MANUAL ON PLANNING & DESIGN OF REINFORCED CONCRETE MULTISTOREYED BUILDING

VOL. 1

BY

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CENTRAL DESIGNS ORGANISATION

UAL ON PLANNING & DESIGN OF REINFORCED CONCRETE MULTISTOREYED BUILDINGS

Volume I

FOREWORD

Over the past 25 years, the Central Public Works Department has planned, designed and constructed a number of multi-storeyed structures for office, residential, industrial and institutional purposes. In this process the department has acquired certain expertise and experience. Many more such structures are currently being planned, designed and constructed by the Department. It was considered worthwhile to bring out a Manual on the planning and design of such structures to serve as a guide for the various design engineers of the department working in different parts of the country.

2. The Central Design Organisation of the Central P. W. D. has compiled this useful Manual which will be published in two volumes—the first volume dealing with planning and analysis aspects of multi-storeyed structures and the second volume dealing with design and detailing. I would like all the Design Engineers in the department to make full use of the Manual and make the structural design more economical and purposeful.

NEW DELHI,
the 4th May, 1976.

V. R. VAISH
Engineer-in-Chief.

PREFACE

There is an increasing trend towards the construction of multi-storeyed buildings for residential as well as non-residential purposes in urban areas. Owing to the migration of population from rural areas to cities, land in urban areas is becoming increasingly scarce and land prices are spiralling from year to year. Consequently, the trend towards high rise construction especially in larger cities is bound to increase in the years to come. It is, therefore, necessary that the methods of proper designing of such buildings are better known.

There are several standard works of reference which lay down the methods of analysis and design of multi-storeyed buildings. They are, however, not readily available to most of the engineers in the department particularly to those who do not have ready access to libraries or to bookshops where such books are available. Moreover, the various methods of design are not explained in sufficient detail in one single book, and one has to read through several different reference books before getting a working knowledge of the various methods of design available for high rise buildings.

Quite a few tall buildings have now been designed in the Central Designs Organisation, C.P.W.D. The experience gained and the knowledge acquired in the process, including examination of the recent publications and articles on the subject, are brought together in this manual which is proposed to be published in 2 volumes. This volume deals with planning, design loading, preliminary design and methods of analysis for vertical and horizontal forces. In the 2nd volume which is under preparation, the design and detailing aspects are dealt with.

It has been possible to bring out this publication on account of the efforts of Shri John Mukand, former Chief Engineer (Designs), Shri M. B. Kodnani, Shri P. K. Ratho, Shri S. S. Kaimal, Shri P. Padmanabhan, Shri K. S. Narayanan, Shri Gurcharan Singh, Shri P. B. Vijay and Shri G. R. Viswanathan, Engineers of the Central Designs Organisation.

I shall be glad to receive any suggestion for improvement so that these can be incorporated in the next edition.

NEW DELHI,
Dated 1st July, 1975.

T. S. VEDAGIRI
Chief Engineer (Designs).

CONTENTS

		Page
TER 1	PLANNING OF MULTISTOREYED OFFICE BUILDINGS	1
1·1	Necessity of multistoreyed office buildings	1
1·2	Economical type of structure	1
1·3	General arrangement of Govt. Offices	1
1·4	Requirements to be considered in the planning of Govt. Office buildings	2
1·5	Norms for Govt. office buildings	4
1·6	Types of R.C.C. structures and their suitability	6
1·7	Water supply and sanitary installations	6
1·8	Electric supply	7
1·9	Telephone and Post-Office	7
1·10	Air-conditioning	
APTER 2	DESIGN LOADINGS	16
2·1	General	16
2·2	Dead loads	16
2·3	Live loads	16
2·4	Horizontal loads on parapet & balustrades	17
2·5	Impact & vibrations	17
2·6	Wind loads	18
2·7	Seismic loads	20
2·8	Special loads	
IAPTER 3	PRELIMINARY DESIGN	40
3·1	Introduction	40
3·2	Slabs	41
3·3	Beams	42
3·4	Flat slab construction	43
3·5	Columns	43
3.6	Shear walls	43
3·7	Foundation	46
3·8	Methods of preliminary design for horizontal loads	
HAPTER 4	METHODS OF ANALYSIS OF FRAMED STRUCTURES FOR VERTICAL LOADS	54
4·0	Notation	54
4·1	General	55
4·2	Methods of analysis	55
4·3	Stiffness	56
4·4	Distribution factor	56
4·5	Carry-over factor	56
4·6	Fixed-end moments	56
4·7	Sign convention	59
4.8	Hardy cross Method of moment distribution	59
4·9	Two-cycle method	67
4·10	Dr. Gaspar Kani's Method of iteration	
4·11	Takabeya's method	
4·12	Cloucek's method of distribution of deformation	
4·13	Relative advantages of different methods	

PAGE

CHAPTPR 5 METHODS OF ANALYSIS OF FRAMED STRUCTURES FOR HORIZONTAL LOADS

5·0 Notation . . . . . .
5·1 Methods of analysis . . . . . .
5·2 Distribution of horizontal force among various frames . . . . . .
5·3 Cantilever method . . . . . .
5·4 Portal method . . . . . .
5·5 Bowman's method . . . . . .
5·6 Factor method . . . . . .
5.7 Lightfoot's method . . . . . .
5·8 Kani's method . . . . . .
5·9 Khan's Method for frames with shearwalls . . . . . .
5·10 Muto's Method for frames with shearwalls . . . . . .

PAGE

105
107
107
107
109
110
113
117
127
134
144

CHAPTER 1

PLANNING OF MULTISTOREYED OFFICE BUILDINGS

1.1 Necessity of *multistoreyed* office buildings

1.1.1 With large expansion in Government Departments and rapid industrialisation and urbanisation pressure on land in metropolitan cities has increased considerably. The high cost of land and non-availability of sufficient land in the heart of the cities have necessitated vertical expansion rather than horizontal. Multistoreyed structures of six to eight storeys are common now-a-days and the present trend is towards having taller and taller buildings. Not only there is saving in land but there is also considerable reduction in the length of essential services such as water mains, sewar lines, electric supply lines, street lighting, telephone lines, etc. Besides, there is economy in cost of roof insulation. However, multistoreyed office structures bring in certain problems such as greater risk from fire, intensification of traffic and hence car parking problems, earthquakes and wind pressure problems, problems of vertical transport and extra cost of pumps for water supply etc.

1.2 Economical Type of Structure

Buildings upto 3 storeys are economical, if constructed with load bearing brick walls. Buildings, from 3 to 5 storeys are generally of partially framed design. For buildings of 6 storevs and above, framed structures are more economical than load bearing walls. For tall buildings say ten storeys and above it may be necessary to provide shear walls to resist horizontal load due to the wind or earthquake.

1.3 General Arrangement of Govt. Offices

1.3.1 For Government buildings, the arrangement usually adopted is one with a well defined corridor with rooms flanking on either side of it, but this arrangement may not necessarily be the most economical. This arrangement imposes certain restrictions on the choice of the frame pattern. The essential features of this arrangement are the additional loads due to corridor partitions and greater number of doors. These factors add to the cost of the building. The load of partitions can be considerably reduced by having light partitions of prefabricated materials like hard board, asbestos, foam concrete etc. However, this involves extra expense.

1.4 Requirements to be considered in the Planning of Govt. Office Buildings

1.4.1. The planning of a multistoreyed structure starts with a preliminary survey of the site, its location, shape, accessability from existing roads, situation of water supply sewerage and electric mains, levels of ground nature of soil etc. It may be necessary to carry out a soil survey to ascertain its bearings capacity and what type of foundation such as isolated footings, raft, piles will be suitable.

1.4.2. The various requirements to be considered at the initial stage of planning of multistoreyed office buildings are as follows :—

(1) Users' functional requirements.

(2) Space requirements for various categories of staff.

(3) Requirements of services.

(4) Miscellaneous requirements.

1.4.3 FUNCTINOAL REQUIREMENTS OF THE USERS

The functions of various Government departments vary and as such it is very essential to assess the users' exact requirements at the beginning of the planning itselt to ensure efficient utilisation of built up space. If the exact needs of the client are not taken into account, it may lead to infructuous labour and revision of plans subsequently causing delay and unnecessary expense.

1.4.4 REQUIREMENTS FOR VARIOUS CATEGORIES OF STAFF

Government of India, Ministry of Works, Housing & Urban Development have fixed the scales of accommodation for various categories of officers of the Central Government (Annexure 1.1)

The latest scale of office accommodation admissible to various categories of officers in the Gr ment of India is given below :—

(i) officers drawing Rs 1300/- or more	23·0 Sq. m. (260 sq. ft.)
(ii) Gazetted Officers (excluding Superintendents/Section Officers)	14.5 sq. m. (160 sq. ft.)
(iii) Technical staff such as Draughtsmen, Tracers & Estimators	5·5 sq.m. (60 sq. ft.)
(iv) Ministerial staff (Section Officers, Superintendents, Head Clerks, Clerks and Daftries etc.)	3.5 Sq. m. (40 sq. ft.)

In addition 10% of the accommodation allowed for ministerial staff is admissible for records.

1.4.5 General & Special Requirements

The general and special requirements to be considered and provided for in addition to the required for offices proper along with the approximate space required for them are given in Table 1

1.4.6 Minor Requirements

Minor requirements to be considered and provided are :—

(1) Niches for water coolers.

(2) Spaces for keeping brushes, and brooms etc.

(3) Chute for disposing of waste paper and refuse.

(4) Chute for posting the letters.

(5) Niches for electrical switch boards etc.

(6) Lightning conductors.

(7) First-aid-room.

1.4.7. Service Requirements

They are broadly divided into six main parts :—

(1) Sanitary installation.

(2) Water supply installation.

(3) Electric supply.

(4) Telephones.

(5) Air-conditioning.

(6) Fire fighting.

1.5 Norms for Govt. Office Buildings

The following norms for Government office buildings are recommended :—

Land coverage	25 to 40%
Ratio of carpet area to plinth area . . .	55 to 65%

TABLE 1·1

Particulars	Space required (as a rough guide)	Location	Remarks
(1)	(2)	(3)	(4)
(1) Record and stationery rooms.	2·5 percent of floor area for normal office buildings.	Basement (water proofed)/ Ground floor is suitable. For records occupying considerable area of the building, a separate block for the storage of records is recommended.	A higher provision may be made for offices where the expected volume of records is likely to be higher.
(2) Conference rooms . .	46·5 sq. m. (500 sft) per 9300 sq. m. (1,00,000 sft.) of carpet area.	At convenient place.	Conference rooms require accoustic treatment.
(3) Library	As per Users' requirement.	At convenient place.	..
(4) Canteen and Tiffin rooms .	..	Do.	Conditions vary from place to place, depending upon social habits and the availability of suitable catering service facilities near by. Each case should be examined on its own merits.
(5) Recreation rooms . .	As per Users' requirements.	..	..
(6) Co-operative Stores . .	As per Users' requirement.	At convenient place.	..
(7) Garages and cycle-stands .	As required.	Ground floor.	Spacing of Cols. and sizes of bays are planned for office accommodation on the upper floors, and by using Ground floor for parking purposes, space is wasted. Plinth area rate of framed construction of a multi-storeyed building is normally higher than normal garage block. Hence separate two storey garage blocks are recommended where sufficient land is available.
(8) Airconditioning plant room.	5·5 sq. m. for every 100 sq. m. of carpet area to be conditioned.	Ground floor/basement . .	Requirement of area for A.C. plant etc. generally depends upon the type of air conditioing adopted and also the floor area to be air-conditioned.
(9) Ducts and other requirements for air-conditioning.	As required.	..	..
(10) Lifts	..	..	..
(11) Telephone exchange (12) Post Office	0·75 percent of floor araea.	Post office may be located near the main entrances. Telephone exchange may be located centrally with respect to the rooms where the telephone instruments are provided.	..
(13) Water tanks and Pump House.	..	A portion of the actual requirement may be kept in overhead storage tanks and the balance in the underground reservoir.	Pump house may be constructed very near to the underground sump.

