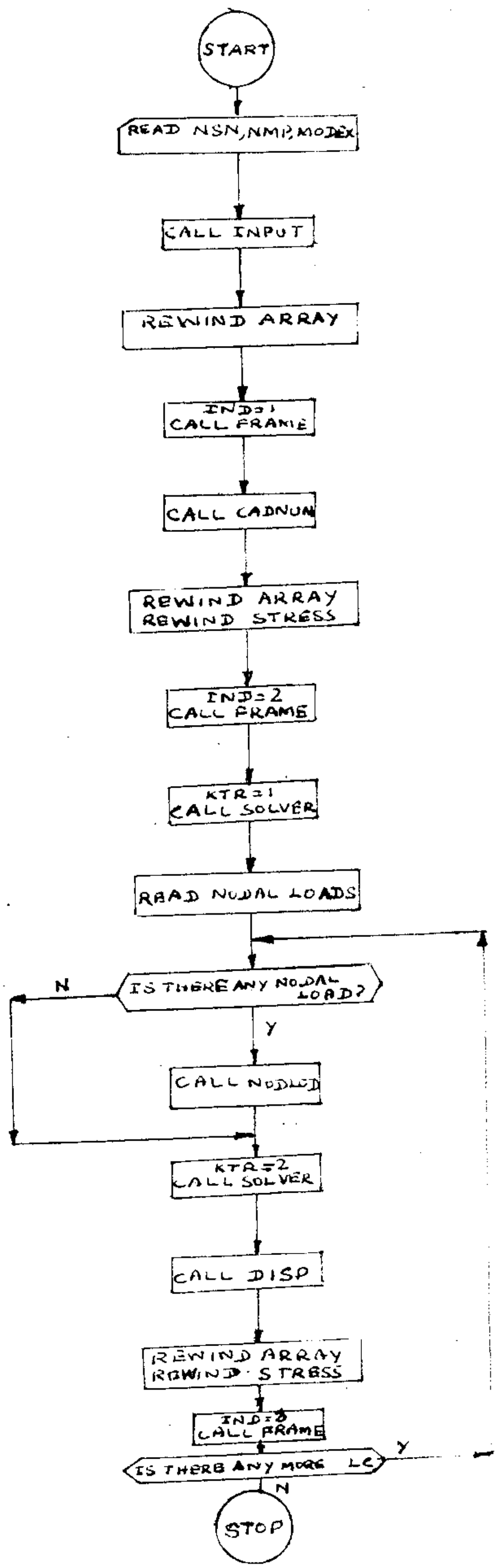


FIG 3.5 FLOW CHART

****

trame-1

****

STRUCTURE DATA

NUMBER OF STRUCTURE NODES 27
NUMBER OF MATERIAL GROUPS.... [illegible]
NUMBER OF LOADING CONDITIONS... [illegible]

DATA OF SOLUTION PHASE
****

NUMBER OF EQUATIONS....(NEQ)= 135

NUMBER OF ELEMENTS IN SK....(NSKY)= 5238

ELEMENT LOAD MULTIPLIERS

LOAD CASE NO. 1
1.00

LOAD CASE NO.... 1

NODAL LOAD DATA

NODAL DISPLACEMENTS

NODE	X-DISP	Y-DISP	Z-DISP	ROTA-X	ROTA Y	ROTA Z
1	-.579E-03	.519E-03	.160E-04	.330E-04	.168E-04	[illegible]
2	-.580E-03	.627E-03	.426E-04	.624E-04	-.307E-04	.556E-04
3	-.580E-03	.599E-03	.151E-04	-.141E-04	-.334E-04	-.196E-04
4	-.596E-03	.520E-03	.269E-04	-.424E-05	[illegible]	.450E-05
5	-.600E-03	.625E-03	.496E-04	-.103E-04	-.104E-04	.729E-05

6	-.603E-03	.652E-03	.442E-04	-.260E-04	-.257E-04	-.740E-04
7	-.504E-03	.521E-03	-.478E-05	-.731E-04	-.378E-04	.293E-05
8	-.503E-03	.624E-03	.111E-04	-.106E-03	-.609E-05	.358E-05
9	-.501E-03	.651E-03	.181E-06	-.806E-04	.147E-04	-.138E-05
10	-.428E-03	.399E-03	.912E-05	-.363E-04	-.140E-04	.668E-04
11	-.428E-03	.443E-03	.225E-04	-.433E-04	-.260E-04	.327E-04
12	-.428E-03	.445E-03	.115E-04	-.341E-04	-.515E-04	.541E-04
13	-.452E-03	.400E-03	.670E-05	-.466E-04	.151E-04	-.415E-04
14	-.452E-03	.441E-03	.154E-04	-.694E-04	-.101E-03	.175E-04
15	-.452E-03	.444E-03	.189E-04	-.516E-04	.220E-04	-.506E-04
16	-.384E-03	.399E-03	-.847E-05	.693E-05	-.497E-04	.360E-05
17	-.385E-03	.440E-03	-.210E-05	.250E-04	-.259E-04	-.512E-06
18	-.385E-03	.444E-03	-.400E-05	-.516E-05	-.293E-05	-.280E-05
19	.000E+00	.000E+00	.000E+00	-.181E-03	-.207E-03	.668E-04
20	.000E+00	.000E+00	.000E+00	-.200E-03	-.201E-03	.327E-04
21	.000E+00	.000E+00	.000E+00	-.206E-03	-.188E-03	.541E-04
22	.000E+00	.000E+00	.000E+00	-.177E-03	-.234E-03	-.415E-04
23	.000E+00	.000E+00	.000E+00	-.186E-03	-.175E-03	.175E-04
24	.000E+00	.000E+00	.000E+00	-.196E-03	-.237E-03	-.506E-04
25	.000E+00	.000E+00	.000E+00	-.203E-03	-.167E-03	.360E-05
26	.000E+00	.000E+00	.000E+00	-.233E-03	-.179E-03	-.512E-06
27	.000E+00	.000E+00	.000E+00	-.219E-03	-.191E-03	.280E-05

BEAM FORCES AND MOMENTS

ELEM	NODE	AXIAL	SH-Y	SH-Z	TWIST	BM-Y	BM-Z
1	I	1.6352	.1224	-.1117	-.5183	-2.2994	-.0402
	J	-1.6352	-.1224	.1117	.5183	2.6351	.4074
2	I	-.2454	.7996	2.4176	1.3462	-6.7514	1.7505
	J	.2454	-.7996	-2.4176	-1.3462	-.5013	.6481
3	I	-1.2448	4.4329	.1271	-.1882	-.4074	1.6736
	J	1.2448	5.6923	-.1271	.1882	.0262	-3.5624
4	I	1.6740	8.9178	.7348	-.3570	-1.4563	3.2872
	J	-1.6740	11.7072	-.7348	.3570	-.7481	-7.4712
5	I	-.8229	8.7672	-.4790	.7340	.0651	5.6684
	J	.8229	9.7456	.4790	-.7340	1.3671	-7.7439
6	I	2.7862	-.9871	-.9427	.1058	-4.4756	-1.5672
	J	-2.7862	.9871	.9427	-.1058	7.3035	-1.3943
7	I	2.9706	-1.2358	1.1649	.2763	-7.9682	-1.2576
	J	-2.9706	1.2358	-1.1649	-.2763	4.4735	-2.4499
8	I	-.6904	6.7116	.8814	1.1489	-1.2675	4.6278
	J	.6904	3.4134	-.8814	-1.1489	-1.3765	.3196
9	I	1.2916	12.9649	1.1059	-.0764	-1.5860	9.0220
	J	-1.2916	7.6601	-1.1059	.0764	-1.6318	-1.0649
10	I	.9923	7.3277	-.1106	-.7113	.5984	5.7599
	J	-.9923	2.7973	.1106	.7113	-.3061	1.0358
11	I	-.9948	.9106	-.9781	.5761	-1.2316	1.3707
	J	.9948	-.9106	.9781	-.5761	4.1655	1.3612
12	I	-1.4170	.2285	.0395	-.4435	-2.9188	.3063
	J	1.4170	-.2285	-.0395	.4435	2.8002	.3791
13	I	-4.5446	1.5082	-1.1225	.3672	1.1756	2.0364
	J	4.5446	-1.5082	1.1225	.3672	2.1919	2.4881
14	I	-13.2704	.6584	2.0865	-.2015	2.3546	1.0220
	J	13.2704	-.6584	2.0865	.2015	3.9044	.9532
15	I	-2.4176	-.2456	.7996	.6481	-1.0524	-.2354
	J	2.4176	.2456	-.7996	.6481	-1.3462	-.5013

16	I	-13.3465	2.0319	-.4328	-.3257	.3388	2.9574
	J	13.3465	-2.0319	.4328	.3257	.9596	3.1382
17	I	-22.5643	-.1865	-.6312	.2178	.5135	-.9435
	J	22.5643	.1865	.6312	.2178	1.3804	.3841
18	I	-16.7264	-1.4239	.8921	.2063	-1.1502	-2.4858
	J	16.7264	1.4239	-.8921	-.2063	-1.5260	-1.7859
19	I	-2.4353	-.1133	-.2202	.0059	.9173	-.2572
	J	2.4353	.1133	.2202	.0059	-.2566	-.0827
20	I	-8.6778	-.6836	.6095	-.0360	-1.3737	-.8800
	J	8.6778	.6836	-.6095	.0360	.0455	-1.1707
21	I	-2.7578	-1.3065	.7638	-.0124	-1.5992	-1.8306
	J	2.7578	1.3065	-.7638	.0124	-.5923	-2.0889
22	I	-.4283	1.0329	.9124	.1225	-4.3566	1.7994
	J	.4283	-1.0329	-.9124	-.1225	1.5194	1.2992
23	I	-.2389	1.2443	2.4902	-.1616	-6.5256	1.7097
	J	.2389	-1.2443	-2.4902	.1616	-.9450	2.0233
24	I	-.6194	-2.3838	-.1354	-.5132	-2.1666	-3.4252
	J	.6194	2.3838	.1354	.5132	2.5730	-3.7263
25	I	1.1105	-3.1647	.0074	1.3187	-2.3074	-4.3641
	J	-1.1105	3.1647	-.0074	-1.3187	2.7852	-5.1299
26	I	1.2050	-2.6582	.1831	-1.2934	-2.5714	-3.7309
	J	-1.2050	2.6582	-.1831	1.2934	2.1220	-4.2438
27	I	-.8383	-1.2719	2.3473	.4013	-8.2635	-2.0843
	J	.8383	1.2719	-2.3473	-.4013	1.2217	-1.7315
28	I	.1076	1.0248	2.2475	-.3132	-8.0136	1.7800
	J	-.1076	-1.0248	-2.2475	.3132	1.2710	1.2945
29	I	.4498	4.1961	-.1119	1.1414	-.1630	1.0798
	J	-.4498	5.9289	.1119	-1.1414	.4986	-5.6790
30	I	.9247	9.3525	-.3886	-1.3207	.4586	5.6211
	J	-.9247	11.2725	.3886	1.3207	.7074	-6.5012
31	I	.3649	3.8457	.1320	.4385	-.5481	.5582
	J	-.3649	6.2793	-.1320	-.4385	.1522	-4.3085
32	I	.5760	.3565	-2.0935	-.3180	.3254	.5046
	J	-.5760	-.3565	2.0935	.3180	5.9554	.5649

33	I	.2546	.0820	-.8832	[illegible]	[illegible]	[illegible]
	J	-.2546	-.0820	.8832	[illegible]	[illegible]	[illegible]
34	I	-6.0160	.9445	-.7090	.0000	[illegible]	[illegible]
	J	6.0160	-.9445	.7090	.0000	[illegible]	[illegible]
35	I	-14.8573	.8552	-.7646	.0000	[illegible]	.0000
	J	14.8573	-.8552	.7646	.0000	[illegible]	[illegible]
36	I	-7.5660	-.6677	.8390	.0000	[illegible]	.0000
	J	7.5660	.6677	-.8390	.0000	[illegible]	[illegible]
37	I	-4.4193	1.2172	-.6355	.0000	.0000	.0000
	J	4.4193	-1.2172	.6355	.0000	1.9064	[illegible]
38	I	-10.1469	.3635	-.5700	.0000	[illegible]	.0000
	J	10.1469	-.3635	.5700	.0000	[illegible]	[illegible]
39	I	-12.4700	-1.2653	.7074	.0000	[illegible]	.0000
	J	12.4700	1.2653	-.7074	.0000	[illegible]	[illegible]
40	I	5.5871	.5746	-1.0266	.0000	[illegible]	.0000
	J	-5.5871	-.5746	1.0266	.0000	[illegible]	[illegible]
41	I	1.3844	.7507	-1.2595	.0000	[illegible]	.0000
	J	-1.3844	-.7507	1.2595	.0000	[illegible]	[illegible]
42	I	2.6383	-.9199	1.0467	.0000	[illegible]	.0000
	J	-2.6383	.9199	-1.0467	.0000	[illegible]	[illegible]

```
**********************************************************************************
                              frame-2
**********************************************************************************
                              STRUCTURE DATA
```

NUMBER OF STRUCTURE NODES 57
NUMBER OF MATERIAL GROUPS.... 1
NUMBER OF LOADING CONDITIONS... 1

DATA OF SOLUTION PHASE

****

NUMBER OF EQUATIONS....(NEQ)= 285

NUMBER OF ELEMENTS IN SK....(NSKY)= 20307

ELEMENT LOAD MULTIPLIERS

LOAD CASE NO. 1
1.00

LOAD CASE NO.... 1

BEAM FORCES AND MOMENTS

ELEM	NODE	AXIAL	SH-Y	SH-Z	TWIST	[illegible]	[illegible]
1	I	-4.1038	12.0661	-1.0959	-.2084	[illegible]	[illegible]
	J	4.1038	14.9339	1.0959	.2084	[illegible]	[illegible]
2	I	-5.8915	15.8776	-.6660	-.2948	[illegible]	[illegible]
	J	5.8915	11.1224	.6660	.2948	[illegible]	[illegible]
3	I	-10.0546	12.6961	.2563	1.4098	[illegible]	[illegible]
	J	10.0546	14.3039	-.2563	-1.4098	[illegible]	[illegible]
4	I	-8.1588	13.9517	-.1427	8.6581	[illegible]	[illegible]
	J	8.1588	13.0483	.1427	-8.6581	[illegible]	[illegible]

5	I	11.7109	.6971	.7441	-.1941	-21.0345	1.8055
	J	-11.7109	-.6971	-.7441	.1941	17.3140	1.6824
6	I	12.9106	1.1884	.7073	.3258	-20.9533	2.7809
	J	-12.9106	-1.1884	-.7073	-.3258	17.4169	3.1610
7	I	11.0388	-.1502	.0453	-.2994	-19.8649	.5162
	J	-11.0388	.1502	-.0453	.2994	19.6382	-1.2672
8	I	7.5410	-.1272	-.2410	-.2022	-21.0759	.2830
	J	-7.5410	.1272	.2410	.2022	22.2809	-.9191
9	I	-8.6031	.5535	-.5810	.4567	-20.8452	.2477
	J	8.6031	-.5535	.5810	-.4567	23.7503	2.5199
10	I	.3363	10.9886	-.6994	-.2229	1.5872	2.3377
	J	-.3363	16.0114	.6994	.2229	.5110	-9.8719
11	I	1.9710	13.4486	-2.9495	-1.0500	2.5722	6.6264
	J	-1.9710	13.5514	2.9495	1.0500	5.2762	-8.7805
12	I	.4451	12.9750	-4.6159	-1.1594	7.1541	7.8743
	J	-.4451	14.0250	4.6159	1.1594	6.6927	-9.4491
13	I	-.1649	15.6469	-5.4547	-7.5924	7.4169	9.6706
	J	.1649	11.3531	5.4547	7.5924	8.9453	-3.2300
14	I	1.8916	-1.4399	1.5068	-.9354	-16.2615	-6.4379
	J	-1.8916	1.4399	-1.5068	.9354	10.2341	.6783
15	I	-.1989	-.8183	.7254	.1061	-15.8949	-4.3235
	J	.1989	.8183	-.7254	-.1061	12.9935	1.0504
16	I	3.5556	-.1033	.2461	-.0512	-12.9615	3.5338
	J	-3.5556	.1033	-.2461	.0512	11.9770	5.9468
17	I	-1.0722	11.6237	3.6842	.6991	-6.0957	4.1114
	J	1.0722	15.3763	-3.6842	-.6991	-4.9589	-9.7403
18	I	-.6444	14.3084	3.7449	6.1191	-5.0640	7.9252
	J	.6444	12.6916	-3.7449	-6.1191	-6.1706	-5.5000
19	I	8.3494	1.2780	-3.1603	.4503	-7.9768	3.5878
	J	-8.3494	-1.2780	3.1603	-.4503	20.6187	1.5242
20	I	.7943	.7221	-2.6263	-1.2903	-9.8415	1.9977
	J	-.7943	-.7221	2.6263	1.2903	20.3467	.8907
21	I	-4.2210	-1.0230	2.5822	-2.4981	-18.1878	-4.2534
	J	4.2210	1.0230	-2.5822	2.4981	7.8593	1.8518