(1)	(2)	(3)	(4)
(14) Electrical Sub-Station	As required.	Ground floor/preferably located away from the main building a in separate building/room from the point of view of general safety.	The actual requirement should be determined in consultation with the Electrical Engineer.
(15) Quarters for caretaking staff.	Caretaker Qr. 1 Electrician's Qr. 1 Chowkidar Qrs. 3 Water fitter 1	Per 9300 Sq. m. (1 lakh of carpet area).	For every extra 9300 sq. m. (1 lakh sq. ft.) of carpet area an extra chowkidar's Qr. may be provided upto a maximum of 5 quarters.
(16) Space for receptionist and Security Staff.	As per requirement	Near the entrances in Ground Floor.	Staff engaged for security purposes have to be provided with a rest room.
(17) Banks	..	..	Requirment depends upon the population occupying the building.

Area occupied by external walls & columns	4 to 6% of plinth area.
Area occupied by internal walls and partitions	3 to 7% of plinth area.
Clear width of corridor	2.00 m (6'-6")
Area of horizontal circulation	12 to 14% of plinth area.
Entrance halls and lobbies	1.5% of plinth area.
Lifts	1 per 2320 Sq.m. (25,000 sq. ft.) to 2790 Sq. m. (30,000 sq. ft.) of plinth area.
Disposition of staircases travel distances	46 m (150 ft) (of this not more than 30.5 m.) (100') along the corridor.
Area of vertical circulation, i.e. lifts and staircases	6 to 8% of plinth area

	For male personnel	*For female personnel*
W.C.	1 for 25 persons or part thereof	1 for 15 persons or part thereof
Urinals	Nil upto 6 persons 1 for 7-20 persons 2 for 21-45 for persons 3 for 46-70 persons 4 for 71-100 persons From 101 to 200 persons add at the rate of 3 per cent. For over 200 persons add at the rate of 2.5 per cent.	
Wash basins	1 for every 25 persons or part thereof	

Area of W.C. Blocks	2 to 3% of plinth area.
Floor to floor height	3.35 m. (11'-0").
Windows (Steel windows preferable)	15 to 20% of carpet area.
Orientation	East West exposure to be avoided.

1.6 Types of R.C.C. Structure & their Suitability

1.6.1 Multi-storeyed office buildings may be constructed either of reinforced cement concrete or of steel. In case of reinforced cement concrete construction, the structures can be further divided into the following types:—

(1) Framed structures, with columns, beams and slabs.

(2) Flat slab structures with columns and slabs only. Slabs may be either flat slabs with column heads and drop panels or flat plate slabs.

(3) Combination of either (1) or (2) with shear walls.

(4) Walls and slabs only.

The choice of the type of the structure will depend on the architecture of the building. Arrangement of columns and sizes of panels play an important part in the overall economy of a fully framed structures. For Govt. office buildings, the arrangement usually adopted is one with the well defined corridor and rooms flanking on either side of it. Framed structures, with columns, beams and slabs are the most suitable for this type of arrangement. Frame having three spans with the central span approximately 10% larger than the end spans (of equal length) or frame with three equal spans are generally more economical than other arrangements. But these may not always be acceptable due to other considerations.

Some typical sections of Multistoreyed Govt. office buildings constructed by C.P.W.D. are shown in Annexure 1.A.

For proper natural ventilation and lighting, depth of a room perpendicular to the face of the building should not exceed twice the floor to floor height. As per COPP, the recommended floor to floor height is 11'-0". According to this, the maximum depth of a room should be 22'-0", and minimum longitudinal bay width consistent with utility and architecture would be 11'-0".

Some of the main advantages of flat slabs construction are given below:—

(1) It presents a flat ceiling without projecting beams.

(2) Floor to floor height can be reduced resulting in saving in building cost.

(3) It gives better diffusion of natural light in the absence of beams and girders.

(4) It is easier to construct.

(5) It requires cheaper form-work.

(6) It gives better ventilation because of the absence of pockets in ceiling.

(7) It reduces fire hazards due to absence of sharp corners.

Shear wall construction becomes necessary when the horizontal loads on the structure either due to wind or seismic forces are so large that the dimensions of the members of structure without shear walls become large and uneconomical.

Walls and slabs type of structure is suitable where the partitions are located one above the other on every floor. This type of construction is normally suitable in the case of multi-storeyed residential buildings.

1.6.2 Shear Walls

In case of tall buildings subjected to wind load (and even in medium buildings subjected to heavy seismic loads), heavy moments and shears are induced in the columns and beams of lower storeys. Although some increase in allowable stresses are permitted by the codes when wind and seismic loads are taken into account, considerable increases in sizes of structural members and the quantity of reinforcement may become necessary in many cases. Such increases in dimensions or quantity of reinforcement may be objectionable from the functional, architectural or economic points of view. In such cases, it is desireable to provide continuous diaphragm walls extending down to foundation.

In all framed buildings there are many walls like those around lavatory blocks, lift shafts, staircases etc. which can be made structural and their great stiffness against horizontal loads can be used so as to release the structural frame from the additional burden due to the horizontal loads. Not only internal walls but external facade walls whether solid with openings or grill facades formed by vertical and horizontal or inclined members can be designed to resist horizontal loads. Such walls which are made use of to resist horizontal loads are termed "shear walls". A "shear wall" can therefore, be defined as "a wall solid or perforated which resists horizontal forces in its own plane". "shear wall" may also be defined as "structural system providing stability against wind, earth tremours or blast, deriving its stiffness from inherent structural form".

The following considerations may be kept in view while planning buildings with shear walls:—

(a) As far as possible a symmetric arrangement of shear walls should be adopted to eliminate frame torsion. In case this is not possible, effect of frame torsion should be considered while designing the shear walls.

(b) Shear walls should be so located as to serve not only the structural but other functional purposes also e.g. like walls around lift pits, lavatory blocks etc.

(c) Shear walls should be made to carry as high a vertical load as possible as this will reduce the vertical tensile reinforcement.

(d) For shear walls form part of the elevation openings should be kept within limits.

1.7 Water Supply & Sanitary Installations

Water supply is required in an office building for :—

(1) Drinking purposes.

(2) W.Cs and baths.

(3) Canteens and kitchens.

(4) Fire fighting.

(5) Make up water tank of A.C. Plant.

(6) Horticultural purposes.

Provision should be made for the storage of at least one day's requirement of water. A part of this as per actual requirement, may be kept in overhead storage tanks and the balance in the underground reservoir. After ascertaining the day's requirement, the sizes and location of the underground water reservoir and overhead storage tanks should be determined. The underground reservoir may be located centrally and provided with ventilation pipes and automatic alarm arrangements. The top of the underground reservoir may be 0·30 m. (1'-0") below the ground level so that a lawn may be developed over it. Near this, a central pump house may be located which should have sufficient pumps capable of delivering the day's requirement of water supply.

Additional provision for make up water of A.C. plant should be made depending upon the type of A.C. plant to be installed for which Air-conditioning Engineers should be consulted. For fire fighting, an extra storage may be provided on the basis of local municipal regulations, or as per advice of fire fighting authorities.

Where unfiltered water supply is available, pipes may be laid for supplying unfiltered water to the lawns. For this, no storage is necessary and the supply can be taken direct from the main.

It is desirable to provide lavatory block on all floors at convenient points instead of having isolated sanitary fittings here and there. A common shaft may be provided for taking all water supply, drainage, soil and vent pipes for all the W.C. rooms situated one over the other.

1.8 Electric Supply

1.8.1 Substation & Transformer Room

(a) *Location.*—The Electric Substation shall preferably be located in a separate building, especially when there is a generating set. If this is not possible, due to site conditions, the substation may be located in the ground floor. When both of these possibilities cannot be fulfilled, the substation may be located in the first basement with suitable facilities of ramp having a mild slope for easy movement of the equipments to and from the substation. According to Rule 68(1)(a) of I.E. Rules, Substations and switch rooms shall preferably be erected above ground, but where necessarily constructed U/G, due provision for ventilation and drainage shall be made.

(b) The load and the transformer capacity depend upon the area of the building and also type of building. The loadings shall first be calculated on the following basis:—

(i) Normal lighting 21.5 KW/M² (2 KW per Sft) of plinth area.

(ii) Lighting with lifts, pumps but without central air conditioning. 32 KW/m² (3 KW per Sft).

(iii) Additional for air conditioning load. 0.096 KW/m² (0.9 KW per 100 Sft).

(iv) Additional for technical bldgs. like laboratories. 11 KW/m² (1 KW per Sft).

The transformer size can be calculated by multiplying the total connected load calculated as above by a factor 0.6. The substation area and transformer room area for different capacities are tabulated below:—

Substation with Transformer capacity of	Total Transformer Room area required	Total substation area required i/c H.T., L.T. panels Transformers but without generators
2 × 500 KVA	36·00 Sqm (390 Sft)	130·00 Sqm (1400 Sft)
3 × 500 KVA	54·00 Sqm (585 Sft)	172·00 Sqm (1850 Sft)
2 × 800 KVA	39·00 Sqm (416 Sft)	135·00 Sqm (1450 Sft)
3 × 800 KVA	58.00 Sqm (624) Sft)	181·00 Sqm (1950 Sft)
2 × 1000 KVA	39·00 Sqm (416 Sft)	149·00 Sqm (1600 Sft)
3 × 1000 KVA	58·00 Sqm (624 Sft)	197·00 Sqm (2120 Sft)

(c) Additional area that is required for generators is given below :—

25 KW 56 Sqm (600 Sft)

48 KW 56 Sqm (600 Sft)

100 KW 65 Sqm (700 Sft)

150 KW 72 Sqm (770 Sft)

The height required for the generating set room shall be a minimum of 4.57 metres (15 ft) clear from the soffit of the beam.

1.8.2 Lifts

The size, location and speed of lifts to be installed in multistoreyed office buildings have to be determined quite in advance as the provisions for the same are to be made in the structural analysis. Provision of a machine room at the top of the lift well to accommodate the machinery for the lift and the lift pit at the bottom below the lowest floor served to accommodate the buffers is to be made while designing the structure. The outline dimensions for the various parts of the lift well and machine room are given in Annexure 1.2 for various speeds of the passenger/goods lifts.

1.8.3 Pumps

The pumps provided for M.S. buildings are generally of centrifugal type, usually ranging from 5 to 10 H.P. rating. There is not any appreciable difference in the space required for the installation of pumps of these two capacities. The space required for the pump installation is given as under:—

2 pump installation . . . . . 28m²(300 Sft)
6.0 m x 4.6 m (20′ x 15′)

3 pump installation . . . . . 39m² (420 Sft)
8.5 m x 4.6m (28′ x 15′)

1.9 Telephones & Post Office

An approximate area of 0.75% of floor area for the post office and telephone exchange may be provided. Telephone exchange may be located centarally with respect to rooms where the telephone instruments are provided. Suitable ducts for telephone lines may be provided in consultation with P & T Department.

1.10 Air Conditioning

1.10.1 Systems of Refrigeration Type of Air-Conditioning

Schemes for refrigeration type of air conditioning are based on the following two systems:—

(a) Direct Expansion system.

(b) Central Chilled water system.

(a) Direct Expansion system is generally employed for air conditioning of smaller areas with just one or two weather makers located in proximity to the refrigerating plant. Reciprocating Compressors are generally employed for this type of plant. The air is conditioned by drawing it across bank of finned coils, in which refrigerant is expanded. The conditioned air is distributed to the areas through G.I. or Aluminium or occasionally masonary ducts. Where the weather maker is to serve areas in different floors, a masonary riser may be provided, preferably along the corridor walls for easy entry of supply ducts to the space above the false ceiling in the corridors, where the ducts are generally run for distribution.

Conditioned air is supplied to the rooms through grills from the ducts run in the corridors. The return air may be collected either in the space above the false ceiling or through the corridors, in which case, necessary grills should be provided above the false ceiling and on the doors respectively. The ends of the corridors are provided with Swing doors.

(b) Central Chilled Water System is generally adopted where several scattered weather makers are required. In this type of plant water is chilled centrally in the plant room and the chilled water is circulated to different weather makers by insulated chilled water pipe lines in a closed circuit. In the weather makers the chilled water is passed through finned coils dry or sprayed over which, the air is drawn and conditioned. Sometimes chilled water is sprayed in a chamber called air washer, through which the air is drawn and conditioned. The distribution of the conditioned air shall be as explained in the previous system in para (a) above. Thus the air conditioning installation consists of refrigerating machines, cooling towers, chilled water pipes, weather makers, ducts etc., as parts of the installation for producing controlled conditions of temperature and humidity inside the rooms.

1.10·2 Capacity of the Air Conditioning Plant

The capacity of the plant depends upon various factors such as orientation of the building, glass area, thickness and matreials of the walls, material used for roof exposed to sun, volume of the area, occupancy, equipment load etc. As a rough guide, the capacity of the plant can be estimated as one Ton per 18.5 Sqm. (200 Sft) of the area to be air conditioned with the ceiling height not exceeding 3 metres (10 ft.).

1.10.3 Plant Room

The plant room houses compressors, condensers, Chillers and pumps and also any air compressor for pneumatic control. The plant room shall be located in such a manner, that it is near to the weather maker (i.e. the total length of Chilled water lines is minimum) and also to the electric substation feeding power to the plant. The chilled water lines should be able to be routed with ease from the plant room to the weather maker rooms.

The plant room shall be provided of an area of 5.5 Sqm for 100 Sqm of carpet area to be conditioned. The minimum requirement of area of the smallest plant room is 10 Sqm. It shall have a minimum clear height of 3.60 Metres (12′).

The plant room shall be provided with adequate drainage and fresh water supply facilities. The inside facing of the walls of the plant room may be lined with accoustic materials wherever necessary to reduce the transmission of the air borne noise, the machinery being erected on floating foundations to prevent transmission of mechanical vibration to the building.

1.10.4 Cooling Towers

The condenser water is cooled in a cooling tower or a spray pond. There are three types of cooling towers namely; Natural draught, forced draught and Induced draught type. The forced draught and induced draught type are preferred, though more noisy, owing to higher cooling efficiency and reliability of performance apart from the smaller size of the tower. The cooling tower is generally located on the roof of the building and must be suitably located for a proper Architectural blending with the building, in addition to its functional value. The cooling ponds are sometimes used for large plants, but their application is not much preferred because (1) loss of water is more (2) their maintenance is costlier due to collection of dust in the pond and (3) the floor area occupied by a pond having depth from 1.2 to 1.7 metres is considerably greater than that required for forced draught cooling tower.

A natural draught cooling tower shall require 4.5 Sq. m. of area whereas the induced draught cooling tower shall require 1.5 Sq. m. of area per 100 Sq. m. of area to be air-conditioned. The minimum requirement of area for the smallest cooling tower is 3 Sq. m. The area requirements for cooling pond are given below:—

Conditioned Area	Cooling Pond Area
Upto 1860 Sq. m. (20000 Sq. ft.)	18 Sq. m. (per 100 Sq. m. of area to be conditioned).
1860 Sq. m. to 3720 Sq. m. (20000 to 40000 Sq. ft.) . . .	15 Sq. m. Do.
3720 Sq. m. to 5580 Sq. m. (40000 to 60000 Sq. ft.). . .	13 Sq. m. Do.
5580 Sq. m. and above (60000 Sq. ft. and above) . . .	11 Sq. m. Do.

Care should be taken that adequate space is provided around the cooling tower/pond for free flow of air and dissipation of heat.

Due to evaporation and drift, there is a certain amount of loss of water, in the cooling tower or pond. To make up for this loss, a suitable make up water tank of suitable capacity shall be provided to store the water for about 8 to 10 hours of consumption.

1.10.5 Chilled Water Pipes

Chilled water pipes are generally of G.I. for less than 10 cm dia and mild steel for higher diameter. M.S. pipes are generally of the welded type. They are generally insulated with insulation material. Normal sizes of the pipes are upto 60 cm (24"). Routing of the chilled water pipe may be underground or overhead as may be convenient and should be preplanned before start of the work to avoid future complications.

1.10.6 Weather Maker Rooms

The entire building to be conditioned is divided into a number of zones to be fed by individual weather makers. The location of W.M. rooms can be decided on the basis of a number of factors such as get up of the building, areas to be fed with the conditioned air from the Weather makers, routing of ducts, fresh air entry etc. It should be as far as possible in close proximity to the area to be conditioned.

It shall have an area of at least 4.5 Sq. m. for every 100 Sq. m. of carpet area to be conditioned. The weather maker room shall have a clear height of 3 metres.

The weather maker room shall be provided with suitable fresh water supply and drainage facilities. The weather maker shall be provided with an opening of 0.03 Sq.m. per ton for fresh air entry.

1.10.7 Ducting

The space required for the ducting depends upon factors like location of weather maker with respect to the area to be conditioned, distribution arrangement i/c collection of return air etc. There shall be about 45 Nos of duct suspenders for every 100 Sq. m. of area to be conditioned.

The duct suspenders may be of extended rod type where the buidling is to be conditioned immediately and of recessed type where provision is made for future air-conditioning.

1.10.8 Water Requirement

Water loss due to evaporation, drift etc. in case of natural cooling towers is of the order of 90 litres per hour for every 100 Sq. m. of area to be conditioned whereas corresponding figures for forced/induced draft tower & cooling pond are 65 litres and 185 litres respectively.

1.10.9 Power Requirement

Power requirement for A.C. is of the order of 8.5 K.W. per 100 Sq. m. of area to be conditioned. This may vary slightly depending upon the type of plant.

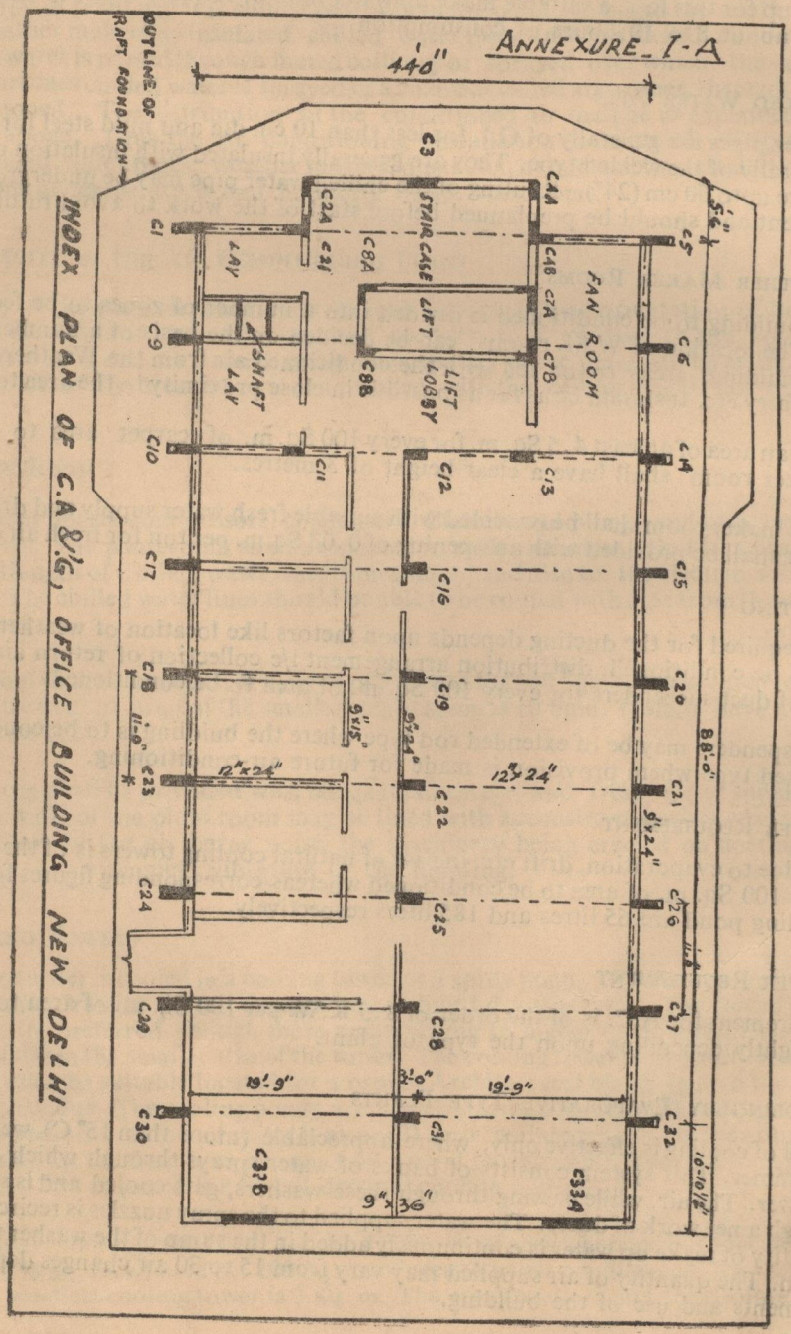
1.10.10 Cooling by Evaporative Type Plants

This method of cooling is effective only, where appreciable (more than 15° C) wet bulb depression prevails during summer. This system consists of banks of water sprays through which dry outside air is sucked by a blower. The air, while passing through these washers, gets cooled and is supplied to the required areas through a net work of ducts. The water supplied to the spray nozzles is recirculated by pumps and requisite quantity of make up water is continuously added in the sump of the washer to meet the water loss by evaporation. The quantity of air supplied may vary from 15 to 30 air changes depending upon the particular requirements and use of the building.

The evaporative plant shall be effective only in dry summer months i.e. for about 3 months from April, to June. These shall be ineffective during monsoon months, when humidity in air is quite high. The blowers can however, be used for ventilation during monsoon.

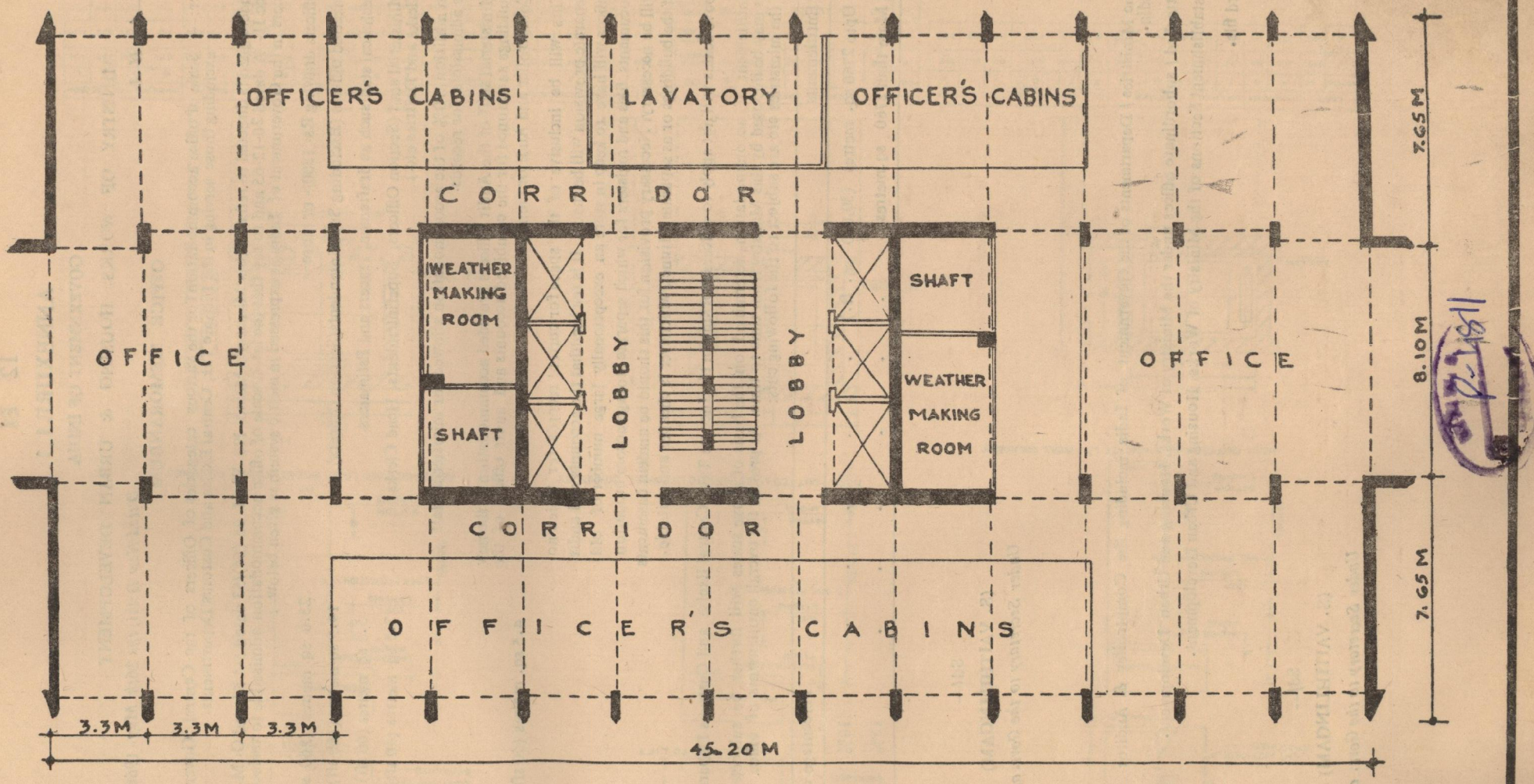
Requirements for A.C. Plants

1. Area for the plant room	20 Sq. m. + 2·7 Sq. m. per 100 Sq. m. area to be cooled.
2. Fresh air opening	1·0 Sq. m. for every 100 Sq. m. area to be cooled.
3. Water requirements	70 Litres per hour for every 100 Sq. m. area to be cooled.
4. Power Requirements	2 KW for every 100 Sq. m. area to be cooled.