22	I	-.6362	9.3861	-.3642	-.1507	.6165	3.0560
	J	.6362	17.6139	.3642	.1507	.4757	-15.9977
23	I	-.3588	21.8806	-.7708	-.0276	.8683	21.2967
	J	.3588	5.1194	.7708	.0276	1.4441	3.8453
24	I	8.4987	-.6546	3.2286	.5571	-21.4637	-.8372
	J	-8.4987	.6546	-3.2286	-.5571	8.5493	-1.7811
25	I	.0054	.1997	2.2427	1.2975	-20.2651	.4014
	J	-.0054	-.1997	-2.2427	-1.2975	11.2942	.3976
26	I	-3.4501	-.6642	-2.5372	1.3471	-7.8314	-.5826
	J	3.4501	.6642	2.5372	-1.3471	17.9802	-2.0740
27	I	-1.8368	.3256	-2.8531	-.9754	-2.6963	.5999
	J	1.8368	-.3256	2.8531	.9754	11.2556	.3770
28	I	-1.6004	-1.0779	2.5890	-5.5686	-10.8356	-1.4803
	J	1.6004	1.0779	-2.5890	5.5686	3.0685	-1.7533
29	I	-11.3220	12.8068	-3.4067	.0389	4.9890	17.1775
	J	11.3220	-12.8068	3.4067	-.0389	5.2310	21.2429
30	I	-30.1042	12.4805	-.5993	.3077	.4952	16.4017
	J	30.1042	-12.4805	.5993	-.3077	1.3026	21.0397
31	I	-23.7732	10.1167	-4.3133	.1069	8.8164	12.1897
	J	23.7732	-10.1167	4.3133	-.1069	4.1234	18.1602
32	I	-28.4965	7.9402	1.7685	.0705	-3.9381	9.9929
	J	28.4965	-7.9402	-1.7685	-.0705	-1.3674	13.8277
33	I	-12.4673	-8.7459	7.6053	-.3333	-15.2138	-14.0508
	J	12.4673	8.7459	-7.6053	.3333	-7.6021	-12.1870
34	I	-11.7327	11.0115	.3608	.0952	1.0613	15.9435
	J	11.7327	-11.0115	-.3608	-.0952	-2.1437	17.0911
35	I	-30.1673	10.6604	-.4462	.0778	.4190	15.3913
	J	30.1673	-10.6604	.4462	-.0778	.9198	16.5898
36	I	-25.1319	7.4807	-.1311	1.5640	.1230	9.3513
	J	25.1319	-7.4807	.1311	-1.5640	.2706	13.0909
37	I	-28.6126	6.9014	-.2427	.8632	.6411	9.1799
	J	28.6126	-6.9014	.2427	-.8632	.0367	11.5244
38	I	-11.8309	-6.7041	-.9645	1.4964	.1718	7.4882
	J	11.8309	6.7041	.9645	-1.4964	2.7221	-12.6240

39	I	-16.2239	-2.7734	1.3010	.5526	[illegible]	[illegible]
	J	16.2239	2.7734	-1.3010	.5526	[illegible]	[illegible]
40	I	-33.1292	-.9330	-.4246	-.9729	[illegible]	[illegible]
	J	33.1292	.9330	.4246	.9729	[illegible]	[illegible]
41	I	-15.3770	4.0316	-1.4212	-2.5758	[illegible]	[illegible]
	J	15.3770	-4.0316	1.4212	2.5758	[illegible]	[illegible]
42	I	-2.9971	-.5135	2.5688	.0704	[illegible]	.5441
	J	2.9971	.5135	-2.5688	-.0704	-3.5492	.9964
43	I	-34.6256	.3823	.2450	-.0520	1.9762	.9423
	J	34.6256	-.3823	-.2450	.0520	[illegible]	.2047
44	I	-1.8536	.1198	-.1325	-.6150	[illegible]	.2504
	J	1.8536	-.1198	.1325	.6150	[illegible]	.6137
45	I	-3.5543	5.6457	1.1823	.9152	-2.3898	[illegible]
	J	3.5543	-5.6457	-1.1823	-.9152	[illegible]	[illegible]
46	I	-.8392	5.4477	-.0366	-.0225	[illegible]	[illegible]
	J	.8392	-5.4477	.0366	.0225	[illegible]	[illegible]
47	I	-3.6150	-6.0390	-.9362	-.9945	[illegible]	[illegible]
	J	3.6150	6.0390	.9362	.9945	[illegible]	[illegible]
48	I	2.7839	13.3304	-.7929	.2497	[illegible]	[illegible]
	J	-2.7839	13.6696	.7929	-.2497	[illegible]	[illegible]
49	I	2.6378	8.3881	-1.7434	[illegible]	[illegible]	[illegible]
	J	-2.6378	18.6119	1.7434	[illegible]	[illegible]	[illegible]
50	I	.8186	8.2417	-.2356	-.2988	[illegible]	[illegible]
	J	-.8186	18.7583	.2356	.2988	[illegible]	[illegible]
51	I	4.7172	37.7385	.2906	[illegible]	[illegible]	[illegible]
	J	-4.7172	34.2615	-.2906	[illegible]	[illegible]	[illegible]
52	I	-10.6003	.5850	2.8489	1.2281	[illegible]	[illegible]
	J	10.6003	-.5850	-2.8489	-1.2281	[illegible]	[illegible]
53	I	-9.9998	1.2044	3.1843	[illegible]	[illegible]	[illegible]
	J	9.9998	-1.2044	-3.1843	.3022	[illegible]	[illegible]
54	I	-4.5111	.6047	1.6646	1.8825	[illegible]	[illegible]
	J	4.5111	-.6047	-1.6646	-1.8825	[illegible]	[illegible]
55	I	-3.4104	.1451	.8331	[illegible]	[illegible]	[illegible]
	J	3.4104	-.1451	-.8331	[illegible]	[illegible]	[illegible]

56	I	10.2594	.2054	-5.6146	-4.4212	-10.2246	.2422
	J	-10.2594	-.2054	5.6146	4.4212	38.2973	.7346
57	I	1.3267	11.4223	.6679	-.2636	-1.5255	5.8129
	J	-1.3267	15.5777	-.6679	.2636	-.4783	-10.0460
58	I	2.0631	14.6237	2.1996	-3.0829	-2.6858	10.0756
	J	-2.0631	12.3763	-2.1996	3.0829	-3.9129	-6.7044
59	I	.8185	13.3658	1.4945	-.0432	-1.8880	8.5785
	J	-.8185	13.6342	-1.4945	.0432	-2.5956	-8.9812
60	I	.5428	14.6001	-.5826	-1.6082	1.1095	3.8349
	J	-.5428	12.3999	.5826	1.6082	.6383	-5.5347
61	I	6.2936	1.7058	3.9939	.0821	-25.5804	2.8884
	J	-6.2936	-1.7058	-3.9939	-.0821	9.6049	3.9349
62	I	8.5885	.2826	3.5198	.0354	-24.6377	.4649
	J	-8.5885	-.2826	-3.5198	-.0354	10.5586	.6655
63	I	6.6009	.8549	-1.3832	.3736	-14.6907	1.4300
	J	-6.6009	-.8549	1.3832	-.3736	20.2235	2.0195
64	I	1.7753	13.2724	.6710	-.0491	-.8404	1.1933
	J	-1.7753	13.7276	-.6710	.0491	-1.1725	-6.8759
65	I	1.5601	13.3575	-.6076	-.3546	.6006	6.5917
	J	-1.5601	13.6425	.6076	.3546	1.2222	-7.0192
66	I	-6.8728	1.0843	2.5887	-.8436	-22.7945	2.8111
	J	6.8728	-1.0843	-2.5887	.8436	12.4395	1.5262
67	I	-16.1406	.0227	2.6676	1.2603	-22.8336	.0232
	J	16.1406	-.0227	-2.6676	-1.2603	12.1650	.0674
68	I	.4399	.9384	-3.6413	-.5276	[illegible]	2.4240
	J	-.4399	-.9384	3.6413	.5276	[illegible]	1.3295
69	I	2.6221	16.1389	-1.3207	.0775	1.9494	8.4898
	J	-2.6221	10.8611	1.3207	-.0775	2.0121	-.5730
70	I	3.0791	13.5951	-1.1965	.6504	1.8768	4.9111
	J	-3.0791	13.4049	1.1965	-.6504	1.7126	-4.6259
71	I	-5.5906	.2044	5.7611	.1823	-25.5258	.5528
	J	5.5906	-.2044	-5.7611	-.1823	2.4811	.4649
72	I	-9.7833	.0684	5.7474	-1.2160	-25.6222	.1204
	J	9.7833	-.0684	-5.7474	1.2160	2.6324	.1533

73	I	3.5791	.1506	-2.5359	1.6799	-11.5745	.3036
	J	-3.5791	-.1506	2.5359	-1.6799	21.3180	.2990
74	I	1.9036	12.8028	-1.0279	.0544	1.4574	5.3160
	J	-1.9036	14.1972	1.0279	.0544	1.5264	-7.9076
75	I	1.3377	16.2099	.9297	-2.5566	-1.4956	11.2099
	J	-1.3377	10.7901	-.9297	2.5566	-1.2938	-5.0802
76	I	-21.8035	2.9993	-.0377	.0000	.0000	.0000
	J	21.8035	-2.9993	.0377	.0000	.1509	11.9973
77	I	-48.9776	3.4312	.4590	.0000	.0000	.0000
	J	48.9776	-3.4312	-.4590	.0000	-1.8361	13.7246
78	I	-86.7007	3.8070	-.8105	.0000	.0000	.0000
	J	86.7007	-3.8070	.8105	.0000	3.2419	15.2280
79	I	-46.4216	4.2943	1.0949	.0000	.0000	.0000
	J	46.4216	-4.2943	-1.0949	.0000	-4.3795	17.1772
80	I	-41.1143	1.8044	2.6827	.0000	.0000	.0000
	J	41.1143	-1.8044	-2.6827	.0000	-10.7309	7.2175
81	I	-3.1593	-.2568	-.3809	.0000	.0000	.0000
	J	3.1593	.2568	.3809	.0000	1.5234	-1.0273
82	I	-3.1501	-.8710	.0218	.0000	.0000	.0000
	J	3.1501	.8710	-.0218	.0000	-.0872	-5.4842
83	I	2.9394	-2.6190	.0123	.0000	.0000	.0000
	J	-2.9394	2.6190	-.0123	.0000	-.0492	-10.4759
84	I	2.3083	-3.0199	-.1045	.0000	.0000	.0000
	J	-2.3083	3.0199	.1045	.0000	.4180	-12.0795
85	I	-3.6623	-3.6276	.2278	.0000	.0000	.0000
	J	3.6623	3.6276	-.2278	.0000	-.9111	-14.5107
86	I	-18.3000	-2.8794	.1472	.0000	.0000	.0000
	J	18.3000	2.8794	-.1472	.0000	-.5888	11.5178
87	I	-32.7028	-3.2881	.0505	.0000	.0000	.0000
	J	32.7028	3.2881	-.0505	.0000	-.2019	-13.1526
88	I	-13.7265	-3.4502	.2224	.0000	.0000	.0000
	J	13.7265	3.4502	-.2224	.0000	-.3895	13.8006
89	I	-15.9637	3.1162	-.8267	.0000	.0000	.0000
	J	15.9637	-3.1162	.8267	.0000	3.3064	2.4646

90	I	-28.8118	3.4572	.2579	.0000	.0000	.0000
	J	28.8118	-3.4572	-.2579	.0000	-1.0315	13.8286
91	I	12.4781	3.3073	-.7493	.0000	.0000	.0000
	J	-12.4781	-3.3073	.7493	.0000	2.9971	13.2292
92	I	-22.2968	-2.0010	1.0253	.0000	.0000	.0000
	J	22.2968	2.0010	-1.0253	.0000	-4.1013	-8.0041
93	I	-36.9937	-2.3779	.5976	.0000	.0000	.0000
	J	36.9937	2.3779	-.5976	.0000	-2.3903	-9.5117
94	I	-16.9410	-3.3889	.2506	.0000	.0000	.0000
	J	16.9410	3.3889	-.2506	.0000	-1.0022	-13.5557

CHAPTER IV

ANALYSIS INCORPORATING SOIL STRUCTURE INTERACTION

4.1 Interactive analysis for frames with isolated footings:

The multistoreyed frame is considered as a space frame. An initial analysis should be carried out, assuming that there is no settlement in the foundation. With the calculated axial loads in columns the foundation area can be obtained from the allowable bearing capacity of soil. Then the settlement at each footing can be calculated from the following formula.

$$S = q.B.(1- \mu^2).Iw/Es \qquad (1)$$

where

S = settlement of footing

q = intensity of contact pressure

B = least lateral dimension of footing

Iw = influence factor which depends on shape and rigidity of footing

Es = static stress strain modulus

μ = Poisson's ratio.

The influence value for flexible footing, can be computed from the equation proposed by Steinbrenner (2) for the corner of a rectangular area of dimensions BxL as

$$I_w = \frac{1}{\pi}\left[\frac{L}{B}\,\ln\left(\frac{1+\sqrt{(L/B)^2+1}}{(L/B)}\right) + \ln\left(\frac{L}{B}+\sqrt{\left(\frac{L}{B}\right)^2+1}\right)\right] \qquad (2)$$

The influence value for the center of the footing is obtained by summing two corner values from the above equation with due attention paid to the B and L dimensions of contributing area.

Similarly the rotation of column due to settlement can be calculated using the formula, suggested by

$$\tan \theta = \frac{V.e.\ (1-\mu^2)}{B.\ L^2\ .\ E_s} \times Im \qquad (3)$$

where

θ = angle of rotation, with horizantal

V = axial force in column

e = eccentricity of force

Im = influence factor

The calculated settlements should be entered in the input file SPIN.DAT according to the position of column member data. Again the analysis should be carried out. The results in second analysis will be different. With these column loads, the settlements in the footings are to be found out. The same procedure should be followed until the result converges.

This is a modified form of the method suggested by O.P.Jain (1), in which he considered the frame as two dimensional frame, in both directions. In this method, the number of iterations are more. But, in this thesis work since the frame is considered as a space frame, the effect of settlement in the plane frames in two direction is included in a single step. So, the results converge in two or three iterations.

4.2. Interactive Analysis for Rafted Frame:

For the interactive analysis of rafted frame, the method suggested by L.Strat, has been followed. Since this method, does not include the soil media directly in the interactive analysis, the programming and input data preparation will be easier.

An initial analysis, without interaction is to be carried out. With these column forces, an approximate raft foundation design is to be done by assuming a rigid base, in order to get the dimensions of the foundation slab and beams. For the second analysis, the foundation members are also included. The foundation beam members are subjected to upward thrust. In the matrix method of structural analysis the element loads should be converted into nodal loads. So, we have to find out the pressure distribution below the foundation. For this, we can follow any of the procedure given in Appendix-E or Appendix-F based on IS 2950 (Part I) - 1981. The concrete raft foundation is neither a rigid, nor a perfectly flexible one. Since the procedure given in Appendix - F accounts for non uniform column spacing and load intesity, the same can be used satisfactorily. A program has been developed to find out the moments about two axes, vertical shear and the settlements at the columns. These calculated nodal moments and settlements are incorporated in the input for second analysis. The same procedure has to be followed, until the results converge.

Although, this method is an iterative procedure, it will be easier to follow. It is found that the results are converging within three or four iterations. The moments, shear and settlement calculation includes the modulus of subgrade, poisson Ratio and modulus of elasticity of soil. Thus the effect of soil is indirectly incorporated. The problems considered in the previous chapter are analysed, by considering the interaction between foundation (on winkler's model) and super structure.

4.3 Program for the analysis of raft foundation as per IS: 2950 (Part I) - 1981.

The raft foundation is assumed as flexible. In practice the column spacing and column load intensity will not be uniform. So the procedure given in Appendix-F of IS: 2950 (part-I) - 1981, is used. The soil media is assumed as winkler's media. For every column load, the moment about x and y directions, shear and settlements at various points can be calculated using the formulae and charts. To get the total moments and settlements, the calculated moments and settlement for each column load, should be added. This procedure is cumbersome if followed manually. So a computer program is developed. The input data should be stored in a file RAFTIN.DAT. The results are stored in a file RAFTOUT.DAT.

INPUT DATA

1. No: of points where the moments have to be calculated. (I).

   N = Number of points.

2. Raft material properties (E10.3, F 5.3, F 6.4, E 10.3)

   em,pr,k,csg

   em = Young's modulus of Raft material (concrete)

   pr = Poisson's Ratio

   t = Thickness of Raft

   csg = modulus of subgrade.