ANNEXURE - I-A

20 STOREYED OFFICE BUILDING AT NIZAM PALACE - CALCUTTA



ANNEXURE 1.1

GOVERNMENT OF INDIA

MINISTRY OF WORKS, HOUSING & URBAN DEVELOPMENT

No. 26/19/65—*Acc. II-A* *Dated New Delhi, the 26th May.* 1966.

OFFICE MEMORANDUM

SUBJECT:— Scale of office accommodation for the various categories of Officers of the Central Government excluding those serving in the Income Tax, Central Excise and Customs Departments.

The undersigned is directed to refer to this Ministry's Memo. No. W. II. 95 (64)/53 dated 5-4-54 and O. M. No. 26/19/65—Acc.II- A dated 20-12-65 and to say that the latest scale of office accommodation admissible to various categories of officers in the Government of India as expressed in metric system is given below :—

(i)	Officers drawing Rs. 1300/- or more	23.0 sq. metres (260 sft)
(ii)	Gazetted officers (Excluding Superintendents/Section Officers)	14.5 sq.metres (160 sft)
(iii)	Technical staff such as draftsmen, Tracers and Estimaters.	5.5 sq. metres (60 sft)
(iv)	Ministerial staff (Section Officers, Superintendents, Head Clerks, Clerks and Daftries etc.)	3.5 sq. metres (40 sft)
	In addition, 10% of the accommodation allowed for ministerial staff is admissible for records.	
(v)	Ministerial staff of the Audit Officers provided accommodation in the new buildings sanctioned for the combined accounts and audit offices of the Comptroller and Auditor General of India.	4.5 sq. metres (50 sft.)
	This will be inclusive of the space for current records for which no separate provision will be made. It is realisedthat the accounts and audit offices will have to keep in store an exceptionally large number of old documents, files and registers for which separate storage accom modation will be necessary. Necessary provision for this should be madein basements of the buildings or as separate adjunct, whichever is cheaper and suitable.	

2. The above scales will not apply to officers working in the Income Tax, Central Excise and Customs Departments.

3. The entitlement of accommodation determined on the basis of above scales will further be subject to such *ad hoc* cuts as may be imposed by this Ministry from time to time. At present the overall entitlements of office accommodation of the ministries etc are subject to the following cuts:—

Entitlements	Percentage cut
Upto 2760 sq. metres (30,000 sq. ft.)	10%
More than 2760 sq.metres.	15%

Sd/—

(S. VAITHILINGAM)

Under Secretary to the Govt. of India.

To

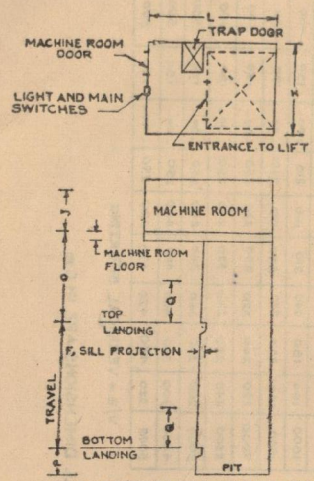
1. All the Ministries / Departments of the Government of India including the Comptroller & Auditor General of India.
2. All attached / subordinate offices under the Ministry of Works, Housing and Urban Development.
3. All establishment Sections of the Ministry of Works, Housing and Urban Development.
4. Guard file.

Sd/—

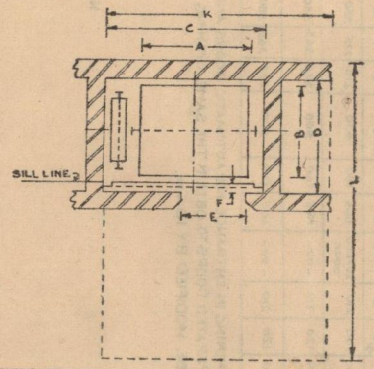
(S. VAITHLINGAM)

Under Secretary to the Govt. of India

PASSENGER LIFTS, CAR SPEEDS UPTO AND INCLUDING 1 M/S (INSTANTANEOUS SAFETY AND SPRING BUFFERS)



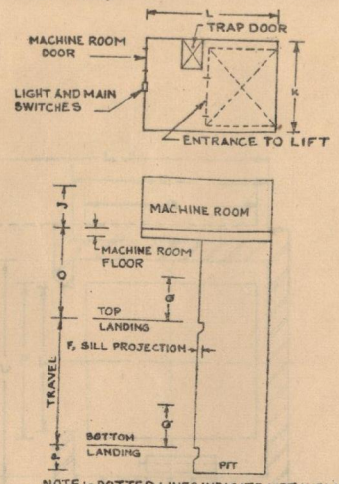
NOTE :- DOTTED LINES INDICATE LIFT WELL BOUNDARY



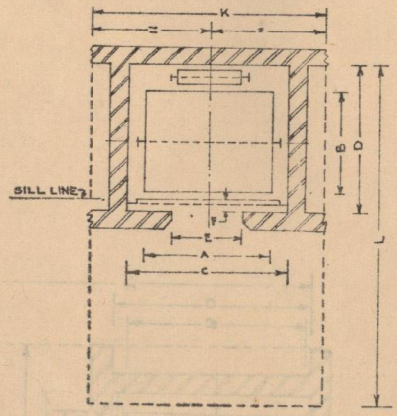
LOAD		PLATFORM		LIFT WELL		ENTRANCE			SILL	PIT	DIMENSION	MACHINE ROOM		
PERSONS	KG	A	B	C	D	✱E	G	TYPE	F	P	O	J	K	L
4	275	110	90	175	115	70	210	SINGLE SLIDE	12.5	140	450	215	245	275
6	410	135	110	195	135	80	210	SINGLE SLIDE	12.5	140	450	215	275	335

✱ IT IS RELATED TO CLEAR OPENING IN ENTRANCE FOR AUTOMATIC DOORS AND IN CASE OF MANUALLY OPERATED DOORS TO OBTAIN THE SAME OPENING, THE LIFT WELL SHOULD BE MODIFIED BY 10 CM.

PASSENGER LIFTS, CAR SPEED ABOVE 1 M/S UPTO AND INCLUDING 1.5 M/S (GRADUAL SAFETY AND SPRING BUFFERS)

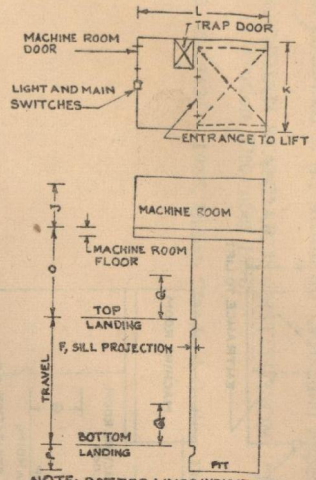


NOTE :- DOTTED LINES INDICATE LIFT WELL BOUNDARY

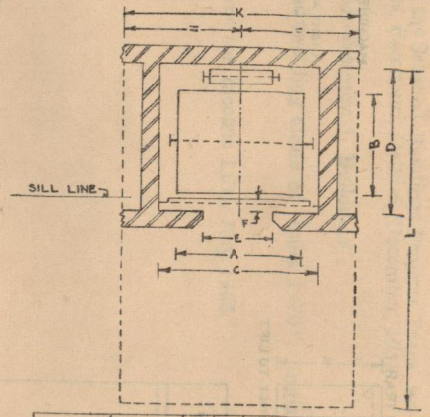


LOAD		PLATFORM		LIFT WELL		ENTRANCE			SILL	PIT	DIMENSION	MACHINE ROOM		
PERSONS	KG	A	B	C	D	✱E	G	TYPE	F	P	O	J	K	L
8	545	150	120	200	170	80	210	CENTRE OPENING	12.5	150	450	215	275	395
10	680	180	120	225	170	90	210	CENTRE OPENING	12.5	150	470	215	305	395
13	810	190	135	255	180	90	210	CENTRE OPENING	12.5	150	470	230	335	425
16	1185	215	150	255	205	105	210	CENTRE OPENING	12.5	150	480	230	335	520
20	1360	215	170	255	220	108	210	CENTRE OPENING	12.5	150	480	230	335	520

PASSENGER LIFTS, CAR SPEED ABOVE 1.5 M/S UPTO AND INCLUDING 2.5 M/S (GRADUAL SAFETY AND OIL BUFFERS)



NOTE:-DOTTED LINES INDICATE LIFT WELL BOUNDARY

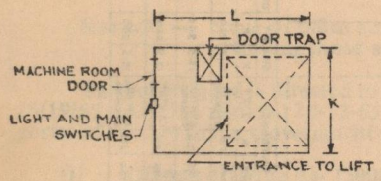


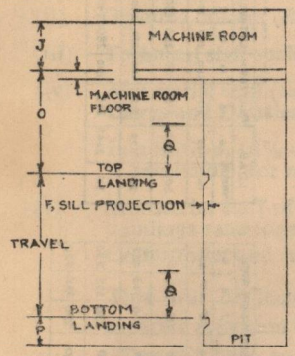
LOAD		PLAT FORM		LIFT WELL		ENTRANCE			SILL	PIT	DIMENSION	MACHINE ROOM		
PERSONS	KG	A	B	C	D	✱E	G	TYPE	F	P	O	J	K	L
16	1135	215	180	250	200	105	210	CENTRE OPENING	12.5	250	580	230	335	450
20	1360	215	170	250	220	105	210	CENTRE OPENING	12.5	260	715	230	395	490
23	1530	215	190	250	240	105	210	CENTRE OPENING	12.5	260	715	230	395	520
30	2045	245	190	280	240	120	210	CENTRE OPENING	12.5	280	730	230	425	520

NOTE :- ALL DIMENSIONS ARE IN CM

OUT LINE DIMENSIONS FOR ELECT. LIFTS AS PER I.S 3534-1966

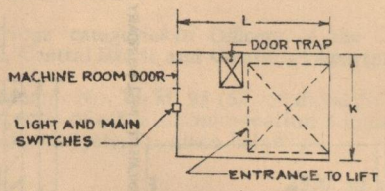
HOSPITAL BED/PASSENGER LIFTS, CAR SPEEDS UPTO AND INCLUDING 1·5 m/s (GRADUAL SAFETY AND SPRING BUFFERS)

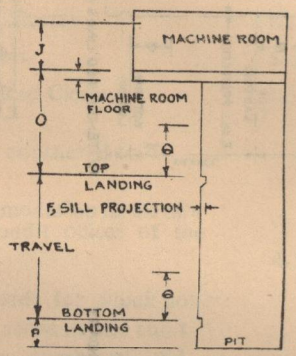




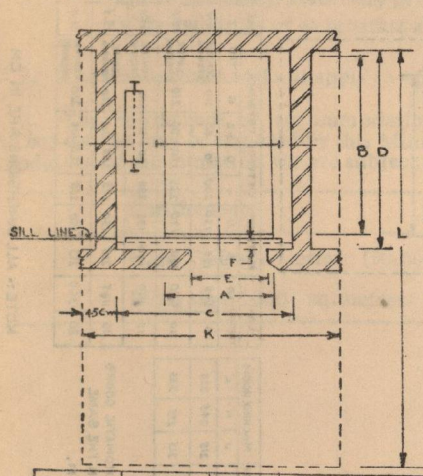
NOTE:- DOTTED LINE INDICATE LIFT WELL BOUNDARY

GOODS LIFTS, CAR SPEED UPTO AND INCLUDING 1 m/SEC. (INSTANTANEOUS SAFETY AND SPRING BUFFERS)



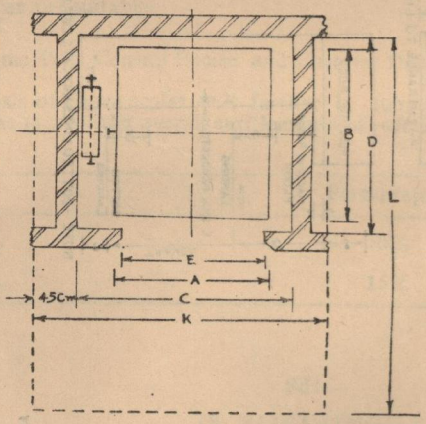


NOTE:- DOTTED LINE INDICATES LIFT WELL BOUNDARY



LOAD		PLAT FORM		LIFT WELL		ENTRANCE			SILL PIT		DIMENSION	MACHINE ROOM		
PERSON	Kg	A	B	C	D	*E	Q	TYPE	F	P	O	J	K	L
16	1135	140	235	200	260	100	210	2SPEED DOOR	16·5	150	450	230	335	550
21	1590	160	260	225	280	120	210	-DO-	16·5	160	500	230	365	580
30	2645	170	265	235	290	120	210	-DO-	16·5	160	500	230	365	580

* IT IS RELATED TO CLEAR OPENING IN ENTRANCE. FOR AUTOMATIC DOORS AND IN CASE OF MANUALLY OPERATED DOORS TO OBTAIN THE SAME OPENING THE LIFT WELL SHOULD BE MODIFIED BY 10 Cm.