3. Nodal Data (I5,3E12.3)

   nn,x(nn),y(nn),p(nn) in one line for each point

   nn= Node number (corresponding to column position)

   x(nn),y(nn) = x and y coordinate of the node.

   p(nn) = Load at the point.

4.4 Results of the analysis:

The results of the analysis are given in table

```
$large
$debug
c       program to find out b.m,s.f and def in raft
c       foundation on the basis of winkler's concept
c       as per IS 2950(part1)-1981
        common x,y,rsf,zs,rdf,ax,ram,tam,p,axam,ayam
        common zd,rmf,zm,rss,zss,rp,ay,ashpu,shpu
           dimension x(100),y(100),rsf(7),zs(7),rdf(9),ax(11)
           dimension zd(9),rmf(10),zm(10),rss(7),zss(7),rp(100),ay(11)
        dimension ram(100),tam(100),p(100),shpu(100,100)
        dimension xam(100,100),yam(100,100),ashpu(100)
        dimension axam(100),ayam(100),adef(100),def(100,100)
           open(5,file='raftin.dat',status='old')
        open(6,file='raftout.dat',status='new')
c
c       coeff for zs(function for shear in flex formula)
c
           ns=7
        rsf(1)=0.0
           rsf(2)=0.7
        rsf(3)=2.0
        rsf(4)=2.8
        rsf(5)=3.0
        rsf(6)=4.0
        rsf(7)=6.0
        zs(1)=0.5
        zs(2)=0.42
        zs(3)=0.16
        zs(4)=0.07
        zs(5)=0.04
        zs(6)=0.0
        zs(7)=0.0
c       coeff for zm(function for moment)
c
           nm=10
        rmf(1)=0.0
        rmf(2)=0.3
        rmf(3)=0.7
        rmf(4)=1.0
           rmf(5)=2.0
           rmf(6)=3.0
        rmf(7)=3.5
        rmf(8)=4.0
        rmf(9)=5.0
        rmf(10)=6.0
        zm(1)=0.0
```

```
      zm(2)=-0.18
      zm(3)=-0.23
         zm(4)=-0.20
         zm(5)=-0.13
         zm(6)=-0.05
      zm(7)=-0.03
      zm(8)=-0.02
      zm(9)=-0.01
      zm(10)=0.0
c     coeff for rdf(function for deflection)
c
         nd=9
      rdf(1)=0.6
         rdf(2)=0.8
      rdf(3)=1.0
      rdf(4)=1.4
      rdf(5)=2.0
      rdf(6)=2.3
         rdf(7)=3.3
         rdf(8)=4.4
      rdf(9)=6.0
      zd(1)=-0.55
         zd(2)=-0.3
      zd(3)=-0.15
      zd(4)=-0.03
      zd(5)=0.03
      zd(6)=0.04
         zd(7)=0.02
      zd(8)=0.01
      zd(9)=0.0
c        coeff for zss(function for shear in shear formula)
c
         nss=7
      rss(1)=0.8
      rss(2)=1.1
      rss(3)=1.8
      rss(4)=2.2
      rss(5)=2.9
      rss(6)=4.0
      rss(7)=6.0
      zss(1)=0.55
      zss(2)=0.34
      zss(3)=0.11
      zss(4)=0.04
      zss(5)=-0.02
      zss(6)=-0.01
```

```
      zss(7)=0.0
c
         read(5,9)nps
    9 format(i3)
      write(6,90)nps
   90 format(10x,'NUMBER OF POINTS=',I5/)
      read(5,11)em,pr,t,csg
      write(6,92)em,pr,t,csg
   11 format(e10.3,f5.3,f8.4,e10.3)
   92 format(27X,'PROPERTIES'//
     *       10X,'E FOR RAFT MATERIAL =',E10.3/
     *       10X,'POISSON RATIO =',f5.3/
     *       10X,'THICKNESS OF RAFT =',F8.4/
     *       10X,'MODULUS OF SUBGRADE =',e10.3//)
c
      write(6,50)
   50 format(22x,'NODAL FORCES AND SETTLEMENTS'/
     *       22x,28('*')/
     *       2x,'NODE NO:',3X,'MOM ABOUT X',3X,'MOM ABOUT Y',
     *       5x,'SHEAR',8X,'DEF'/)
      d=em*t*t/(12.0*(1-pr*pr))
      effs=(d/csg)**0.25
      do 95 i=1,nps
      read(5,12)nn,x(nn),y(nn),p(nn)
      write(*,*)nn,x(nn),y(nn),p(nn)
   12 format(i5,3e12.3)
   95 continue
      write(*,*)' input over'
      do 100 i=1,nps
      do 120 j=1,nps
      rx=x(i)-x(j)
      ry=y(i)-y(j)
      rr=sqrt(rx*rx+ry*ry)
      rs=rr/effs
      if(rs.eq.0) go to 75
c
      call lagint(rsf,zs,rs,zsout,ns)
         write(*,*)rs,zsout
      call lagint(rmf,zm,rs,zmout,nm)
      write(*,*)rs,zmout
      call lagint(rss,zss,rs,zssout,nss)
      write(*,*)rs,zssout
   75 call lagint(rdf,zd,rs,zdout,nd)
      write(*,*)rs,zdout
      if(rs.eq.0) go to 76
c
```

```
      ram(j)=-p(i)*(zdout-((1-pr)*zmout/rs))/4.0
      write(*,*)ram(j)
      tam(j)=-p(i)*(pr*zdout+((1-pr)*zmout/rs))/4.0
      write(*,*)tam(j)
      shpu(i,j)=-p(i)*zssout/(4.0*effs)
      rp(j)=sqrt(x(j)*x(j)+y(j)*y(j))
      cosp=x(j)/rp(j)
      sinp=y(j)/rp(j)
      xam(i,j)=(ram(j)*cosp*cosp)+(tam(j)*sinp*sinp)
      yam(i,j)=(ram(j)*sinp*sinp)+(tam(j)*cosp*cosp)
 76   def(i,j)=p(i)*effs*effs*zsout/(4.0*d)
      if(i.ne.1) go to 86
      go to 120
 86   axam(j)=xam(i,j)+xam(i-1,j)
      ayam(j)=yam(i,j)+yam(i-1,j)
      adef(j)=def(i,j)+def(i-1,j)
      ashpu(j)=shpu(i,j)+shpu(i-1,j)
120   continue
100   continue
      do 301  k=1,nps
 301  write(6,172)k,axam(k),ayam(k),ashpu(k),adef(k)
 172  format(i4,4(5x,e10.3))
      stop
      end
      subroutine lagint(ax,ay,axin,ayout,n)
      dimension ax(11),ay(11)
c     axin =(r/l) value for which z coeff value is needed
c     ayout= z function value
      ayout=0.0
      do 20 i=1,n
      term=ay(i)
      do 10 j=1,n
      if(i.eq.j) go to 10
      term=term*(axin-ax(j))/(ax(i)-ax(j))
 10   continue
      ayout=ayout+term
 20   continue
      return
      end
```

****

frame

****

STRUCTURE DATA

NUMBER OF STRUCTURE NODES 27
NUMBER OF MATERIAL GROUPS.... 1
NUMBER OF LOADING CONDITIONS... [illegible]

NUMBER OF BEAM ELEMENTS = 42

BEAM FORCES AND MOMENTS

ELEM	NODE	AXIAL	SH-Y	SH-Z	TWIST	BM-Y	BM-Z
1	I	.123E+01	.116E+00	.573E+00	-.746E+00	[illegible]	[illegible]
	J	-.123E+01	-.116E+00	.507E+00	.746E+00	[illegible]	[illegible]
2	I	.589E+00	.790E+00	.594E+01	.116E+01	[illegible]	[illegible]
	J	-.589E+00	-.790E+00	.286E+01	-.116E+01	[illegible]	[illegible]
3	I	-.180E+01	.637E+01	.192E+00	[illegible]	[illegible]	[illegible]
	J	.122E+01	.613E+01	-.192E+00	[illegible]	[illegible]	[illegible]
4	I	.253E+01	.925E+01	.869E+00	[illegible]	[illegible]	[illegible]
	J	-.253E+01	.134E+02	-.869E+00	[illegible]	[illegible]	[illegible]
5	I	.164E+01	.893E+01	.582E+00	[illegible]	[illegible]	[illegible]

	J	.164E+01	.103E+02	.582E+00	-.590E+00	.175E+01	-.836E+01
6	I	.291E+01	-.151E+01	.107E+02	-.817E-01	-.530E+01	-.239E+01
	J	-.291E+01	.151E+01	.120E+02	.817E-01	.719E+01	-.215E+01
7	I	.401E+01	-.155E+01	.125E+02	.179E+00	-.854E+01	-.165E+01
	J	-.401E+01	.155E+01	.103E+02	-.179E+00	.525E+01	-.302E+01
8	I	-.686E+00	.770E+01	.147E+01	.164E+01	-.217E+01	.669E+01
	J	.686E+00	.280E+01	-.147E+01	-.164E+01	-.225E+01	.127E+01
9	I	.187E+01	.149E+02	.191E+01	-.838E-01	-.287E+01	.111E+02
	J	-.187E+01	.775E+01	-.191E+01	.838E-01	-.285E+01	-.409E+00
10	I	.966E+00	.717E+01	.305E+00	-.839E+00	.728E-01	.537E+01
	J	-.966E+00	.333E+01	-.305E+00	.839E+00	-.987E-00	.379E+00
11	I	-.138E+01	.143E+01	.406E+01	.460E+00	-.845E+00	.222E+01
	J	.138E+01	-.143E+01	.674E+01	-.460E+00	.487E-01	.238E+01
12	I	-.146E+01	.527E+00	.458E+01	-.469E+00	-.175E+01	.646E+00
	J	.146E+01	-.527E+00	.622E+01	.469E+00	.420E+01	.915E+00
13	I	.135E+01	.104E+01	-.110E+01	.600E+00	.934E-00	.769E-01
	J	-.135E+01	-.104E+01	.110E+01	-.600E+00	.237E+01	.304E+01
14	I	-.319E+01	.131E+01	-.194E+01	.550E-02	.213E+01	.160E+01
	J	.319E+01	-.131E+01	.194E+01	-.550E-02	.368E+01	.233E+01
15	I	.286E+01	.588E+00	.790E+00	.752E+00	-.121E-01	.220E+01
	J	-.286E+01	-.588E+00	-.790E+00	-.752E+00	-.116E-01	-.455E+00
16	I	-.310E+01	.162E+01	-.983E+00	-.297E+00	.104E-01	[illegible]
	J	.310E+01	-.162E+01	.983E+00	.297E+00	.191E-01	.349E+01

17	I	-.385E+01	.681E-01	-.701E+00	-.926E-01	.542E+00	-.895E+00
	J	.385E+01	-.681E-01	.701E+00	.926E-01	.156E+01	.110E+01
18	I	-.562E+01	-.701E+00	.171E+00	.311E+00	-.862E-01	-.554E+00
	J	.562E+01	.701E+00	-.171E+00	-.311E+00	-.428E+00	-.155E+01
19	I	-.686E+01	.945E-01	-.745E+00	.327E-01	.143E+01	-.510E+00
	J	.686E+01	-.945E-01	.745E+00	-.327E-01	.807E+00	.794E+00
20	I	-.191E+02	-.183E+01	.971E+00	.128E+00	-.239E+01	-.245E+01
	J	.191E+02	.183E+01	-.971E+00	-.128E+00	-.521E+01	-.303E+01
21	I	-.955E+01	-.177E+01	.439E+00	.514E-01	-.141E+01	-.194E+01
	J	.955E+01	.177E+01	-.439E+00	-.514E-01	.904E-01	.336E+01
22	I	-.466E+00	.832E+00	.132E+02	-.612E+00	-.164E+02	.150E+01
	J	.466E+00	-.832E+00	-.244E+01	.612E+00	-.714E+01	.100E+01
23	I	-.501E+00	.111E+01	.145E+02	-.837E+00	-.151E+02	[illegible]
	J	.501E+00	-.111E+01	-.369E+01	.837E+00	-.122E+02	.284E+01
24	I	-.150E+01	-.361E+01	.527E+01	-.124E+01	-.210E+01	-.564E+01
	J	.150E+01	.361E+01	.553E+01	.124E+01	.249E+01	-.519E+01
25	I	.889E+00	-.303E+01	.519E+01	.124E+01	-.251E+01	-.420E+01
	J	-.889E+00	.303E+01	.561E+01	-.124E+01	.314E+01	-.498E+01
26	I	.217E+01	-.116E+01	.559E+01	-.214E+01	-.259E+01	-.125E+01
	J	-.217E+01	.116E+01	.521E+01	.214E+01	.201E+01	-.222E+01
27	I	-.973E+00	-.241E+01	.190E+02	.100E+00	-.178E+02	-.389E+01
	J	.973E+00	.241E+01	.376E+01	-.100E+00	-.505E+01	-.352E+01

28	I	.709E-05	.213E+01	.185E+02	-.668E+00	-.170E+02	.355E+01
	J	-.709E-05	-.213E+01	.423E+01	.668E+00	-.438E+01	.285E+01
29	I	-.446E-02	.240E+01	-.134E+01	.627E+00	.170E+01	-.161E+01
	J	.446E-02	.810E+01	.134E+01	-.627E+00	.231E+01	-.693E+01
30	I	.721E+00	.990E+01	-.218E+01	-.184E+01	.310E+01	.335E+01
	J	-.721E+00	.128E+02	.218E+01	.184E+01	.343E+01	-.755E+01
31	I	.725E+00	.576E+01	-.104E+01	-.299E+00	.124E+01	.300E+01
	J	-.725E+00	.474E+01	.104E+01	.299E+00	.188E+01	-.146E+01
32	I	.976E+00	.146E+01	-.793E-01	-.110E+01	.608E+01	.254E+01
	J	-.976E+00	-.146E+01	.109E+02	.110E+01	.104E+02	.204E+01
33	I	-.163E-02	.115E+01	.841E+00	-.519E-02	.302E+01	.122E+01
	J	.163E-02	-.115E+01	.996E+01	.519E-02	.107E+02	.155E+01
34	I	.110E+02	.584E+01	-.177E+01	-.482E-06	.556E-05	.755E-05
	J	-.110E+02	-.584E+01	.177E+01	.482E-06	.532E-01	.175E+02
35	I	.583E+01	.646E+01	-.765E+00	.652E-06	.223E-05	.160E-04
	J	-.583E+01	-.646E+01	.765E+00	-.652E-06	.230E-01	.152E+02
36	I	-.199E+01	-.551E+01	-.265E+00	.115E-06	.292E-06	-.115E-04
	J	.199E+01	.551E+01	.265E+00	-.115E-06	.795E-06	-.165E+02
37	I	.219E+02	.484E+01	-.189E+01	-.518E-06	.317E-05	.210E-04
	J	-.219E+02	-.484E+01	.189E+01	.518E-06	.567E+01	.145E+02
38	I	.860E+01	.447E+01	-.590E+00	.139E-06	.620E-06	[illegible]
	J	-.860E+01	-.447E+01	.590E+00	-.139E-06	.177E+01	[illegible]
39	I	-.172E+02	-.487E+01	-.512E+00	.304E-06	.156E-05	-.568E-05

	J	.172E+02	.487E+01	.512E+00	-.304E-06	.153E+01	-.146E+02
40	I	.132E+01	.241E+01	-.220E+01	.121E-06	.688E-05	.111E-04
	J	-.132E+01	-.241E+01	.220E+01	-.121E-06	.660E+01	.722E+01
41	I	-.180E+02	.303E+01	-.138E+01	-.587E-06	.879E-06	.822E-05
	J	.180E+02	-.303E+01	.138E+01	.587E-06	.414E+01	.909E+01
42	I	-.148E+02	-.281E+01	.141E-01	.153E-07	.494E-06	-.586E-05
	J	.148E+02	.281E+01	-.141E-01	-.153E-07	-.423E-01	-.842E+01

CHAPTER V

ANALYSIS USING APPROXIMATE METHODS

The exact analysis of a space frame is impossible without the use of a computer. If this facility is not available, some approximate analysis procedures can be followed.

For the analysis, for gravity loads substitute frame method can be followed, in which the frame is considered as a plane frame. A substitute frame consists of beams at a given floor level together with columns above and below with their far ends fixed. The substitute frames are analysed, for all spans loaded condition.

For the wind load and earthquake analysis, either portal method or cantilever method can be followed. In the available literature it is given that, the portal method is supposed to be satisfactory for most buildings upto about 25 storeys, whereas, the cantilever method is good enough for about 35 storeys.

The example problems are also analysed by the approximate methods and the results are compared with the exact analysis.