LOAD Kg	PLATFORM		LIFT WELL		ENTRANCE			PIT	DIMENSION		MACHINE
	A	B	C	D	E	Q	TYPE	F	O	J	K
500	110	130	190	160	110	210	V/B DOOR	150	450	230	340
1000	160	180	240	210	160	210	-DO-	150	450	230	350
1500	190	220	270	250	190	210	-DO-	150	450	230	390
2000	220	260	320	290	220	250	-DO-	150	500	230	430
2500	240	290	340	320	240	250	-DO-	150	500	230	470
3000	260	330	360	360	260	250	-DO-	150	500	230	490
4000	300	380	400	410	300	250	-DO-	150	500	230	530
5000	320	440	420	470	320	250	-DO-	150	500	230	550

V/B = VERTICAL BIPARTING

NOTE:- ALL DIMENSIONS ARE IN Cm

REFERENCES

1. NATIONAL BUILDINGS ORGANISATION, "Symposium on Multi-storeyed Office Buildings 20th—22nd December, 1958", Published for National Buildings Organisation, 11A, Janpath, New Delhi by Indian Construction News, January, 1959.

2. COMMITTEE ON PLAN PROJECTS, "Report of the Selected Buildings Projects Team on Multi-storeyed Buildings" Printed at the Coronation Printing Works, Fatehpuri, Delhi-6, December, 1958.

3. KAIMAL S. S., Article on "Structural Aspects of Multi-storeyed Buildings".

4. INDIAN STANDARDS INSTITUTION, "National Building Code of India 1970", Indian Standards Institution, December 1970.

5. IS : 3534 — 1966, Indian Standard Outline Dimensions for Electric Lifts, July 1966.

HINE ROOM	
K	L
300	335
350	455
380	455
430	520
470	520
490	530
530	640
550	750

CHAPTER 2

DESIGN LOADINGS

2.1 General

Multistoreyed buildings are subjected to various types of loads acting on them and they can be broadly divided into three categories:—

(1) General loads such as Dead loads of the floors, columns, partition walls, foundations and Live loads.

(2) Wind and seismic loads.

(3) Special loads such as loads due to overhead water tanks, loads due to lifts, air conditioning plants etc.

The magnitude of live load on any floor will depend on the use to which the floor is subjected e.g. office, canteen, library etc. Loads coming under category (2) may vary to a great extent depending upon many factors such as location, nature of ground type of cladding etc. IS code of practice 875-1964 (revised) recommends the dead loads, live loads, wind loads, erection loads etc. and IS 1893-1970 (second revision) recommends the loads due to earthquakes.

2.2 Dead Loads

The dead load in a building comprises the weight of all walls, partitions, floors, floor finishes, roofs, false ceilings and the dead weights of all other permanent constructions in the building. The unit weight of building materials are given in Appendix 2.1(a).

2.2.1 Partition Walls

Loads due to partitions will depend on the right design and specifications and also on their positioning in accordance with the plans. The loads thus assessed shall be included in the dead load for the design of the floors and the supporting structure. However, where the actual loads of the partitions cannot be assessed before hand owing to lack of knowledge of the final positioning of the partitions, the floors and the supporting structures in such areas shall be designed to carry, in addition to other loads, a uniformly distributed load per sq. metre over the entire area of not less than 33-1/3 percent of the weight per metre run of finished partitions, subject to a minimum uniformly distributed load of 100 Kg/m² in the case of floors used for office purposes. This allowance is for light weight partitions weighing not more than 300 Kg./m.

In the case of beam and slab construction where heavier partitions like brick work are expected to be installed, it may be assumed that the heavy partitions will be placed directly over the beams and provision should be made in the design of the beams for this extra load due to heavy partitions. However the slabs need not be designed to take this extra load.

2.3 Live Loads

2.3.1 Live loads on floors shall comprise all loads other than dead loads. The minimum live loads on different floors for different uses shall be as given in Appendix 2.1. The loads mentioned in Appendix 2.1 are uniformly distributed static loads in Kg./sq.m. on the plan area and provide for normal effects of impact and acceleration, but do not take into account special loads, such as machinery, lifts etc.

2.3.2 Reduction in Floor Live Loads

The reductions in assumed total live loads on floors may be made as per values given in Appendix 2.1 in designing columns, walls, piers, their supports and foundations.

2.3.3. Live Loads on Roofs

Provision for live loads on flat roofs, sloping roofs, and curved roofs, shall be as shown in Appendix 2.1.

2.4 Horizontal Loads on Parapets & Balustrades

Parapets and balustrades should be designed for the horizontal loads acting at handrail or coping level as specified in Appendix 2.1.

2.5 Impact and Vibrations

For structures carrying live loads which induce impact or vibration, as far as possible calculations shall be made for the increase in the live load due to impact or vibration. In the absence of sufficients data for such calculations , the increase in the live load shall be as given in Appendix 2.1.

2.6 Wind Loads

The liability of a multistroreyed building to be subjected to high wind pressures depends not only upon the geographical location and proximity of other obstructions to air flow but also upon the characteristics of the structure itself.

2.6.1 Basic Wind Pressure

The basic wind pressure 'p' as defined in clause 4.2 of IS 875/1964 (revised) shall, in the absence of meteorological data, be taken as given in clause 4.2.2. of IS 875/1964 (Revised) (Appendix 2.1).

2.6.2. Wind Load on Buildings

The effect of wind shall be calculated according to the basic pressure for the entire height of the building and any projections thereof, having due regard to the level of mean retarding surface and variation in wind pressure with height. In making the calculations, due regard shall also be given to shape factors. The effect of wind on the structure as a whole is determinded by combined action of external and internal pressures acting upon it.

For structures of various plan shapes other than rectangular plan shapes, the external pressures acting on the projected area in the plane perpendicular to the wind, shall be the product of basic pressure and the shape factors given in Appendix 2.1.

The stability calculations of the building as a whole shall be made considering the combined effect of dead loads. live loads, and wind loads as well as dead loads and wind loads only.

2.6.3. Internal Air Pressures

Internal air pressure in a building depends upon the degree of permeability of the cladding to the flow of air. In practice, the four cases given in Appendix 2.1 may be considered for design purposes.

2.6.4. Wind Pressures on Walls

A wall of any building should be sufficiently strong to resist a total average pressure, outwards or inwards of :

(a) 0.5 p for buildings having a small degree of permeability.

(b) 0.7 p for buildings. of normal permeability.

(c) P for buildings. with large openings and for walls exposed to wind on both faces.

2.6.5. Permissible Increase in Allowance Bearing Pressure of Soils.

In case where the footing is eccentrically loaded and/or the member being supported transmits a moment (due to wind loads acting horizontally at floor levels) to the footing, the permissibe pressure may be exceeded at the edges of the footing by 25% when variation in the intensity of the reaction caused by the transmission of moments to the footings is taken in to account.

2.6.6 Permissible In crease in Material Stresses

Where stresses due to wind effects are combined with those due to dead, live and impact loads, the permissible stresses in concreate and steel reinforcement specified in Chapter VI may be exceeded upto a limit of 33⅓ per cent provided in no case does the direct strees in the reinforcement exceed 2600 kg/cm². Wind and seismic forces need not be considered as acting simultaneously.

2.6.7. Load Factors

Unless otherwise specified by the Engineer-in-charge or the appropriate authority, every member should be designed to carry without failure the effects of the following critical load conditions, when wind load is considered along with dead and live loads.

$$U = 1.5\ DL + 2.2\ LL + 0.5\ WL,$$

or

$$U = 1.5DL + 0.5LL + 2.2\ WL,$$

whichever gives critical conditions provided that no member shall have a capacity less than that required by the condition,

$$U = 1.5\,DL + 2.2\,LL$$

Where U= the ultimate load for which the structure or its elements should be designed according to the relevant IS Code for R.C.C. structure ;

DL = the Dead load of the structure;

LL = the live load on the structure considering its modified values as given in the relevant clauses of the standards; and

WL= the value of wind load adopted for the design.

These load factors may be reduced as per clause 4.2.2 of IS 875—1964 (Revised) (Appendix 2.1)

2.7 Seismic Loads

2.7.1 General Principles & Assumptions

Earthquake shocks cause a movement of ground on which the structure is situated. This movement causes the structure to vibrate. This vibration may be resolved in any three perpendicular directions. The predominent direction of vibration is *h orizontal*.

The response of the structure to the ground vibration is a function of the nature of foundation soil, materials of which the structure is composed, form size and rigidity ; the duration and the intensity of ground motion.

In the case of structures designed for horizontal seismic force only, it shall be considered to act in any one direction at a time so as to give the worst effect. The building should be checked as a rule for seismic forces in both the directions separately. Where both horizontal and vertical seismic forces are taken in to account, horizontal force in any one direction at a time may be considered simultaneously with the vertical force which may be taken as half of the horizontal seismic co-efficient. The vertical seismic coefficient shall be considered in the case of structures in which stability is a criterion of design or for overall stability is criterion of design or for overall stability analysis of structures.

The following assumptions shall be made in the design of structures to resist earthquake forces,

(a) Earthaquakes cause impulsive ground motion which is complex and irregular in character, changing in period and amplitude and lasting for smll duration. Therefore, resonance of the type as visualised under steady state sinusoidal excitations will not occure as it would need time to build up such amplitudes.

(b) An Earthquake is not likely to occur simultaneously with wind or floods.

(c) The structure is standing on a soil which will not considerably settle or slide appreciably due to vibration lasting for a few seconds.

2.7.2. Seismic Co-Efficients for Different Zones

Unless otherwise stated, horizontal seismic coefficient in static design in different zones shall be taken as specified in Appendix 2.2.Seismic fco-efficients or some important towns are given in Appendix 2.3.

2.7.3. Design Criteria For Building;

For building not exceeding 40 M in height, the base shear VB is given by the following formula ;

$$VD = C.\alpha_h.\beta.W.$$ $$VB = C \cdot \alpha_h \cdot B \cdot W$$

where,

C = a cofiefficient defining the flexibility of structure with the increase in number of storeys $= \frac{0.5}{1/3}$ $\frac{0.5}{T^{1/3}}$

α_h = seismic coefficient as defined earlier as the case may be.

β= a co-efficient depending upon the soil foundation system.

W= total dead load plus appropriate amount of live load as defined in Appendix 2.2 and

V= Axial force in column P-105

T = fundamental time period of the building in seconds.

NOTE 1.—The maximum value of C will be limited to 1·333 for building with load bearing walls and 1·000 for framed buildings. The minimum value of *C* will be limited to 0·33.

NOTE 2.—The fundamental time period may either be established by experimental observations on similar buildings or calculated by any rational method of analysis. In the absence of such data T may be determined as follows:

(a) For moment resisting frames without bracing or shear walls for resisting the lateral loads.

$$T = 0{\cdot}1\ n$$

where n = number of storeys includindg basement storeys.

(b) For all others

$$T = \frac{0{\cdot}09\ H}{\sqrt{D}}$$

where H = total height of the main structure of the building in metres.

D = dimension of buidling in metres in a direction parallel to the applied seismic force.

Distribution of forces along with the height of the building is given by the following formula.

$$Q_i = V_b \frac{w_i h_i^2}{\sum_{i=1}^{i=n} w_i h_i^2}$$

where Q_i = lateral forces at roof or floor i,

V_b = the base shear as worked out earlier,

W_i = the weight in floor including dead load and appropriate live load,.

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

n = number of storeys including the basement floors.

NOTE —For calculating weight at level of the roof or floor, the weight of walls and columns in any storey is assumed to be shared half and half between the roof or floor at top and the floor or ground at bottom, and all weights are assumed to be concentrated at the level of the roofs or floors.

For buildings greater than 40 m in height and up to 90m, model analysis is recommended, However the method suggested above may also be used for design of structures in this category in zones I to IV

~~2.4.~~ 2.7.4. DRIFT AND TORSION

Check for drift and torsion is desirable for all buildings, being particularly necessary in cases of buildings greater in height than 40m.

The maximum horizontal relative displacements due to earthquake forces between two successive floors shall not exceed 0.004 times the difference in levels between these floors.

Provision shall be made for the increase in shear resulting from the horizontal torsion due to an eccentricity between the centre of mass and the center of Rigidity.

The design eccentricity shall be taken as 1.5 times the computed eccentricity between the centre of mass and centre of rigidity.

NOTE 1.—Centre of mass:—the point thruagh which the resultant of the masses of a system acts. This corresponds to centre of gravity of system.

NOTE 2.—Centre of Regidity:— the point through which the resultant of the restoring forces of a system acts.

2.7.5. MISCELLANEOUS

Parapets and other vertical cantilever projections attached to buildings and projecting above the roofs shall be designed for five times the horizontal seismic co-efficient specified earlier.

All horizontal projections like cornices and balconies shall be designed to resist a vertical force equal to *five times* the vertical seismic co-efficient multiplied by the weight of the projection.

NOTE .—The increase seismic co-efficients specified above are for designing the projecting part and its connection with the main structure. For the design of the main structure such increase need not be considered.

2.7.6. PERMISSIBLE INCREASE IN ALLOWABLE BEARING PRESSURE OF SOILS

When earthquake forces are included, the permissible increase in allowable bearing pressure of soil shall be as given in Appendix 2.2, depending upon the type of foundation of the structure.

2.7.7. INCREASE IN PERMISSIBLE STRESSES

Whenever earthquake forces are considered along with other normal design forces, the permissible stresses in material in the elastic method of design may be increased by *one-third* provided that, for steels having a definite yield stress, the stress will be limited to the yield stress ; for steels without a definite yield point the stress will be limited to 80% of the ultimate strength.

2.7.8. LOAD FACTORS (FOR ULTIMATE LOAD METHOD OF DESIGN)

(a) For steel and reinforced concrete structures :—

$U = 1.4(D.L. + L.L. + E.L.)$

(b) for prestressed concrete structures :

$U = 1.5(D.L. + L.L. + E.L.)$

where

U = the ulimate load for which the structure or its elements should be designed according to the relevant IS Code for R.C.C. steel and prestressed concrete structures.

DL = the Dead Load of the structure

EL = the value of earthquake load adopted for the design.

LL = the live load on the structure considering its modified values as given in the relevant clauses of the standards.

The members of reinforced or prestressed concrete shall be under reinforced so as to avoid sudden failure due to crushing of concrete. Further it should be suitably designed so that premature failure due to shear or bond may not occur.

2.8 Special Loads

2.8.1. WATER TANK LOADS

Water requirement of a multistoreyed office building is related to the number and type of water and sanitary fittings which, in turn , are governed by number of users who will likely be occupying the building. The number of users is roughly proportional to the plinth area of the building. Assuming an average of 10 sq. metre of plinth area per person and 50 litres per capita per day, the total requirement of water can be calculated. It is the general practice to provide a portion of the total requirement for the overhead storage tanks and the balance for the underground storage tanks. Roughly half of the daily drinking and sanitary requirements of water are stored on the roof and the other half in underground storage tanks. In addition to the above, provision has also to be made for storing water on the roof for fire fighting and air-conditioning. This will have to be worked out in consultation with fire-fighting and air-conditioning departments.

The load due to the water stored in the overhead storage tanks along with the self weight of the structure holding the water are reqiured to be considered for structural design purposes. The general practice adopted is to provide overhead storage tanks just above the toilet room blocks shown in the plans. The capacity of such overhead storage tanks depends upon the number of sanitary installations available in that particular block and average water consumed by the installations. Alternatively an overhead storage tank can be located at a convenient place at the roof level with lines emerging from this tank to various blocks accommodating toilet rooms.

For design purposes, as per IS 875-1957 the static water load and its container weight shall be treated as dead loads.

It is desirable to provide R.C.C. storage tanks over the roofs of buildings in preference to steel tanks which get corroded very rapidly and give rise to problems in maintenance.

2.8.2 LIFT MACHINE ROOM

The size, location and speed of lifts to be installed in multistoreyed office buildings have to be determined/quite in advance as the provision for the same are to be made in the structural analysis,. Provision of a machine room at the top of the lift well to accommodate the machinery for the lift and a lift pit at the bottom below the lowest floor served to accommodate the buffers is to be made.

APPENDIX 2·1(a)

Unit & Weight of Building Materials

(Refer Clause—2·1 Table I of IS—1911—1967)

Sl. No.	Material	Nominal size or thickness mm.	Weight Kg.	per
1	Accoustic Materials			
	Glass Fibre	10	0.388	m^3
	Cork	..	240	m^3
2	Aggregate, Coarse Broken Stone Ballast			
	Dry, Well shaken	..	1600 to 1870	,,
	Perfectly wet	..	1920 ro 2240	,,
	Shingles, 3 mm to 38 mm	..	1460	,,
	Broken Bricks			
	Fine	..	1450	,,
	Coarse	..	1010	,,
	Foamlag, Foundry Pumice	..	700	,,
3	Aggregate, fine Sand			
	Dry, Clean	..	1540 to 1600	,,
	River	..	1840 to	,,
	Wet	..	1760 ~~to~~ 2000	,,
	Brickdust (Surkhi)	..	1010	,,
4	Asbestos Cement Sheeting			
	Corrugated (146 mm)	6	12 to 13.3	m^3
		7	14.1 to 15.6	,,
	Semi-corrugated (340 mm)	6	12 to 13.3	,,
		7	14.1 to 15.6	,,
	Plain	5	9.16	,,
5	Bitumen	..	1040	m^3
6	Blocks			
	Lime-cement cinder solid blocks	..	880 to 1280	,,
7	Bricks			
	Common burnt clay bricks	..	1600 to 1920	,,
	Engineering bricks	..	2160	,,
	Pressed bricks	..	1760 to 1840	,,
	Refreactory bricks	..	1760 to 2000	,,
	Sand cement bricks	..	1840	,,
	Sand Lime bricks	..	2080	,,
8	Cement			
	Ordinary & Aluminous	..	1440	,,
	Rapid-hardening	..	1280	,,
9	Cement Concrete, Plain			
	Aerated	..	760	m^3
	With sand & gravel or crushed natural stone aggregate	..	2240 to 2400	,,
10	Cement Concrete, Reinforce with sand & gravel or crushed natural stone aggregate			
	With 1 percent steel	..	2310 to 2470	m_3
	With 2 percent steel	..	2370 to25 30	,,
	With 5 percent steel	..	2560 to 2720	,,

Sl. No.	Material	Nominal size or thickness mm.	Weight kg.	per
11	Cement concrete, prestressed	..	2400	m^3
12	Cement mortar	..	2080	,,
13	Cement plaster	..	2080	,,
14	Felt, bituminous for water proofing & damp proofing Fibre base			
	Type 1 (underlay)	..	0·68	m^2
	Type 2 (Self-finished felt)			
	Grade 1	..	1·72	,,
	Grade 2	..	2·19	,,
	Grade 3	..	3·08	,,
	Hessian base type 3. (self finished felt)			
	Grade 1	..	2·23	,,
	Grade 2	..	3.64	,,
15	Glass			
	Glass bricks, window glass & looking glass	..	2480 to 2720	m^3
	Sheet	2·0	5·0	m^2
		2·5	6.3	
		3·0	7.5	,,
		4·0	10·0	,,
		5·0	12·5	,,
		5·5	13·7	,,
		6·5	17·0	,,
16	Iron			
	Pig	..	7200	m^3
	Grey, cast	..	7030 to 7130	,,
	White, cast	..	7580 to 7720	,,
	Wrought	..	7700	,,
17	Lime			
	Lime concerete with burnt clay aggregate	..	1920	,,
	Lime mortar	..	1600 to 1840	,,
	Lime plaster	..	1760	,,
18	Soils & Gravels			
	Alluvial ground, undisturbed	..	1600	,,
	Clay fills :			
	Dry, lumps	..	1040	m^3
	Dry, compact	..	1440	,,
	Damp, compact	..	1760	,,
	Wet, compact	..	2080	,,
	Undisturbed	..	1920	,,
	Undisturbed, gravelly	..	2060	,,
	Earth	..	1410 to 1840	,,
	Dry moist	..	1600 to 2000	,,
	Fine sand :			
	Dry	..	1600	,,
	Saturated	..	2080	,,
	Silt, Wet	..	1760 to 1920	,,

Sl. No.	Material	Nominal size or thickness mm.	Weight kg.	per
19	Stones			
	Basalt	..	2850 to 2960	m^3
	Chalk	..	2190	,,
	Dolomite	..	2880	,,
	Gneiss	..	2400 to 2690	,,
	Granite	..	2640 to 2800	,,
	Laterite	..	2080 to 2400	,,
	Limestone	..	2400 to 2640	,,
	Marble	..	2720	,,
	Quarts rock	..	2640	,,
	Sand stone	..	2240 to 2400	,,
	Slate	..	2800	,,
20	Terra cotta	..	1870 to 2370	,,
21	Terrazzo			
	Paving	10	24	m^2
22	Timber			
	Deodar	..	545	
	Kail	..	515	,,
	Kusum	..	1150	,,
	Mahogany	..	675	,,
	Oak	..	865	,,
	Sal	..	865	,,
	Sisso	..	785	,,
	Teak	..	640	,,
23	Water			
	Fresh	..	1000	,,
	Salt	..	1025	,,
24	Wood-Wool building slabs	10	6	m^2

Machine room floors shall be designed to carry a load of not less than 500/kg/m^2 over the whole area and also any load which may be imposed thereon by the equipment used in the machine room or by any reaction from such equipment both during periods of normal operation and repair in consultation with S. W. (Elect.).

The beams supporting the lift machine room will ultimately carry the entire weight including impact which varies with the size and speed of the lift. In addition, the beams at all other floors around the lift well should be made stronger to take the reactions from the guides when the lift is stopped suddenly by the utomatic safety device, on the breaking of the wire rope. These reaction from the guides may be obtained from the lift firms at the time of structural designs.

Suitable lifting beams immediately below the machine room ceiling may be provided for carrying tackle to facilitate lifting of any heavy part of a heavy lift (say about 1,500 kg. capacity); and for capacities below that, suitable suspension hooks may be provided.

In the case of large lift installations, the roof of the machine room also should be designed to take up the pulley which could be used for lifting up parts of the lift machinery for inspection and repair.

2.8.3. Weather Maker Rooms etc.

The structural memebers should be designed for load due to ducts, false ceiling, weather maker rooms, cooling towers and make up water tanks for the airconditioning in consultation with S. W. (air-conditioning). The following loadings may be adopted as a general guide :—

1. *False ceiling* : A uniformly distributed dead load of 40 Kg/m^2 (8 lbs/Sft) may be assumed over the floor area for the provision of false ceiling.