The results of the analysis are given in Table 5.1 and 5.2

****

frame-1

****

STRUCTURE DATA

NUMBER OF STRUCTURE NODES 27
NUMBER OF MATERIAL GROUPS.... 1
NUMBER OF LOADING CONDITIONS... [illegible]

BEAM FORCES AND MOMENTS

ELEM	NODE	AXIAL	SH-Y	SH-Z	TWIST	BM-Y	BM-Z
1	I	.120E+01	.000E+00	.550E+01	.000E+00	-.275E+01	.000E+00
	J	-.120E+01	.000E+00	.485E+01	.000E+00	.179E+01	.000E+00
2	I	.531E+00	.000E+00	.766E+01	.000E+00	-.698E+01	.000E+00
	J	-.531E+00	.000E+00	.269E+01	.000E+00	-.474E+00	.000E+00
3	I	-.119E+01	.429E+01	.000E+00	.000E+00	.000E+00	.158E+01
	J	.119E+01	.606E+01	.000E+00	.000E+00	.000E+00	-.422E+01
4	I	.247E+01	.925E+01	.000E+00	.000E+00	.000E+00	.310E+01
	J	-.247E+01	.134E+02	.000E+00	.000E+00	.000E+00	-.933E+01
5	I	-.160E+01	.880E+01	.000E+00	.000E+00	.000E+00	.545E+01
	J	.160E+01	.101E+02	.000E+00	.000E+00	.000E+00	-.828E+01
6	I	.289E+01	.000E+00	.107E+02	.000E+00	-.527E+01	.000E+00
	J	-.289E+01	.000E+00	.120E+02	.000E+00	.721E+01	.000E+00
7	I	.393E+01	.000E+00	.124E+02	.000E+00	-.852E+01	.000E+00
	J	-.393E+01	.000E+00	.122E+02	.000E+00	.522E+01	.000E+00
8	I	-.666E+00	.760E+01	.000E+00	.000E+00	.000E+00	.601E+01
	J	.666E+00	.275E+01	.000E+00	.000E+00	.000E+00	.126E+01
9	I	.184E+01	.149E+02	.000E+00	.000E+00	.000E+00	.111E+02
	J	-.184E+01	.777E+01	.000E+00	.000E+00	.000E+00	-.452E+00
10	I	.958E+00	.709E+01	.000E+00	.000E+00	.000E+00	.533E+01
	J	-.958E+00	.326E+01	.000E+00	.000E+00	.000E+00	.4[illegible]1E+00
11	I	-.133E+01	.000E+00	.389E+01	.000E+00	-.807E+01	.000E+00
	J	.133E+01	.000E+00	.646E+01	.000E+00	.466E+01	.000E+00

12	I	-.142E+01	.000E+00	.437E+01	.000E+00	-.165E+01	.000E+00
	J	.142E+01	.000E+00	.598E+01	.000E+00	.407E+01	.000E+00
13	I	.120E+01	.102E+01	-.108E+01	.000E+00	.915E+00	.104E+00
	J	-.120E+01	-.102E+01	.108E+01	.000E+00	.232E+01	.295E+01
14	I	-.358E+01	.127E+01	-.192E+01	.000E+00	.211E+01	.155E+01
	J	.358E+01	-.127E+01	.192E+01	.000E+00	.364E+01	.225E+01
15	I	.269E+01	.531E+00	.778E+00	.000E+00	-.119E+01	.207E+01
	J	-.269E+01	-.531E+00	-.778E+00	.000E+00	-.115E+01	-.474E+00
16	I	-.296E+01	.164E+01	-.953E+00	.000E+00	.100E+01	.144E+01
	J	.296E+01	-.164E+01	.953E+00	.000E+00	.186E+01	.347E+01
17	I	-.385E+01	.423E-01	-.679E+00	.000E+00	.511E+00	-.921E+00
	J	.385E+01	-.423E-01	.679E+00	.000E+00	.153E+01	.105E+01
18	I	-.545E+01	-.711E+00	.190E+00	.000E+00	-.115E+00	-.616E+00
	J	.545E+01	.711E+00	-.190E+00	.000E+00	-.454E+00	-.152E+01
19	I	-.664E+01	.110E+00	-.730E+00	.000E+00	.141E+01	-.463E+00
	J	.664E+01	-.110E+00	.730E+00	.000E+00	.781E+00	.792E+00
20	I	-.186E+02	-.176E+01	.956E+00	.000E+00	-.237E+01	-.236E+01
	J	.186E+02	.176E+01	-.956E+00	.000E+00	-.498E+00	-.293E+01
21	I	-.924E+01	-.171E+01	.448E+00	.000E+00	-.141E+01	-.188E+01
	J	.924E+01	.171E+01	-.448E+00	.000E+01	.633E-01	-.324E+01
22	I	-.443E+00	.000E+00	.245E+01	.000E+00	-.157E+02	.000E+00
	J	.443E+00	.000E+00	-.237E+01	.000E+00	-.690E+01	.000E+00
23	I	-.464E+00	.000E+00	.139E+02	.000E+00	-.146E+02	.000E+00
	J	.464E+00	.000E+00	-.360E+01	.000E+00	-.118E+02	.000E+00
24	I	-.144E+01	.341E+01	.000E+00	.0000+00	.000E+00	-.548E+01
	J	.144E+01	.351E+01	.000E+00	.000E+00	.000E+00	-.506E+01
25	I	.887E+00	-.300E+01	.000E+00	.000E+00	.000E+00	-.414E+01
	J	-.887E+00	.300E+01	.000E+00	.000E+00	.000E+00	-.485E+01
26	I	.208E+01	-.119E+01	.000E+00	.000E+00	.000E+00	-.131E+01
	J	-.208E+01	.119E+01	.000E+00	.000E+00	.000E+00	-.225E+01
27	I	-.970E+00	.000E+00	.187E+02	.000E+00	-.173E+02	.000E+00
	J	.970E+00	.000E+00	.397E+01	.000E+00	-.475E+01	.000E+00
28	I	-.105E-01	.000E+00	.182E+02	.000E+00	-.166E+02	.000E+00
	J	.105E-01	.000E+00	.445E+01	.000E+00	-.409E+01	.000E+00

29	I	.321E-02	.241E+0[illegible]	.000E+00	.000E+00	.000E+00	-.155E+01
	J	-.321E-02	.794E+01	.000E+00	.000E+00	.000E+00	-.676E+01
30	I	.716E+00	.993E+01	.000E+00	.000E+00	.000E+00	.340E+01
	J	-.716E+00	.127E+02	.000E+00	.000E+00	.000E+00	-.758E+01
31	I	.699E+00	.564E+01	.000E+00	.000E+00	.000E+00	.289E+01
	J	-.699E+00	.471E+01	.000E+00	.000E+00	.000E+00	-.151E+01
32	I	.930E+00	.000E+00	-.121E+00	.000E+00	.589E+01	.000E+00
	J	-.930E+00	.000E+00	.105E+02	.000E+00	.100E+02	.000E+00
33	I	-.950E-02	.110E+01	.767E+00	.000E+00	.296E+01	.146E+01
	J	.950E-02	-.110E+01	.958E+01	.000E+00	.103E+02	.185E+01
34	I	.104E+02	.562E+01	-.172E+01	.000E+00	.415E-05	-.158E-06
	J	-.104E+02	-.562E+01	.172E+01	.000E+00	.515E+01	.169E+02
35	I	.499E+01	.622E+01	-.754E+00	.000E+00	.189E-05	.120E-04
	J	-.499E+01	-.622E+01	.754E+00	.000E+00	.226E+01	.187E+02
36	I	-.209E+01	-.530E+01	-.230E+00	-.103E-07	.790E-07	-.751E-05
	J	.209E+01	.530E+01	.230E+00	.103E-07	.691E+01	-.159E+02
37	I	.217E+02	.468E+01	-.183E+01	-.475E-06	.391E-05	.848E-05
	J	-.217E+02	-.468E+01	.183E+01	.475E-06	.548E+01	.140E+02
38	I	.860E+01	.428E+01	-.575E+00	.275E-06	.113E-05	.451E-05
	J	-.860E+01	-.428E+01	.575E+00	-.275E-06	.173E+01	.128E+02
39	I	-.169E+02	-.471E+01	-.472E+00	.306E-06	.103E-05	-.519E-05
	J	.169E+02	.471E+01	.472E+00	-.306E-06	.142E+01	-.141E+02
40	I	.142E+01	.233E+01	-.214E+01	.135E-05	.481E-05	.658E-05
	J	-.142E+01	-.233E+01	.214E+01	-.135E-05	.643E+01	.699E+01
41	I	-.171E+02	.292E+01	-.137E+01	.780E-06	.205E-05	.321E-05
	J	.171E+02	-.292E+01	.137E+01	-.780E-06	.410E+01	.377E+01
42	I	-.141E+02	-.271E+01	.436E-01	-.108E-06	.557E-06	-.761E-06
	J	.141E+02	.271E+01	-.436E-01	.108E-06	-.131E+00	-.314E+01

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STRUCTURE DATA

NUMBER OF STRUCTURE NODES 57
NUMBER OF MATERIAL GROUPS.... 1
NUMBER OF LOADING CONDITIONS... 1

BEAM FORCES AND MOMENTS

ELEM	NODE	AXIAL	SH-Y	SH-Z	TWIST	BM-Y	BM-Z
1	I	-.317E+01	.140E+02	.000E+00	-.000E+00	.000E+00	-.315E+01
	J	.317E+01	.280E+02	.000E+00	.000E+00	.000E+00	-.178E+02
2	I	-.355E+00	.185E+02	-.000E+00	.000E+00	.000E+00	.572E+01
	J	.355E+00	.235E+02	.000E+00	.000E+00	-.000E+00	-.132E+02
3	I	-.891E+00	.142E+02	-.000E+00	.000E+00	.000E+00	.205E+01
	J	.891E+00	.278E+02	.000E+00	.000E+00	.000E+00	-.223E+02
4	I	-.268E+01	.203E+02	.000E+00	.000E+00	.000E+00	.106E+02
	J	.268E+01	.217E+02	.000E+00	.000E+00	.000E+00	-.127E+02
5	I	.134E+02	.000E+00	.417E+02	.000E+00	-.270E+02	.0000+00
	J	-.134E+02	-.000E+00	.383E+02	.000E+00	.182E+02	.000E+00
6	I	.165E+02	.000E+00	.472E+02	.000E+00	-.226E+02	.000E+02
	J	-.165E+02	-.000E+00	.400E+02	.000E+00	.225E+02	.000E+00
7	I	.172E+01	.000E+00	.381E+02	.000E+00	-.181E+02	.000E+00
	J	-.172E+01	-.000E+00	.419E+02	.000E+00	.275E+02	.000E+00
8	I	.330E+01	.000E+00	.367E+02	.000E+00	-.171E+02	.000E+00
	J	-.330E+01	-.000E+00	.433E+02	.000E+00	.336E+02	.000E+00
9	I	-.914E+01	-.000E+00	.458E+02	.000E+00	-.397E+02	-.000E+00
	J	.914E+01	.000E+00	.342E+02	.000E+00	.106E+02	-.000E+00
10	I	.202E+02	.242E+01	-.000E+00	.000E+00	.000E+00	-.238E+02
	J	-.202E+02	.396E+02	.000E+00	.000E+00	.000E+00	-.319E+02
11	I	.529E+02	.632E+01	-.000E+00	.000E+00	.000E+00	-.[illegible]E+02

	J	-.529E+02	.357E+02	.000E+00	.000E+00	.000E+00	-.531E+02
12	I	.388E+02	.453E+01	-.000E+00	.000E+00	.000E+00	-.118E+02
	J	-.388E+02	.375E+02	.000E+00	.000E+00	.000E+00	-.576E+02
13	I	.254E+02	.119E+02	-.000E+00	.000E+00	.000E+00	-.337E+00
	J	-.254E+02	.301E+02	.000E+00	.000E+00	.000E+00	-.270E+02
14	I	-.259E+02	.000E+00	.467E+02	.000E+00	-.376E+02	.000E+00
	J	.259E+02	.000E+00	.333E+02	.000E+00	.401E+01	.000E+00
15	I	-.146E+02	.000E+00	.474E+02	.000E+00	-.444E+02	.000E+00
	J	.146E+02	.000E+00	.326E+02	.000E+00	.758E+01	.000E+00
16	I	.984E+01	.000E+00	.457E+02	.000E+00	-.338E+02	.000E+00
	J	-.984E+01	.000E+00	.343E+02	.000E+00	.535E+01	.000E+00
17	I	-.192E+01	-.664E+01	.000E+00	.000E+00	.000E+00	-.341E+02
	J	.192E+01	.486E+02	.000E+00	.000E+00	.000E+00	-.488E+02
18	I	.945E+01	-.385E+01	.000E+00	.000E+00	.000E+00	-.180E+02
	J	-.945E+01	.458E+02	.000E+00	.000E+00	.000E+00	-.565E+02
19	I	-.142E+02	.000E+00	.404E+02	.000E+00	-.108E+02	.000E+00
	J	.142E+02	.000E+00	.486E+02	.000E+00	.538E+02	.000E+00
20	I	-.245E+02	-.000E+00	.325E+02	.000E+00	-.140E+02	.000E+00
	J	.245E+02	.000E+00	.475E+02	.000E+00	.514E+02	.000E+00
21	I	-.341E+02	.000E+00	.688E+02	.000E+00	-.105E+03	.000E+00
	J	.341E+02	.000E+00	.112E+02	.000E+00	-.386E+02	.000E+00
22	I	.137E+02	-.324E+02	.000E+00	.000E+00	.000E+00	-.374E+02
	J	-.137E+02	.744E+02	.000E+00	.000E+00	.000E+00	-.123E+03
23	I	.588E+01	.934E+02	.000E+00	.000E+00	.000E+00	.164E+03
	J	-.588E+01	-.514E+02	.000E+00	.000E+00	.000E+00	.535E+02
24	I	.282E+01	-.000E+00	.574E+02	.000E+00	-.619E+02	-.000E+00
	J	-.282E+01	.000E+00	.546E+02	.000E+00	.522E+02	-.000E+00
25	I	-.307E+02	-.000E+00	.561E+02	.000E+00	-.627E+02	-.000E+00
	J	.307E+02	.000E+00	.559E+02	.000E+00	.619E+02	-.000E+00
26	I	-.451E+02	.000E+00	.401E+02	.000E+00	.174E+02	.000E+00
	J	.451E+02	.000E+00	.719E+02	.000E+00	.937E+02	.000E+00
27	I	.160E+01	-.542E+01	.000E+00	.000E+00	.000E+00	-.705E+01
	J	-.160E+01	.542E+01	.000E+00	.000E+00	.000E+00	-.921E+01
28	I	.383E+01	-.712E+01	.000E+00	.000E+00	.000E+00	-.705E+01

	J	-.383E+01	.712E+01	.000E+00	.000E+00	.000E+00	-.143E+02
29	I	.277E+02	.170E+02	.186E+00	.000E+00	.553E+00	.228E+02
	J	-.277E+02	-.170E+02	-.186E+00	.000E+00	-.111E+01	.231E+02
30	I	-.646E+01	.139E+02	.756E+01	.000E+00	-.111E+02	.188E+02
	J	.646E+01	-.139E+02	-.756E+01	.000E+00	-.116E+02	.229E+02
31	I	.390E+00	.887E+01	.494E+01	.000E+00	-.488E+01	.112E+02
	J	-.390E+00	-.887E+01	-.494E+01	.000E+00	-.995E+01	.154E+02
32	I	-.114E+02	.475E+01	.690E+01	.000E-00	-.105E+02	.631E+01
	J	.114E+02	-.475E+01	-.690E+01	.000E-00	-.102E+02	.793E+01
33	I	-.675E+02	-.187E+02	.815E+01	.000E+00	-.142E+02	-.268E+02
	J	.675E+02	.187E+02	-.815E+01	.000E+00	-.103E+02	-.293E+02
34	I	.358E+02	.107E+02	-.169E+02	.000E+00	.248E+02	.149E+02
	J	-.358E+02	-.107E+02	.169E+02	.000E+00	.258E+02	.171E+02
35	I	-.592E+01	.132E+02	-.279E+02	.000E+00	.403E+02	.178E+02
	J	.592E+01	-.132E+02	.279E+02	.000E+00	.434E+02	.217E+02
36	I	.115E+01	.132E+02	-.272E+02	.000E+00	.389E+02	.157E+02
	J	-.115E+01	-.132E+02	.272E+02	.000E+00	.426E+02	.240E+02
37	I	-.715E+01	.148E+02	-.252E+02	.000E+00	.371E+02	.195E+02
	J	.715E+01	-.148E+02	.252E+02	.000E+00	.387E+02	.249E+02
38	I	-.608E+02	.456E+01	-.148E+02	.000E+00	.187E+02	.118E+02
	J	.608E+02	-.456E+01	.148E+02	.000E+00	.257E+02	.191E+01
39	I	.385E+02	.692E+00	-.243E+02	.000E+00	.316E+02	.473E+01
	J	-.385E+02	-.692E+00	.243E+02	.000E+00	.413E+02	-.266E+01
40	I	-.112E+02	.832E+01	-.343E+02	.000E+00	.484E+02	.143E+02
	J	.112E+02	-.832E+01	.343E+02	.000E+00	.544E+02	.107E+02
41	I	-.118E+03	.331E+02	-.182E+02	.000E+00	.248E+02	.350E+02
	J	.118E+03	-.331E+02	.182E+02	.000E+00	.297E+02	.642E+02
42	I	.138E+03	-.422E+01	-.188E+02	.000E+00	.226E+02	-.655E+01
	J	-.138E+03	.422E+01	.188E+02	.000E+00	.337E+02	-.610E+01
43	I	-.642E+02	.451E+01	.381E+00	.000E-00	.162E+02	.561E+01
	J	.642E+02	-.451E+01	-.381E+00	.000E-00	-.173E+02	.795E+01
44	I	.158E+02	.725E-01	-.254E+00	.000E+00	.130E+01	-.485E+00
	J	-.158E+02	-.725E-01	.254E+00	.000E+00	.130E+01	-.485E+00
45	I	.600E+02	.277E+02	-.358E+01	.000E+00	.371E+01	.387E+02