2. *Weather maker Rooms.*—Weather maker room floors may be designed for a uniformly distributed live load of 1200 kg/m^2.

APPENDIX 2·1

Live Loads on Floors

Clause 3·1·1· of IS 875-1964 (Revised Para 2.3)

Loading Class No.	Types of ffoors	Minimum live loads in Kg/m² of floor area	Alternative minimum live load
1	2	3	4
200 . .	Floors in dwelling houses, tenements, hospital wards, bed rooms and private sitting rooms in hostels, and dormitories.	200	
250 . .	Office floors other than entrance halls, ffoors of light workrooms.	*250—400	Subject to a minimum total load of 2·5 times the values in column 3 for any given slab panel and 6 times the values in column 3 for any given beam.
300 . .	Floors of banking halls, office entrance halls and reading rooms.	300	
400 . .	Shop ffoors used for the display and sale of merchandise; floors of workrooms generally; floors of class-rooms, in schools floorso places of assembly with fixed seating, restaurants; circulation space in machinery halls, power stations, etc. where not occupied by plant or equipment.	400	This total load shall be assumed uniformly distributed on the entire area of the slab panel or the entire length of the beam
500 . .	Floors of warehouses, workshops, factories and other buil dings or parts of buildings of simiiar category for light weight loads; office floors for storage and filing purposes; floors of places of assembly without fixed seating, public rooms in hotels, dance halls, waiting halls, etc.	500	
750 . .	Floors of warehouses, workshops, factores and other buildings or part of buildings of similar category for medium weight loads.	750	..
1000 . .	Floors of warehouses, workshops, factories and other building or parts of buildings of similar category for heavy weight loads, floors of book stores and libraries, roofs and payement lights over basements projecting under the public footpath.	1000	..
Garage, Light.	Floors used for garages for vehicles not exceedng 2·5 tonnes gross, weight:		
	Slabs	400	or the worst combination of actual wheel loads, whichever is greater
	Beams	250	or the worst combination of actul wheel loads, whicever is greate
Garage, Heavy	Floors used for garages for vehicles not exceeding 4 tonnes gross weight.	750	Subject to a minimum of one-and-a-half times maximum wheel load but not less than 900 kg considered to be distributed over 75 cm square.
Stairs . .	Stairs, landings and corridors for class 200 loading but not liable to overcrowding.	300	Subject to a minimum of 130 Kg concentrated load at the unsupported end of each step for stair constructed out of structurally independent cantiliver steps.
	Stairs, landings and corridors for class 200 loading but liable to overcrowding, and for all other classes.	500	
Balcony . .	Balconies not liable to overcrowding:—		
	For class 200 loading	300	..
	For all other classes	500	
	Balconies liable to overcrowding . . .	~~530~~ 500	..

*The lower value of 250 Kg/m² should be taken where separate storage facilites are provided and the higher value of 400 Kg/m² should be taken where such provisions are lacking.

NOTE 1.—In the above Table a reference to a 'floor' includes a reference to any part of that floor, and a reference to slabs includes boarding and beams or ribs spaced not further apart than one meter between centres, and a reference to beams means all other beams and ribs.

NOTE 2.—Under loading class No. 250, the reference to 'light work rooms' envisages rooms in which some light machines (for example, sewing machines used by milliners or tailors) are operated without a central, power-driven unit, that is the machines are independently operated, either by hand or by small motors. Under loading class No. 400, the reference to 'workrooms' generally envisages the installation of machines operated with a central power-driven unit with the individual machines being belt driven.

NOTE 3.—'Fixed seating' implies that the removal of the seating and the use of the space for other purposes is improbable. The maximum likely load in this case, is therefore, closely controlled.

NOTE 4.—The loading in workshops, ware houses and factories varies considerabl and so three loadings under the terms 'light', 'medium' and 'heavy' are introduced in order to allow for more economical designs but the terms have no special meaning in themselves other than the live load for which the relevant floor is designed. It is, however, important particularly in the case of heavy weight loads, to assess the actual loads to ensure that they are not in excess of 1000 kg/m²; in case where they are in excess, the design shall be based on the actual loading.

NOTE 5.—The load classification for stairs, corridors, balconies and landings provide or the fact that these often serve several occupancies and are used for transporting the furniture and goods.

Reduction in Floor Live loads

(Clause 3·1·2·1. of IS 875-1964 (Revised) Para 2·3·1)

Number of floors carried by member under consideration	Percent reduction of total live load on all floors above the member under consideration
1	0
2	10
3	20
4	30
5 or more . . .	40

Where a single span of a beam or girder supports not less than 50 m² of floor at one general level, the live load may be reduced in the design of the beam or girder by 5% for each 50 m² supported, subject to a maximum reduction of 25%. This reduction or that given above, whichever is greater, may be taken into account in the design of columns supporting such a beam but such reduction shall not be made where the floors are used for storage purposes not in the weight of any plant or machinery which is specially allowed for.

No reduction shall be made in the case of ware-houses, garages and other buildings used for storage purposes and for factories and workshops designed for 500 Kg/m². However, for buildings, such as factories and workshops, designed for a live load of more than 500 kg/m², the reductions given above may be made provided that the loading assumed for any column etc. is not less than it would have been if all the floors had been designed for 500 Kg/m² with no reduction.

Live Loads on Roofs

(Clause 3·2·1· of IS 875-1964 (Revised) Para 2·3·3)

Sl. No.	Type of Roof	Live Load measured on Pln.	Minimum LL measured on plan
(1)	(2)	(3)	(4)
(1)	(i) Flat, sloping or curved roof with slopes upto and including 10 degrees		
	(a) Access provided . . .	150 Kg/m²	375 Kg. uniformly distributed over any span of one metre width of the roof slab and 900 Kg uniformly distributed over the span in the case of all beams.
	(b) Access not provided except for maintenance.	75 Kg/m²	190 Kg. uniformly distributed over any span of one metre width of the roof slab and 450 Kg uniformly distributed over the span in the case of all beams.

(1)	(2)	(3)	(4)
	(ii) Sloping roof with slope greater than 10 degrees.	(a) For roof membrane, sheets or purlings 75 kg/m² less 2 kg/m² for every deg. increase in slope over 10 degrees.	Subject to a minimum of 40 Kg/m².
		(b) For members supporting the roof membrane and roof purlins, such as trusses, beams, girders, etc.—2/3 of load in (a)	
		(c) Loads in (a) and (b) do not include loads due to snow, rain dust collection, etc. and he effects of such loads shall be appropriately considered.	
	(iii) Curved roofs with slope at springing greater than 10°.	$(75-345\gamma^2)$ Kg/m² where $\gamma = \frac{h}{l}$ h = the height of the highest point of the structure measured from its springing. l = chord width of the roof if singly curved and shorter of the two sides, if doubly curved.	Subject to a minimum of 40 Kg/m²

NOTE.—For special type of roofs with highly permeable and absorbant material, the contingency of roof material increasing in weight due to absorption of moisture shall be provided for.

Roofs of buildings used for promenade or incidental assembly purposes shall be designed for a minimum load of 400 Kg/m² or heavier if required.

Roofs subjected to snow loads should be designed for the actual load due to snow or for the live loads specified above whichever is more severe. Actual load due to snow will depend upon the shape of the roof and its capacity to retain the snow and each case shall be treated on its own merits. In the absence of any specific information, the loading due to the collection of snow may be assumed to be 2·5 Kg/m² per cm depth of snow. In the case of roofs with slopes greater than 50° snow load may be disregarded; where, however, there are possibilities of formation of snow pockects, these should be taken into account.

Horizontal Loads on Parapets/Balustrades

(Clause 3.3 of IS 875/1964 (Revised) Para 2·4

Description	Horizontal load Kg./metre run.
(a) Stairways, landings and balconies (private and domestic) :	
Internal	35
External	75
(b) All other stairways, parapets and handrails to roofs.	75
(c) Parapets and balustrades in places of assembly likely to be overcrowded . .	225

The values given are for guidance only and where values for actual loadings are available, they shall be used instead.

Impact and Vibration

(Clause 3·4·1· of IS 875/1964 (Revised) Para 2·5)

Structure	Impact factor
(a) For frames supporting lifts and hoists	100%
(b) For foundations, footings and piers supporting lifts and hoisting apparatus .	40%
(c) For light machinery, shaft or motor units	20% minimum
(d) For reciprocating machinery or power units	50% minimum

Concentrated Live loads with impact and vibrations which may be due to installed machinery shall be considered and provided for in the design. The impact factor shall not be less than 20% which is the amount allowance of light machinery.

Basic Wind Pressure*

(including winds of short duration) Fig 2. 1A

Clause 4·2·2 of IS 875/1964 (revised)

Para 2·6·1 & 2·6·7

Zone	Pressures in Kg/m² upto a ht. of 30 m above the mean retarding surface	Pressures in Kg/m² at a height (metres)									
		35	40	45	50	60	70	80	100	120	150
I . . .	100	104	105	108	111	115	118	122	127	132	138
II . . .	150	156	158	163	167	172	177	183	191	198	207
III . . .	200	208	210	217	222	230	236	244	254	264	276

(For intermediate heights interpolated values may be adopted).

NOTE 1.—The wind pressures given in Fig 2. 1A are the maximum ever likely to occur in any locality including conditions of winds of short duration, meaning thereby winds in which the maximum speed is attained suddenly and lasts for a few minutes only, such as in squalls. The response of a structure to wind pressure depends not only on its own characteristics and those of the wind flow but also on duration of the wind pressures. Certain structures may not respond to short lived phenomena of wind, such as squalls and in such cases winds lasting over a long period as given in Fig 2. 1B need only be taken into consideration. At present, the Committee has no data to make a definite recommendation in regard to the situations in which Fig 2. 1B could be used in place of Fig 2. 1A. It is, therefore, recommended that until the required data becomes available, Fig. 2.1A should alone be used as the basis, and information given in Fig 2. 1B may be taken to assist in the design in a manner appropriate to the given problem, but in general as described in Note 2 below :—

NOTE 2.—In view of the wind phenomena and the nature of wind and the relatively short period over which it acts, it may not be necessary to treat it in the same manner as live loads. In the case of working stress designs higher stresses than those normally permissible in the case of live loads & in the case of ultimate load designs lower load factors than those normally adopted for live loads, could be permitted when wind loads are taken into consideration. In view of the large number of factors which contribute to a final decision on the exact value of the increase in permissible stresses or decrease in load factor, in any given case of structural design these values should be determined by the designer on the basis of the provisions given in the relevant Indian Standard dealing with the design aspects. In the absence of such recommendations and unless otherwise specified by the appropriate authority the permissible stresses and load factors shall be as given below for different values of "K".

K	Permissible stress not more than		Load factor not less than	
1·0	1·0	Times the increased† stress, for wind load in combination with other loads, permissible under the relevant Indian Standard design codes subject to the value being not more than the yield stress or 0·2 percent proof stress of the material.	1·0	Times the load factor for wind load in combination with other loads as per relevant Indian Standard design codes subject to a minimum value of one.
1·5	1·04		0·96	
1·67	1·06		0·95	
2·0	1·08		0·91	
2·5	1·13		0·88	

where K is the ratio = $\frac{\text{Wind pressure from Fig 2·1A for a given location}}{\text{Wind pressure from Fig 2.1B for the same location}}$

NOTE 3.—Reduction factor for design wind pressures for building up to 30 m height shall be as follows :

(a) For buildings up to 10 m height, the intensity of wind pressure specified in Figs. 2. 1A & 2. 1B may be reduced by 25 percent for stability calculations and for the design of framework as well as cladding.

(b) For buildings over 10 m and up to 30 m height, the intensity of wind pressure specified in Figs. 2. 1A & 2. 1B may be reduced by 25 percent for stability calculations and for design of columns only.

NOTE 4.—The permissible soil pressure when wind load is considered shall be taken as the normal allowable bearing pressure on the soil multiplied by the same factors as those for permissible stresses given in the above table.

*Basic wind pressure 'p' is an equivalent static pressure in the direction of flow of wind.

†Where stresses due to wind, temperature and shrinkage of facts combined with those due to dead, live and impact load the permissible stresses in concrete and steel reinforcement may be exceeded upto a limit of 33 1/3 per cent, provided in no case does the direct stress in the reinforcement exceed 2600 kg./cm².