	J	-.600E+02	-.277E+02	.358E+01	.000E+00	.702E+01	.444E+02
46	I	.576E+02	.238E+02	-.455E+01	.000E+00	.771E+01	.358E+02
	J	-.576E+02	-.238E+02	.455E+01	.000E+00	.593E+01	.345E+02
47	I	-.790E+02	-.285E+02	.303E+01	.000E+00	-.297E+00	-.271E+02
	J	.790E+02	.285E+02	-.303E+01	.000E+00	-.880E+01	-.585E+02
48	I	.557E+01	.818E+01	.000E+00	.000E+00	.000E+00	-.131E+02
	J	-.557E+01	.338E+02	.000E+00	.000E+00	.000E+00	-.254E+02
49	I	.535E+01	.766E+01	.000E+00	.000E+00	.000E+00	-.596E+01
	J	-.535E+01	.343E+02	.000E+00	.000E+00	.000E+00	-.340E+02
50	I	.258E+01	.425E+01	.000E+00	.000E+00	.000E+00	-.183E+02
	J	-.258E+01	.378E+02	.000E+00	.000E+00	.000E+00	-.319E+02
51	I	.722E+00	.402E+02	.000E+00	.000E+00	.000E+00	.336E+02
	J	-.722E+00	.438E+02	.000E+00	.000E+00	.000E+00	-.442E+02
52	I	-.110E+02	.000E+00	.439E+02	.000E+00	-.369E+02	.000E+00
	J	.110E+02	.000E+00	.361E+02	.000E+00	.173E+02	.000E+00
53	I	-.969E+01	.000E+00	.401E+02	.000E+00	-.278E+02	.000E+00
	J	.969E+01	.000E+00	.399E+02	.000E+00	.271E+02	.000E+00
54	I	-.175E+02	.000E+00	.352E+02	.000E+00	-.184E+02	.000E+00
	J	.175E+02	.000E+00	.448E+02	.000E+00	.422E+02	.000E+00
55	I	.456E+01	.000E+00	.339E+02	.000E+00	-.144E+02	.000E+00
	J	-.456E+01	.000E+00	.461E+02	.000E+00	.448E+02	.000E+00
56	I	.188E+02	.000E+00	.406E+02	.000E+00	-.341E+02	.000E+00
	J	-.188E+02	.000E+00	.394E+02	.000E+00	.310E+02	.000E+00
57	I	.381E+01	.497E+02	.000E+00	.000E+00	.000E+00	.579E+02
	J	-.381E+01	-.774E+01	-.000E+00	.000E+00	.000E+00	.283E+02
58	I	-.493E-01	.491E+02	.000E+00	.000E+00	.000E+00	.515E+02
	J	.493E-01	-.710E+01	.000E+00	.000E+00	.000E+00	.328E+02
59	I	-.334E+01	.486E+02	.000E+00	.000E-00	.000E+00	.515E+02
	J	.334E+01	-.657E+01	-.000E+00	.000E-00	.000E+00	.512E+02
60	I	-.427E+01	.497E+02	.000E+00	.000E+00	.000E+00	.454E+02
	J	.427E+01	-.770E+01	.000E+00	.000E+00	.000E+00	.407E+02
61	I	-.133E+02	.000E+00	.380E+02	.000E+00	-.275E+02	.000E+00
	J	.133E+02	.000E+00	.420E+02	.000E+00	.375E+02	.000E+00
62	I	.181E+02	.000E+00	.365E+02	.000E+00	-.242E+02	.000E+00

	J	-.181E+02	.000E+00	.435E+02	.000E+00	.416E+02	.000E+00
63	I	.305E+02	.000E+00	.463E+02	.000E+00	-.458E+02	.000E+00
	J	-.305E+02	.000E+00	.337E+02	.000E+00	.142E+02	.000E+00
64	I	-.473E+00	.144E+02	.000E+00	.000E+00	.000E+00	-.242E+01
	J	.473E+00	.276E+02	.000E+00	.000E+00	.000E+00	.173E+02
65	I	-.527E+01	.190E+02	.000E+00	.000E+00	.000E+00	.744E+01
	J	.527E+01	.230E+02	.000E+00	.000E+00	.000E+00	-.135E+02
66	I	-.305E+02	.000E+00	.354E+02	.000E+00	-.256E+02	.000E+00
	J	.305E+02	.000E+00	.446E+02	.000E+00	.485E+02	.000E+00
67	I	-.218E+02	.000E+00	.339E+02	.000E+00	-.210E+02	.000E+00
	J	.218E+02	.000E+00	.461E+02	.000E+00	.515E+02	.000E+00
68	I	.429E+02	.000E+00	.389E+02	.000E+00	-.381E+02	.000E+00
	J	-.429E+02	.000E+00	.411E+02	.000E+00	.436E+02	.000E+00
69	I	-.181E+02	.344E+01	.000E+00	.0000+00	.000E+00	-.283E+02
	J	.181E+02	.386E+02	.000E+00	.000E+00	.000E+00	-.244E+02
70	I	-.118E+02	.337E+02	.000E+00	.000E+00	.000E+00	.218E+02
	J	.118E+02	.835E+01	.000E+00	.000E+00	.000E+00	.161E+02
71	I	-.300E+02	.000E+00	.554E+02	.000E+00	-.572E+02	.000E+00
	J	.300E+02	.000E+00	.566E+02	.000E+00	.614E+02	.000E+00
72	I	-.174E+02	.000E+00	.557E+02	.000E+00	-.592E+02	.000E+00
	J	.174E+02	.000E+00	.563E+02	.000E+00	.615E+02	.000E+00
73	I	.224E+02	.000E+00	.577E+02	.000E+00	-.646E+02	.000E+00
	J	-.224E+02	.000E+00	.543E+02	.000E+00	.527E+02	.000E+00
74	I	-.969E+01	.699E+01	.000E+00	.000E+00	.000E+00	-.134E+02
	J	.969E+01	.350E+02	.000E+00	.000E+00	.000E+00	-.286E+02
75	I	-.863E+01	.212E+02	.000E+00	.000E+00	.000E+00	.114E+02
	J	.863E+01	.208E+02	.000E+00	.000E+00	.000E+00	-.107E+02
76	I	.635E+02	.749E+01	.716E+01	.000E+00	-.167E+02	.138E+02
	J	-.635E+02	-.749E+01	-.716E+01	.360E+00	-.119E+02	.161E+02
77	I	-.779E+01	.317E+01	.965E+01	.000E+00	-.201E+02	.426E+01
	J	.779E+01	-.317E+01	-.965E+01	.000E+00	-.185E+02	.842E+01
78	I	-.432E+02	.481E+00	.770E+01	.000E+00	-.175E+02	-.179E+01
	J	.432E+02	-.481E+00	-.770E+01	.000E+00	-.133E+02	.571E+01
79	I	-.152E+02	.196E+01	.980E+01	.000E+00	-.203E+02	.729E+00

J	.152E+02	-.196E+01	-.980E+01	.000E+00	-.189E+02	.713E+01
80 I	-.152E+03	-.214E+01	.112E+02	.000E+00	-.221E+02	-.420E+01
J	.152E+03	.214E+01	-.112E+02	.000E+00	-.225E+02	-.436E+01
81 I	.122E+03	-.148E+01	-.193E+02	.000E+00	.423E+02	-.574E+01
J	-.122E+03	.148E+01	.193E+02	.000E+00	.348E+02	-.173E+00
82 I	.753E+02	.248E+01	-.217E+02	.000E+00	.456E+02	.336E+01
J	-.753E+02	-.248E+01	.217E+02	.000E+00	.413E+02	.658E+01
83 I	.125E+03	.258E+01	-.211E+02	.000E+00	.448E+02	.602E+01
J	-.125E+03	-.258E+01	.211E+02	.000E+00	.396E+02	.431E+01
84 I	.119E+03	.149E+01	-.209E+02	.000E+00	.445E+02	.384E+01
J	-.119E+03	-.149E+01	.209E+02	.000E+00	.391E+02	.212E+01
85 I	-.154E+03	.119E+01	-.163E+02	.000E+00	.383E+02	.278E+01
J	.154E+03	-.119E+01	.163E+02	.000E+00	.269E+02	.199E+01
86 I	.147E+03	.353E+01	-.174E+02	.000E+00	.423E+02	.739E+01
J	-.147E+03	-.353E+01	.174E+02	.000E+00	.273E+02	.672E+01
87 I	.613E+02	.169E+01	-.212E+02	.000E+00	.474E+02	.395E+01
J	-.613E+02	-.169E+01	.212E+02	.000E+00	.376E+02	.279E+01
88 I	-.216E+03	-.232E+01	-.169E+02	.000E+00	.416E+02	-.216E+01
J	.216E+03	.232E+01	.169E+02	.000E+00	.260E+02	-.711E+01
89 I	.235E+03	-.459E+00	.521E+01	.000E+00	-.155E+02	-.355E+01
J	-.235E+03	.459E+00	-.521E+01	.000E+00	-.531E+01	.172E+01
90 I	.327E+02	.787E+00	.121E+02	.000E+00	-.246E+02	-.834E+00
J	-.327E+02	-.787E+00	-.121E+02	.000E+00	-.237E+02	.398E+01
91 I	-.902E+02	-.417E+01	.769E+01	.000E+00	-.187E+02	-.614E+01
J	.902E+02	.417E+01	-.769E+01	.000E+00	-.121E+02	-.105E+02
92 I	.109E+03	.887E+01	-.396E+01	.000E+00	.101E+02	.151E+02
J	-.109E+03	-.887E+01	.396E+01	.000E+00	.578E+01	.204E+02
93 I	.576E+02	.607E+01	-.554E+01	.000E+00	.121E+02	.102E+02
J	-.576E+02	-.607E+01	.554E+01	.000E+00	.100E+02	.141E+02
94 I	-.154E+03	-.499E+01	-.517E+01	.000E+00	.116E+02	-.633E+01
J	.154E+03	.499E+01	.517E+01	.000E+00	.909E+01	-.136E+02

CHAPTER VI

COMPARISON OF THE RESULTS OF THE ANALYSES

6.1 Introduction:

The results of the three approaches made for the four types of frames are compared here.

6.2 Comparison of the Results:

The variations of the maximum bending moment and average bending moments in beams, average column moments and average axial loads in columns, in each storey, of all frames are shown in Fig 6.1 to 6.16 for the three approaches adopted in this investigation.

It is observed from Figs 6.1, 6.5 and 6.7 that the difference in the values obtained by the three approaches with respect to maximum storey moment, average beam moment and average column moment is less for frame 1 which is a symmetrical one. But, from fig 6.6 it is clear that if the soil structure interaction is taken into account in the analysis. The average axial load in columns is more than 50 % of that obtained by conventional three dimensional analysis.

For the 2nd frame, the maximum storey moment obtained by the analysis incorporating soil-structure interaction is more by 48 % with respect to the value obtained by conventional analysis. This may be due to the unsymmetry of the structure in plan. But the average beam moments obtained by the interactive analysis are more by 60 % compared to the conventional analysis. But in this case the difference in column loads is not too much and is in the range of 26 % . The stress resultant values of approximate analysis are lying in between the values of conventional and interactive analysis, thereby showing that the approximate portal method of analysis gives design moments on the more realistic side than the values obtained by the three dimensional analysis.

The important results are also shown in Table 6.1

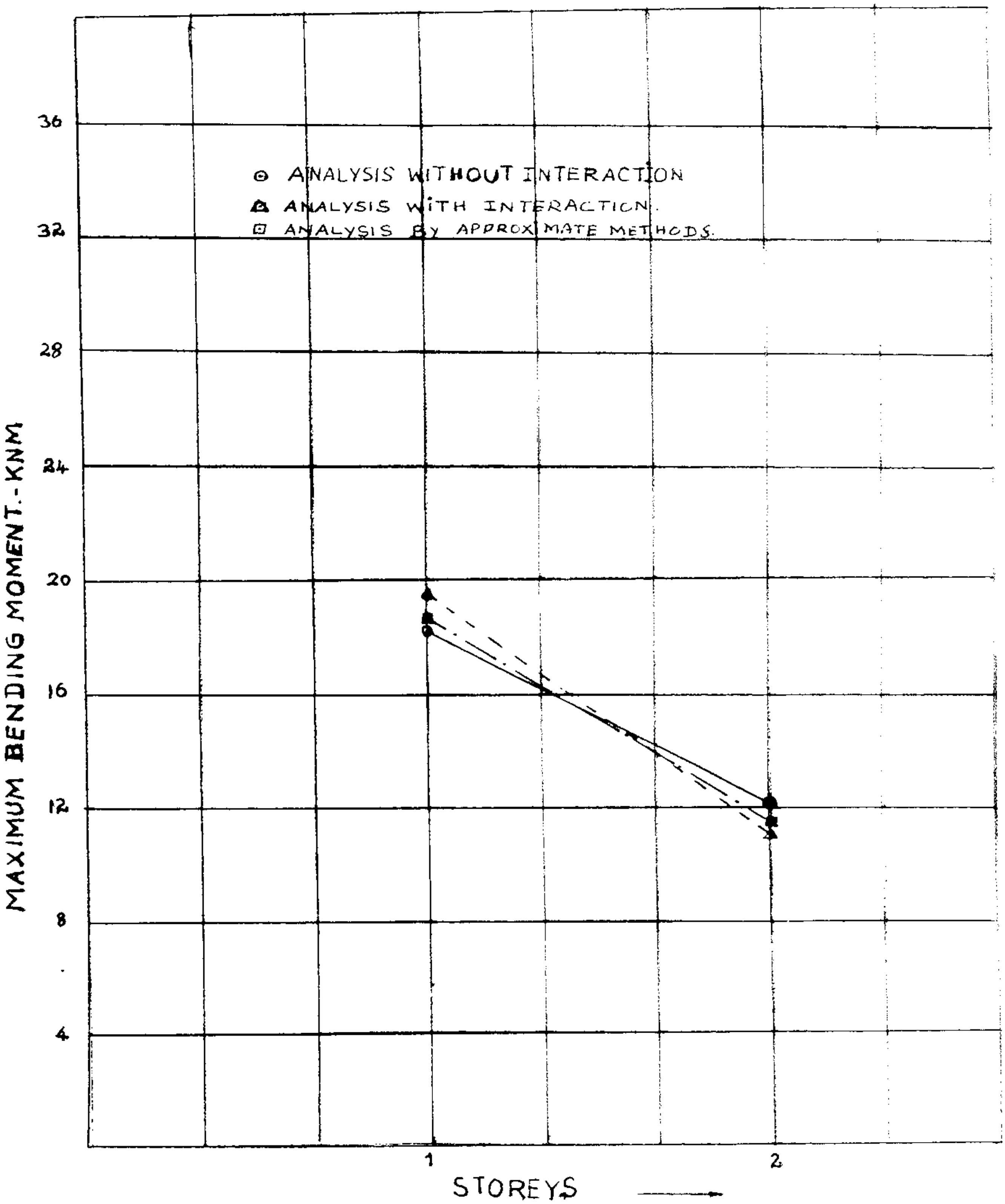

Fig 6.1 Comparison of Analyses for frame 1

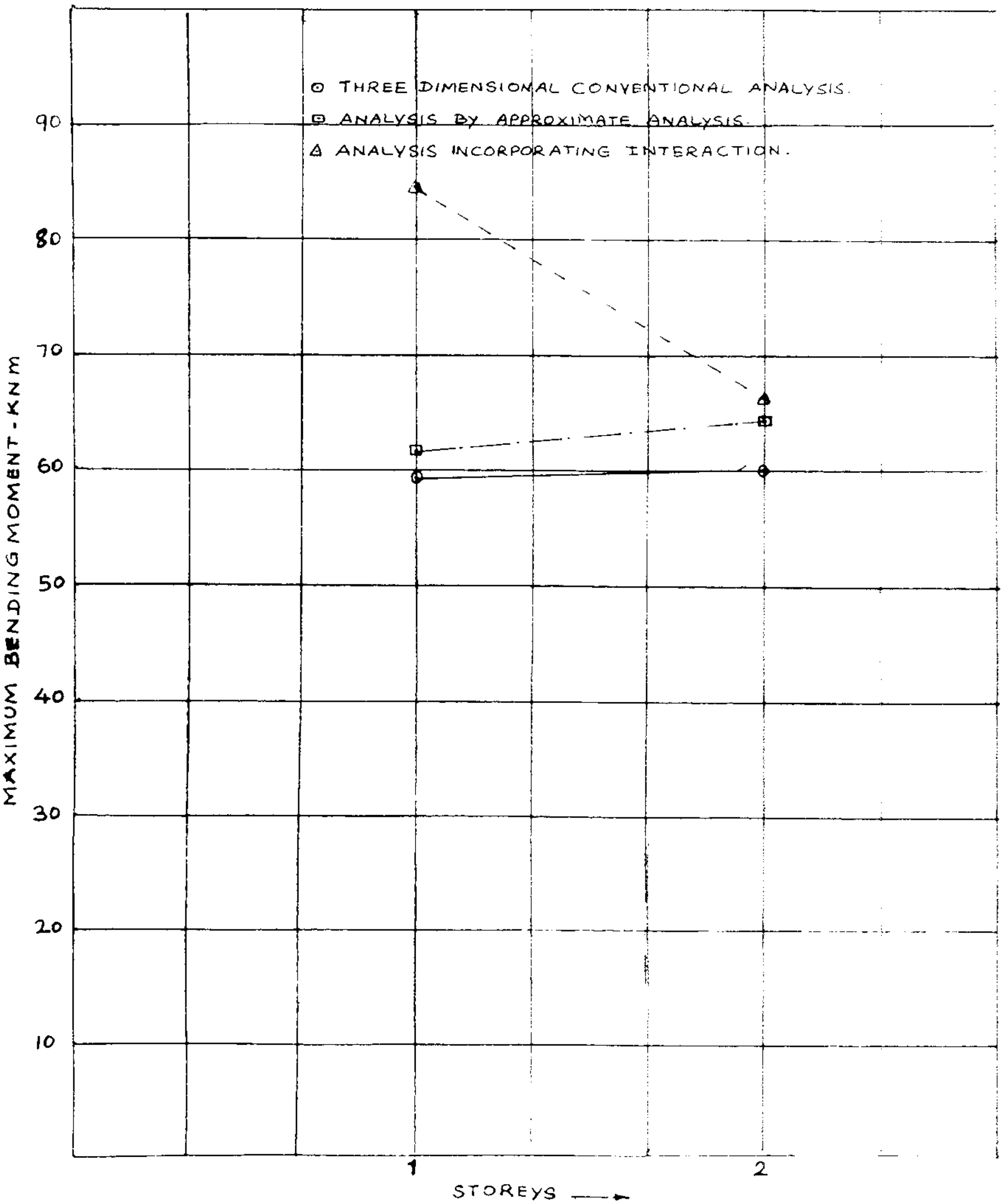

Fig 6.2 Comparison of Analyses for Frame 2

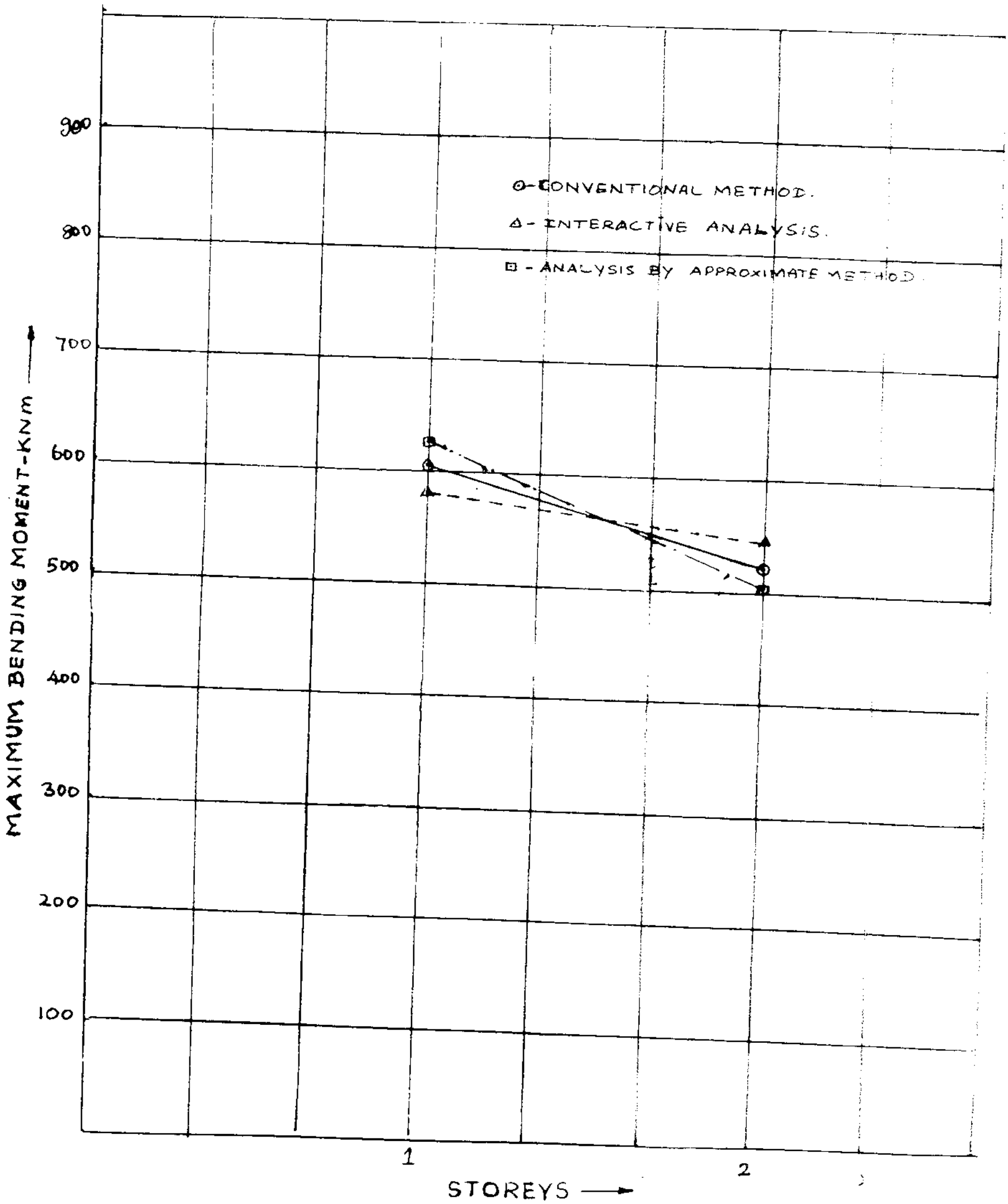

Fig 6.3 Comparison of Analyses for Frame 3

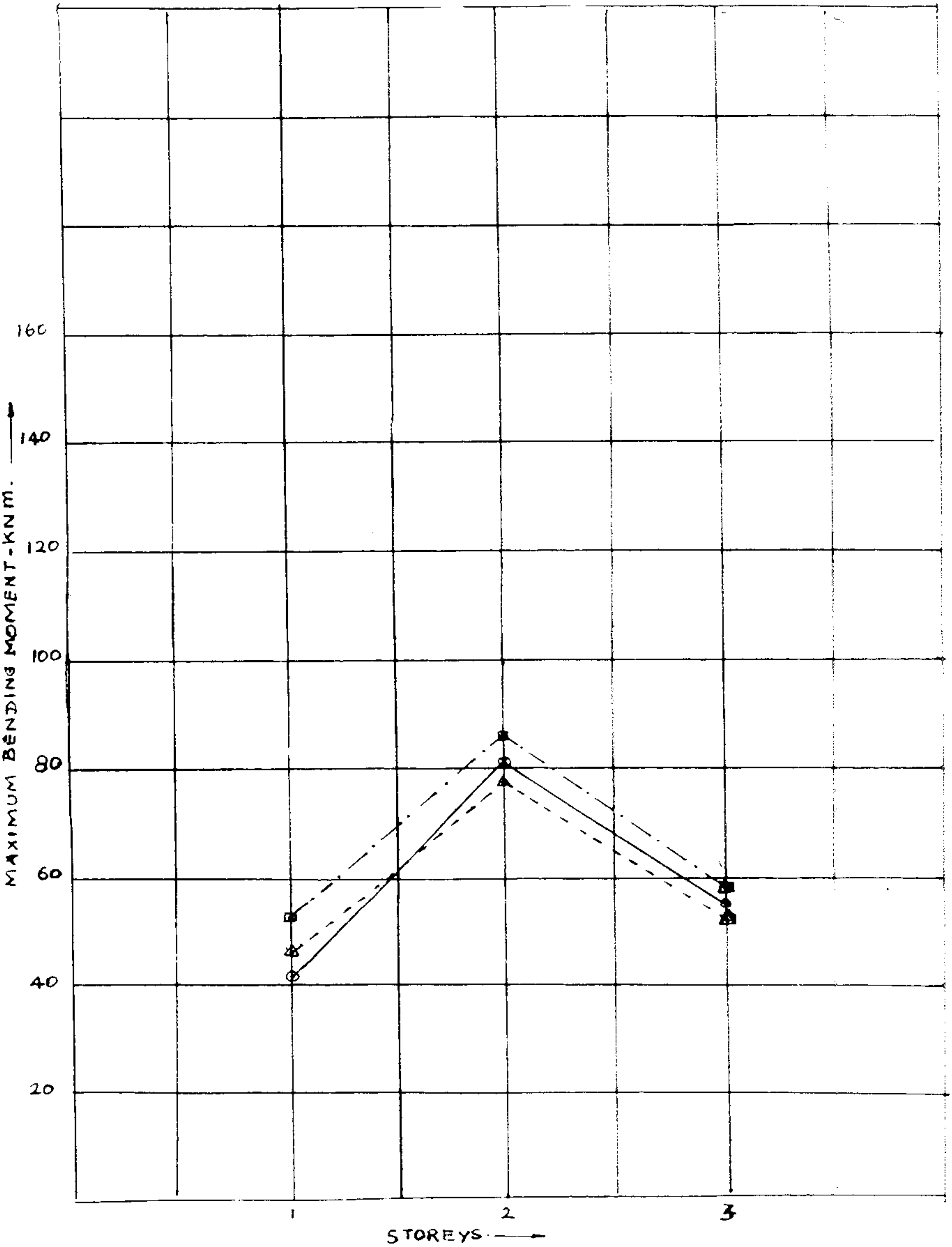

Fig 6.4 Comparison of Analyses for Frame 4

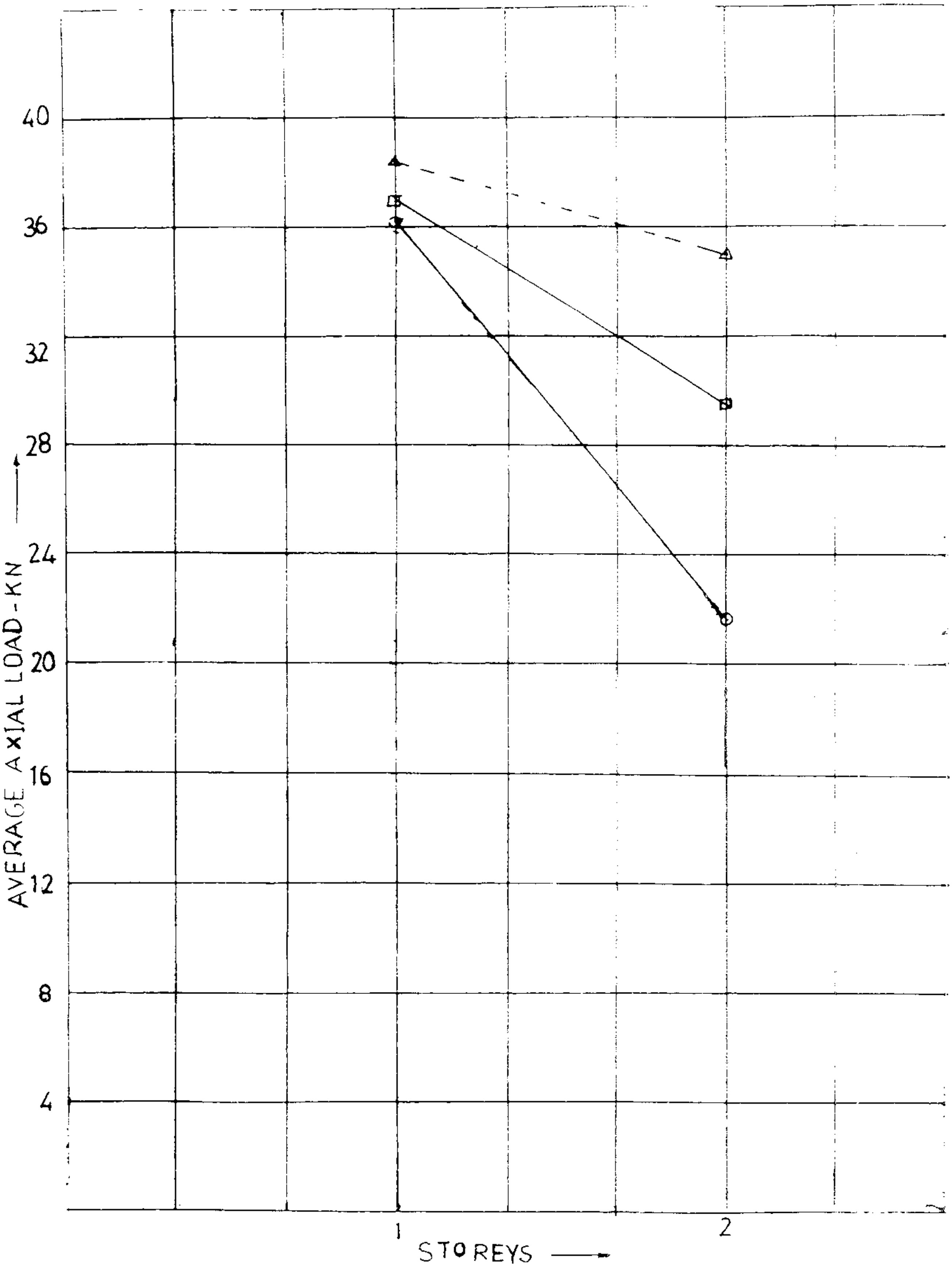

Fig 6.6 Comparison of Axial Load in columns for Frame

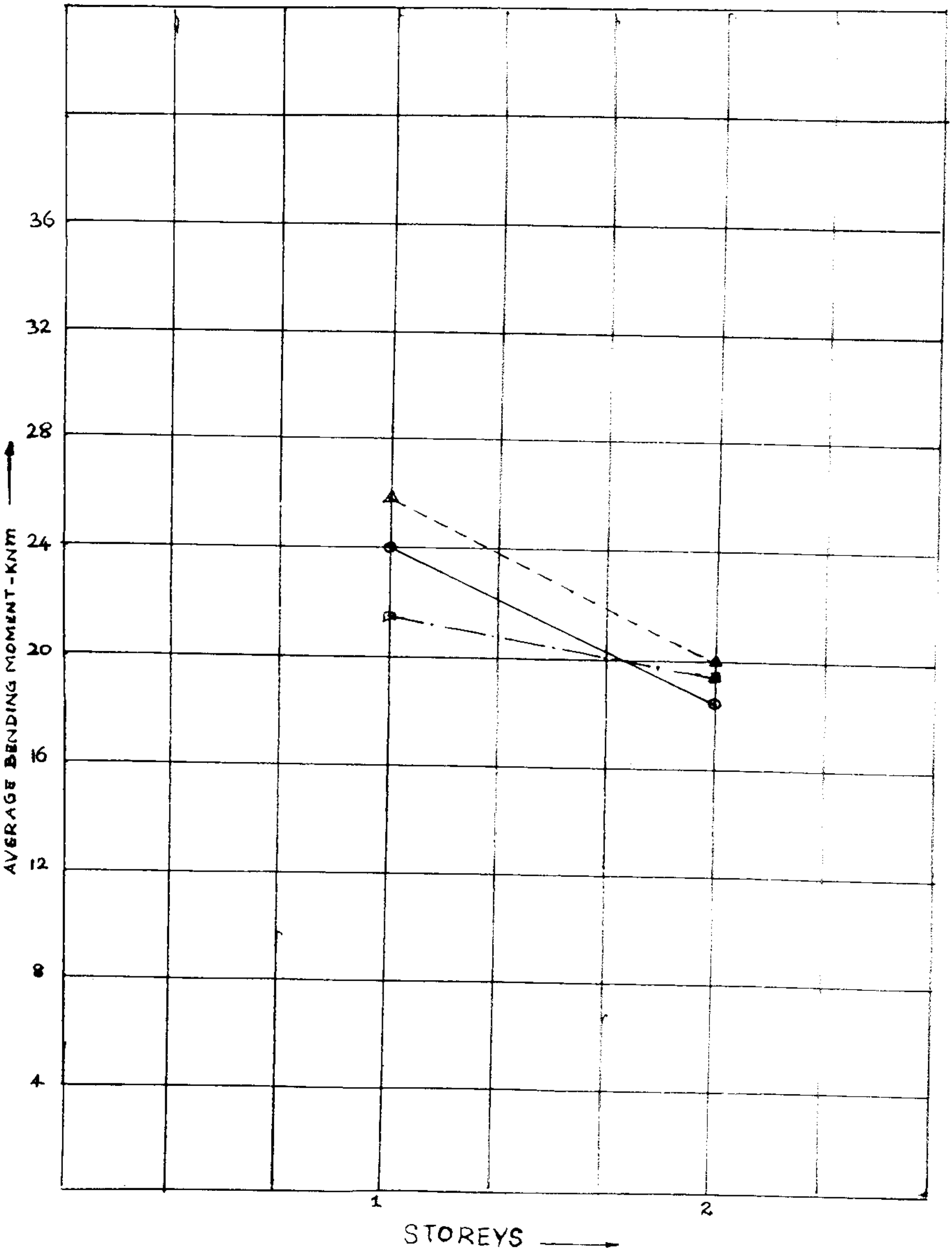

Fig 6.7 Comparison of Average column moments for Frame 1

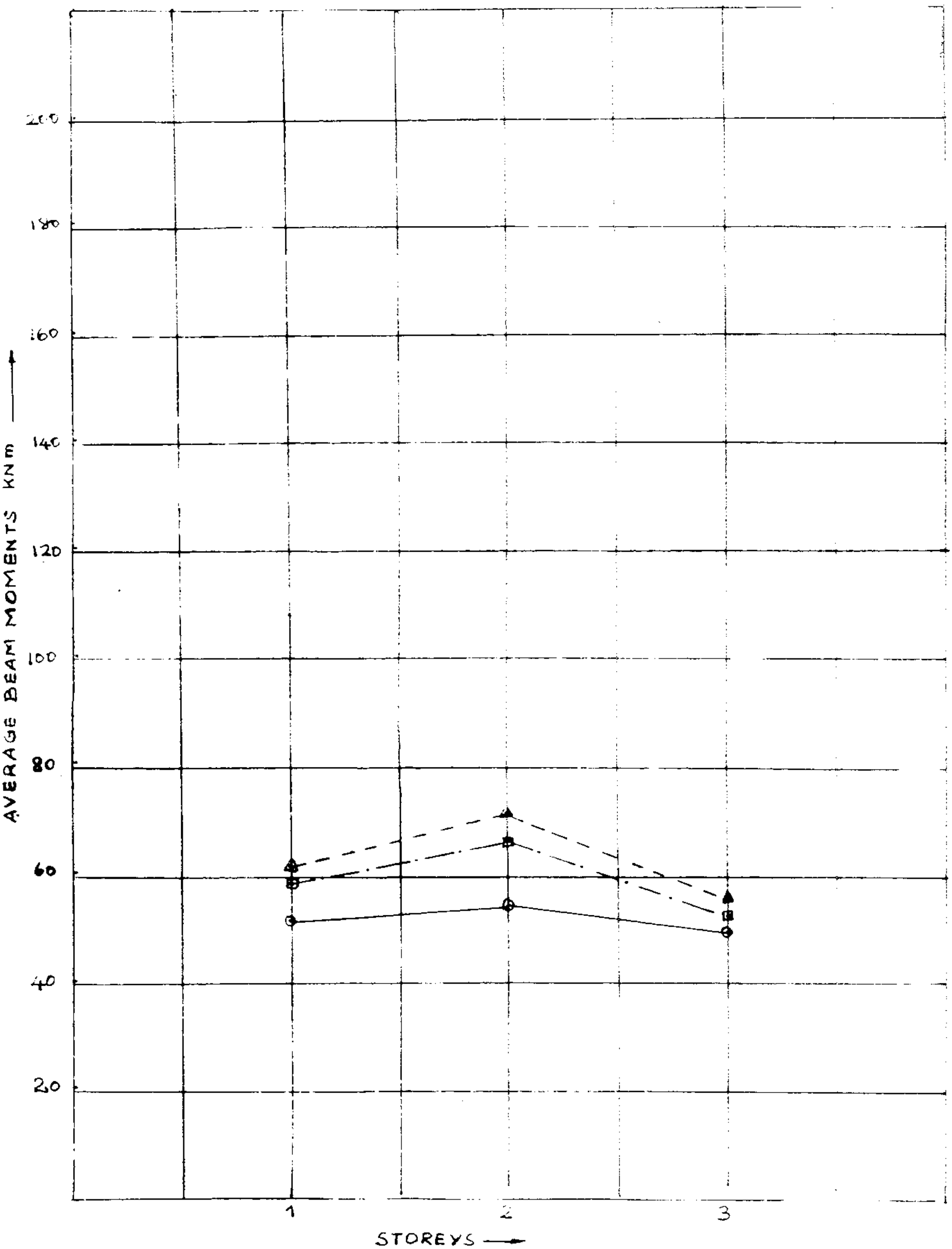

Fig 6.8 Comparison of Average Beam Moment for Frame 4

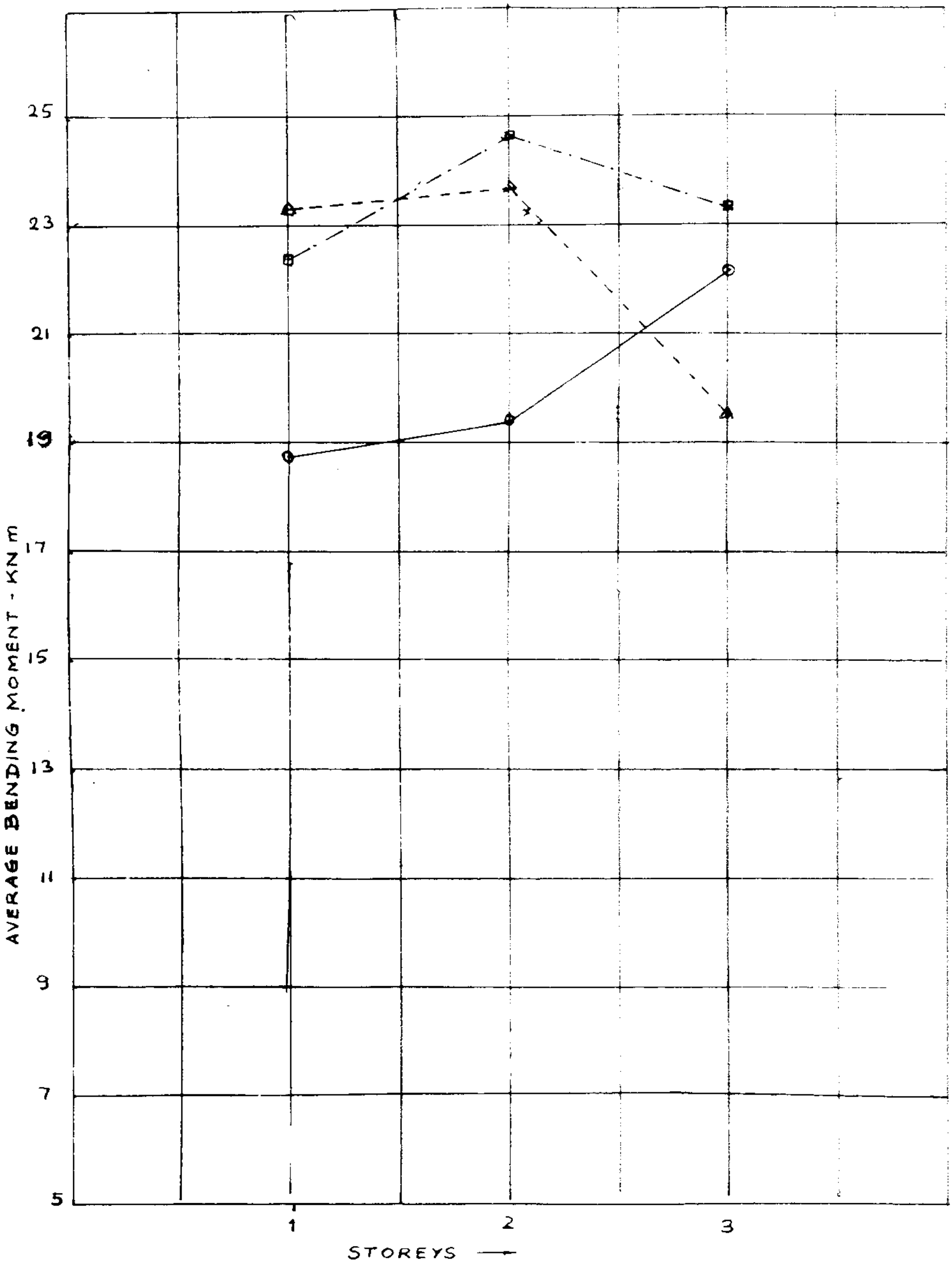

Fig 6.9 Comparison of Average column moments for frame 4

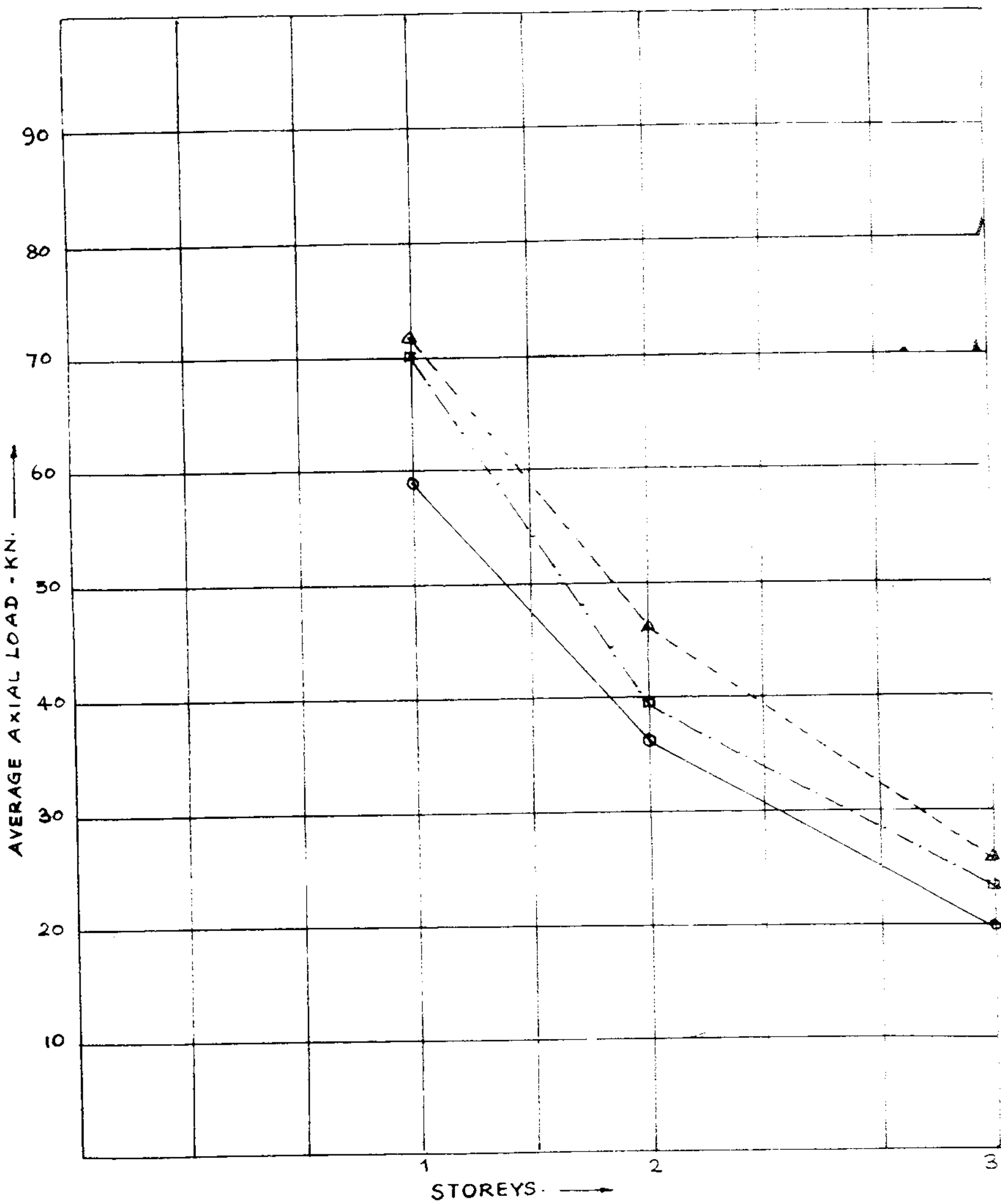

Fig 6.10 Comparison of average Common force in Frame 4

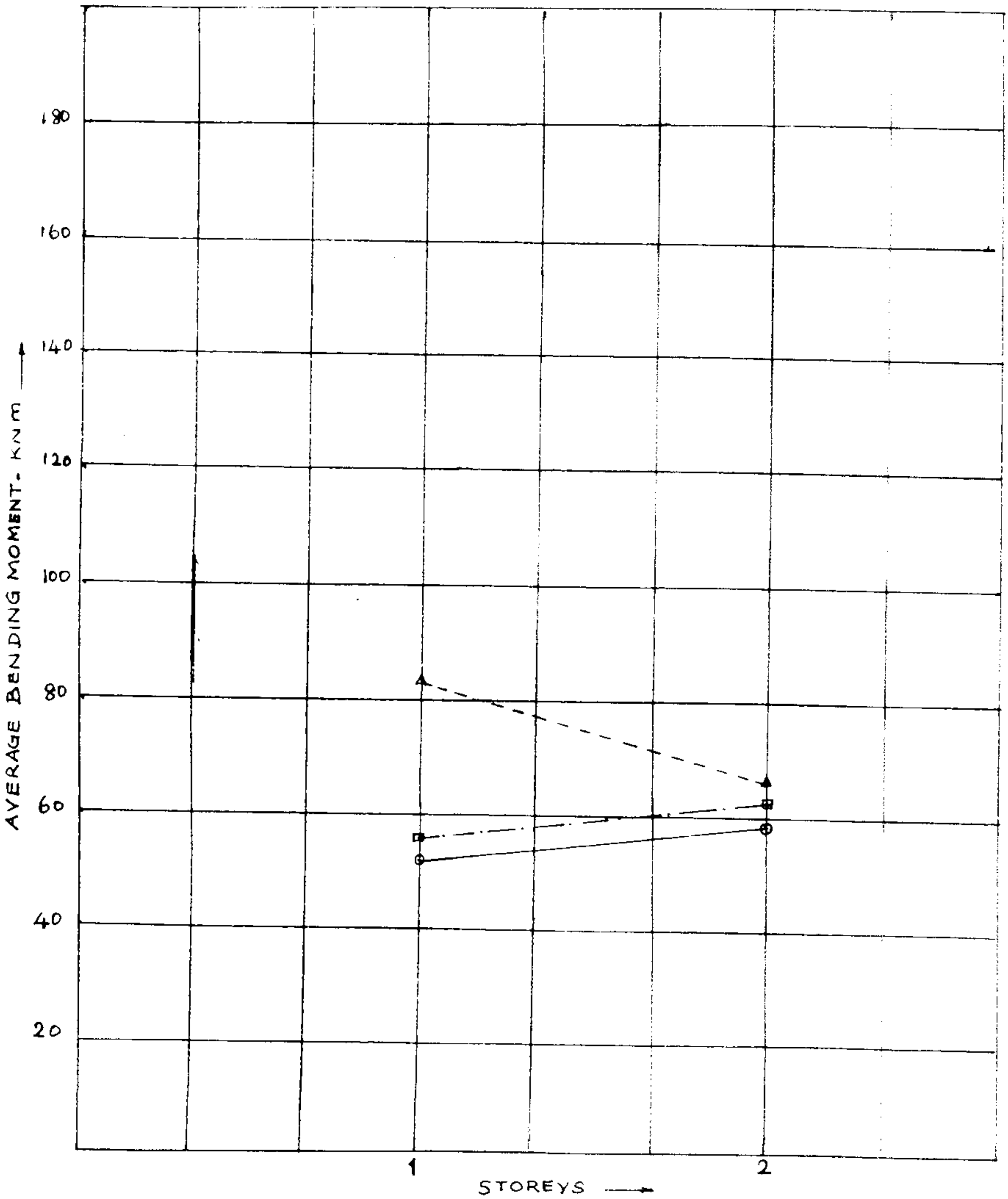

Fig 6.11 Comparison of average Beam Moments for Frame 2

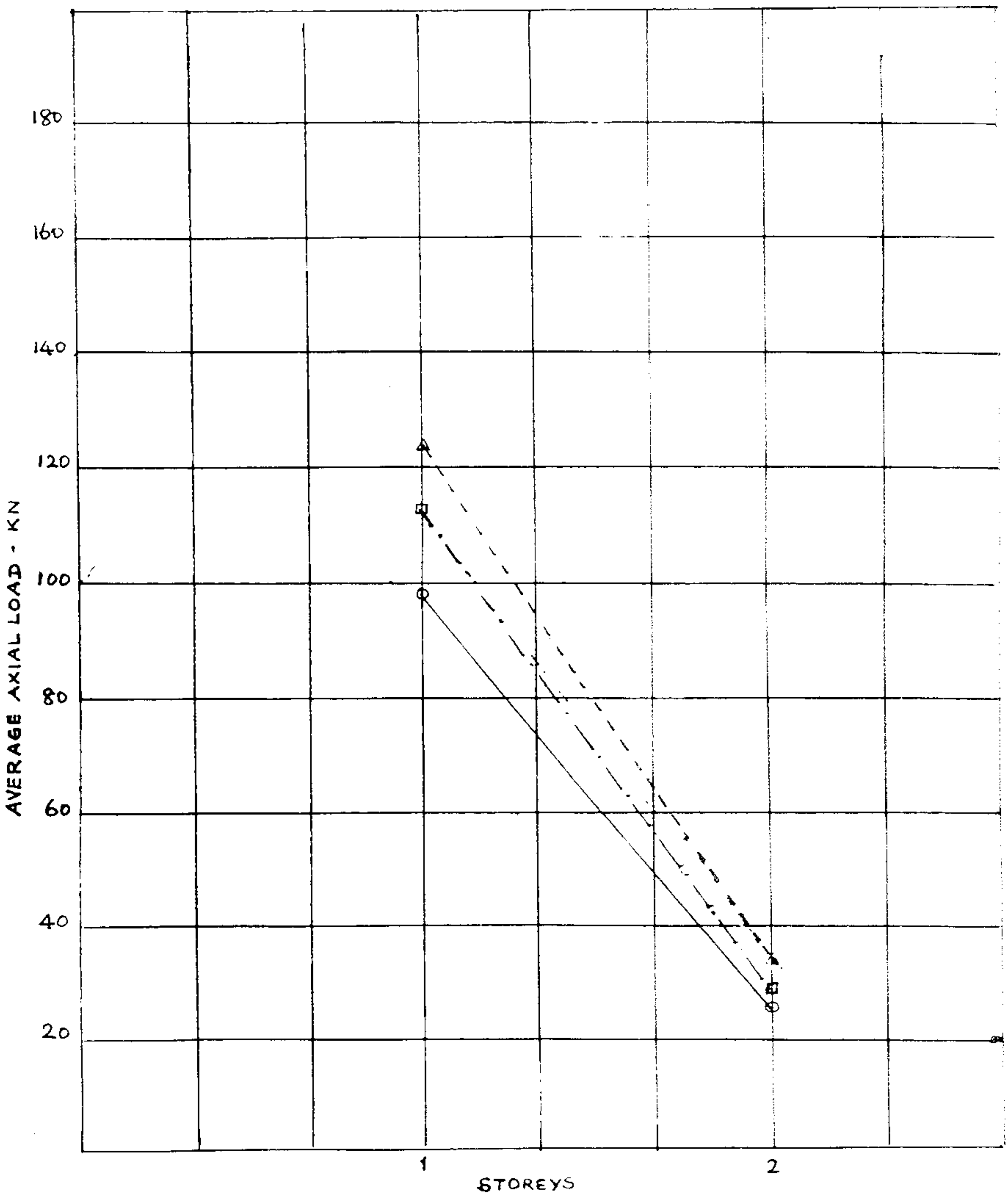

Fig 6.13 Comparison of Average Axial Load in Columns For Frame 2

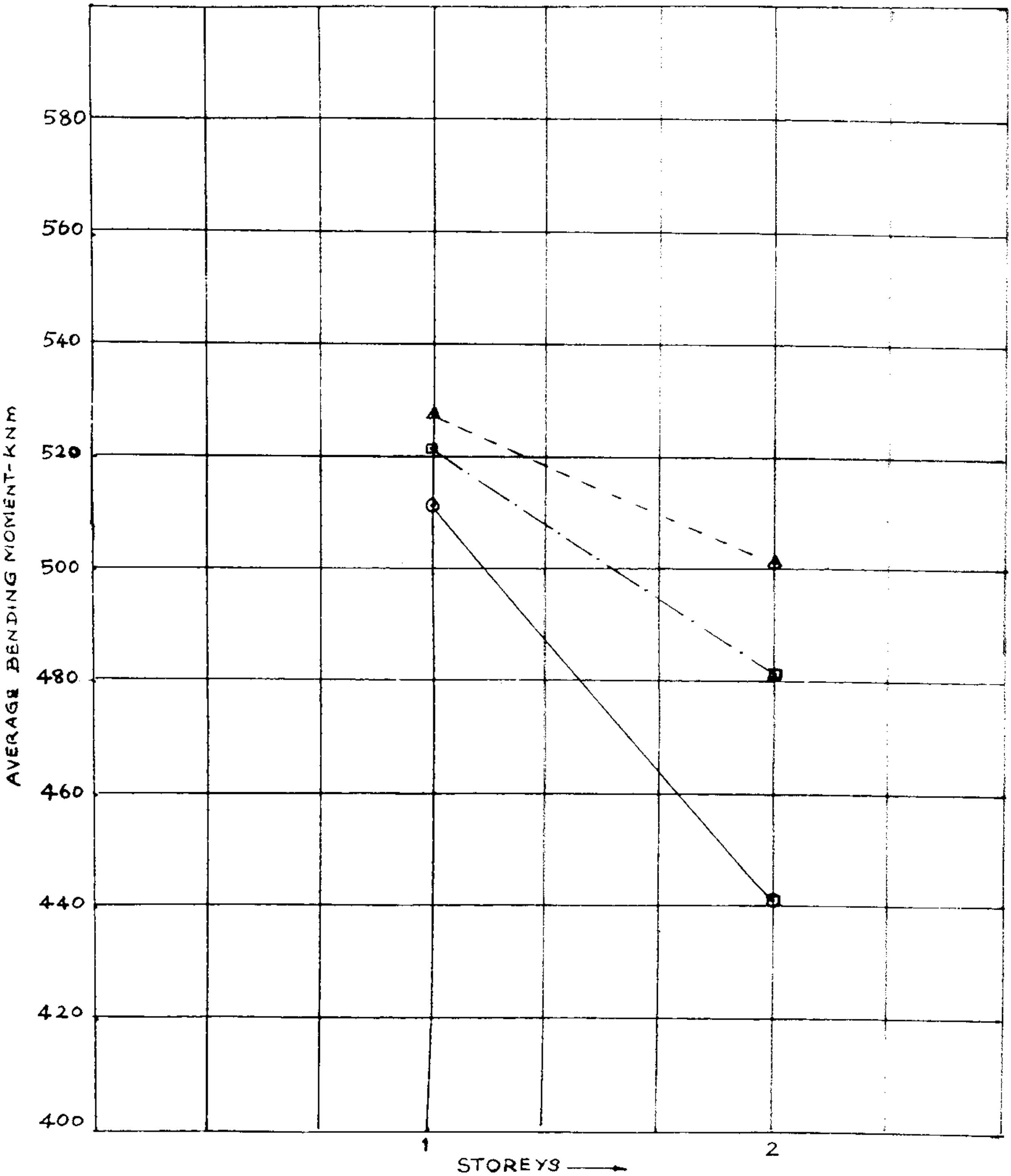

Fig 6.14 Comparison of Average Beam Moments for Frame 3

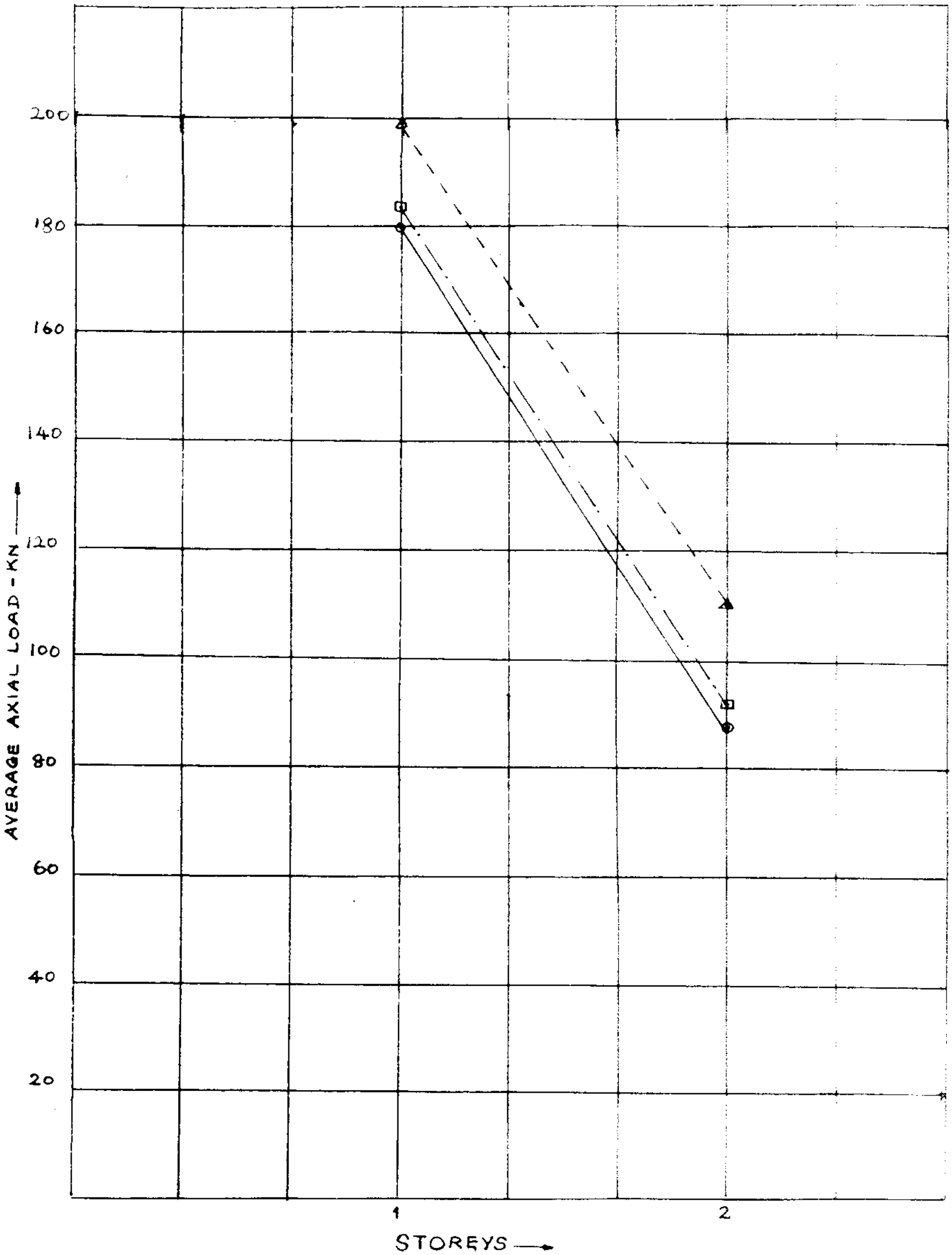

Fig 6.15 Comparison of Average Column Loads for Frame 3

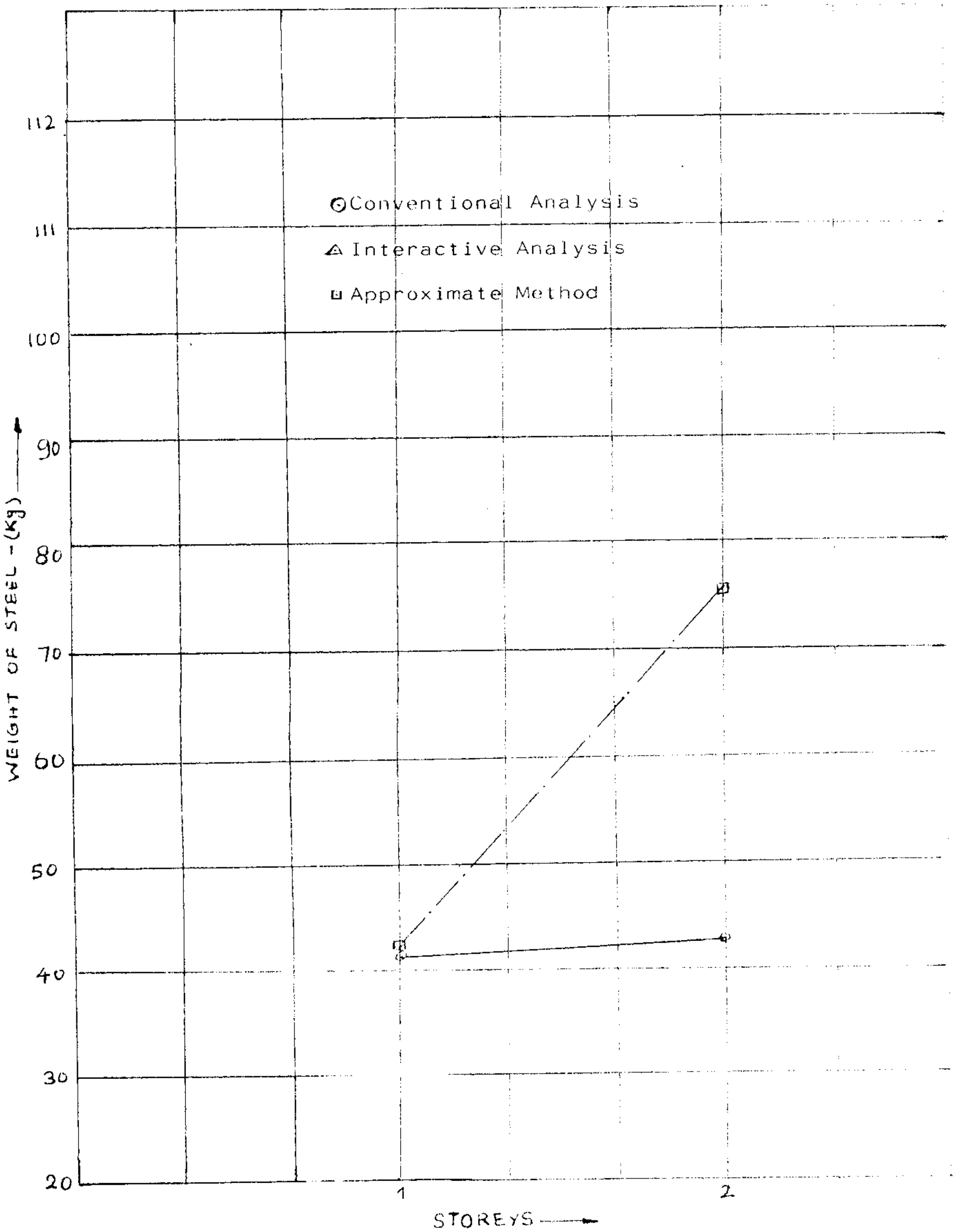

Fig. 6.17 Comparison of weight of steel for frame-2

Table 6.1 Important Results of the Three Approaches

Stress Resultants (Average)	3-D Frame Analysis	Interactive Analysis	Approximate Method
Frame 1			
Second Storey			
Beam Moment	11.20 Knm	11.95 KNM	10.95 KNm
Column Load	21.81 KN	35.09 KN	29.60
B.M.	18.4 KNm	18.9 KNM	19.27 KNm
First Storey			
Beam Moment	17.10 KNm	18.4 KNm	17.80 KNm
Column Axial	36.02 KNm	38.40	37.01
B.M	24.02 KNm	25.80	23.4
Frame 2			
Second Storey			
Beam Moment	57.91	68.21	63.12
Column Load	25.06	34.71	29.26
B.M.	36.27	49.01	42.32
First Storey			
Beam Moment	52.71	84.16	56.25
Column Load	98.43	124.10	113.97
B.M.	33.62	45.65	36.01
Frame 3			
Second Storey			
Beam Moment	482.76	502.32	443.02
Column Load	86.40	109.28	91.07
B.M.	34.52	40.10	42.17
First Storey			
Beam Moment	512.35	527.67	581.53
Column Load	179.32	198.23	183.47
B.M.	47.13	52.59	53.70
Frame 4			
Third Storey			
Beam Moment	58.72	77.36	68.53
Column Load	19.5	26.17	24.82
B.M.	22.92	20.35	24.21
Second Storey			
Beam Moment	54.25	72.35	66.72
Column Load	36.00	46.12	38.91
B.M.	20.57	24.62	25.63
First Storey			
Beam Moment	51.13	62.57	59.12
Column Load	59.20	70.25	70.00
B.M.	19.12	24.4	23.2