Shape (in plan) factor

Refers Clause 4·3·2 IS 875/1964 (revised)

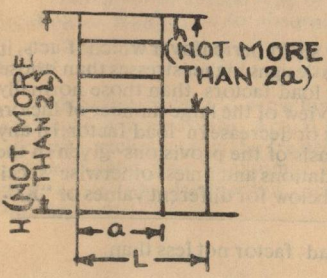
Para 2·6·2

Plan shape of the structure	Ratio of Height to base width 0—4	Ratio of Height to base width 4—8	Ratio of Height to base width 8 or over
Circular	0·7	0·7	0·7
Octagonal	0·8	0·9	1·0
Square (wind perpendicular to diagonal)	0·8	0·9	1·0
Square (wind perpendicular to face)	1·0	1·15	1·30

NOTE 1.—In the case of projections above the general roof level the base width shall be taken as the width of the construction where it rises from the surface of the roof; and the height governing the ratio of height to base width shall be that from the roof surface to the top of construction.

NOTE 2.—The division of wind load into that acting on windward and leeward sides shall be the same as that for rectangular buildings as affected by the openings.

Calculations shall be made for the effect of wind on the design of individual components of the buildings, where, however, adequate stiffening is provided by walls, or floors and walls, calculations for the effects of wind, except in regard to wall panels, roofs and foundations, need not be made on:

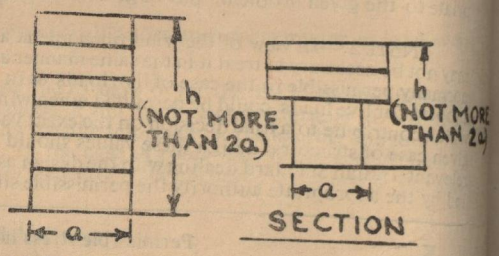


SECTION

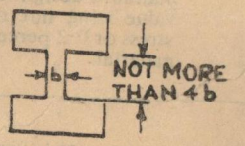
(a) a building or a part of a building, of which the height 'h' does not exceed twice the eff. widths.

(b) A Section adjoining two parts of an adequately stiffened building, if height of the Section exceeds twice its width but the length of the section does not exceed four times its width.

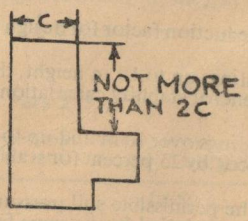
(c) a wing of such a building if it does not project more than twice its width.



SECTION SECTION



PLAN



PLAN

Internal Air Pressures

(Refers Clauses 4·3·5·1 to 4·3·5·3 of 875/1964 (revised) Para 2·6·3)

1	Buildings having a small degree of permeability.	The flow of air through the cladding is practically negligible (e.g. multistoreyed building with panelled walls and no openings.)	Internal air pressure may be neglected.
2	Buildings of normal permeability . .	Buildings where the cladding permits the flow of air but where there are not large openings.	Internal pressure (positive or suction) of 0·2p acting normal to the wall and roof surfaces in addition to the external wind pressure.
3	Buildings with large openings . .	Opening larger than ~~30%~~ 20% of the wall area.	Internal pressures positive or negative of 0.5p in addition to external wind pressures.
4	Buildings of open type . . .	Buildings with roofs but no walls, the roofs will be subject to pressures from both inside and outside.	. .



The territorial waters of India extend into the sea to a distance of twelve nautical miles

emergency relief stores etc. shall be designed for one and a half times the seismic coefficients given above

(efer IS 1498—1959

Refer IS 2131— measured from the appropriate base line.

I

FIG. 2·1A BASIC MAXIMUM WIND PRESSURE MAP OF INDIA INCLUDING WINDS OF SHORT DURATION

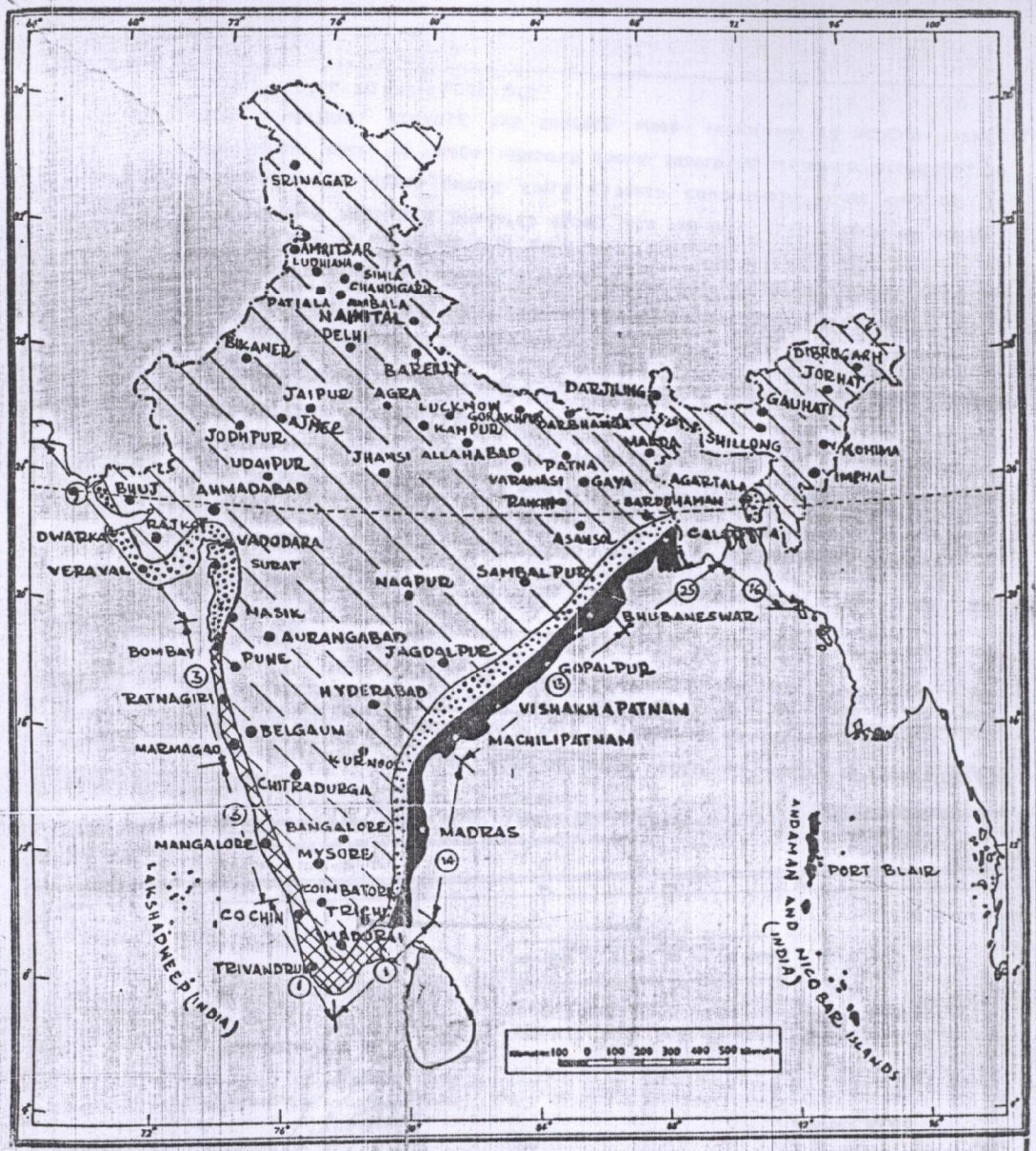
ZONE / PRESSURE IN kg/m² UPTO A HT. OF 30 M ABOVE MEAN RETARDING SURFACE	PRESSURES IN kg/m² AT A HEIGHT OF (EXPRESSED IN METRES)									
	35	40	45	50	60	70	80	100	120	150
100	104	105	108	111	115	118	122	127	132	138
150	156	158	163	167	172	177	183	191	198	207
200	208	210	217	222	230	236	244	254	264	276

(FOR INTERMEDIATE HEIGHTS INTERPOLATED VALUES MAY BE ADOPTED)

NOTE 1. FOR THE PURPOSE OF THIS MAP, A SHORT DURATION OF WIND IS THAT WHICH LASTS ONLY FOR A FEW MINUTES, GENERALLY LESS THAN 5 MINUTES.

NOTE 2. THE RELATIONSHIP BETWEEN WIND PRESSURE AND VELOCITY IS $p = KV^2$ WHERE p IS THE PRESSURE, V THE VELOCITY AND K IS A COEFFICIENT, THE VALUE OF WHICH DEPENDS UPON A NUMBER OF FACTORS, SUCH AS THE WIND SPEED, THE TYPE, PROPORTION AND SHAPE OF STRUCTURE AND THE TEMPERATURE OF AIR. IN THE PREPARATION OF THIS BASIC WIND PRESSURE WITH MAP, A VALUE OF 0.006 HAS BEEN ASSUMED FOR K AND p IS EXPRESSED IN kg/m^2 AND V IN KM/h.

NOTE 3. THE BASIC WIND PRESSURES INDICATED ABOVE ARE THE MAX. EVER LIKELY TO OCCUR IN THE RESPECTIVE AREAS, UNDER FULLY EXPOSED CONDITIONS. IN THE CASE OF MOUNTAINIOUS AREAS THE VALUES INDICATED ABOVE SHOULD BE MODIFIED ACCORDING TO LOCAL CONDITIONS BECAUSE THE SURFACE WIND IS KNOWN TO DEPEND MARKEDLY ON THE LOCAL TOPOGRAPHY ETC.



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

FIG. 2·1B. BASIC MAXIMUM WIND PRESSURE MAP OF INDIA EXCLUDING WINDS OF SHORT DURATION

ZONE	PRESSURE IN Kg/m² UPTO HT. OF 30 m	PRESSURE IN Kg/m² AT A HEIGHT OF, 35	40	45	50	60	70	8	100	120	150
	60	62	63	65	67	69	71	73	76	79	83
	100	104	105	108	111	115	118	122	127	132	138
	150	156	158	163	167	172	177	183	191	198	207
	200	208	210	217	222	230	236	244	254	264	276

(FOR INTERMEDIATE HEIGHTS INTERPOLATED VALUES MAY BE ADOPTED)

NOTE: 1, 2 & 3 SAME AS IN FIG. 2·1A

NOTE 4. THE NUMBER OF SEVERE CYCLONES WHICH HAVE APPROACHED OR CROSSED THE COASTS DURING 1891 TO 1960 IS INDICATED IN CIRCLES IN 5° LATITUDE ZONES (A SEVERE CYCLONE IS ONE IN WHICH THE WIND SPEED EXCEEDS 89 KM/h CORRESPONDING TO A WIND PRESSURE OF 48 Kg/m²). THE INFLUENCE OF A SEVERE CYCLONE MAY BE TAKEN TO EXTEND FROM THE COAST UP TO THE LINE DEMARCATING 60 Kg/m² ZONE.

APPENDIX 2.2

Permissible increase in allowable bearing pressure of soil

(Refer Clause 3.3.3. of IS 1893—1970 (Second Revision) Para 2.7.6)

Type of soil mainly constituting the foundation.	Permissible increase in allowable Bearing pressure, per cent			
	Bearing piles resting on soil type I or Raft Foundations	Friction piles combined or isolated R.C.C. footings with beams	Isolated footing without Tie-beams or unreinforced strip foundations	Well foundations.
1	2	3	4	5
Type I Rock or hard soils-well graded gravels and sand gravel mixtures with or without claybinder, and clayed sands poorly graded or sand clay mixtures (GB, CW, SB, SW SC)* having N** above 30 where N is the standard penetration value	50	50	50	50
Type II Medium soil-All soils with N betn. 10—30 and poorly graded sand or gravelly sands with little or no fines with N>15.	50	25	25	25
Type III. Soft soils-All soils other than SP* with N<10	50	25	0	25

NOTE 1 : The allowable bearing pressure shall be determined in accordance with IS 1904—1966 or IS 1888—1962.

NOTE 2 : If any increase in bearing pressure has already been permitted for forces other than seismic forces the total increase in allowable bearing pressure when seismic force is also included shall not exceed the limits speciefied above.

NOTE 3 : Loose sands and soils with standard penetration values<15 the vibrations caused by earthquake may cause liquefaction or excessive total and differential settlements. In important projects this aspect of the problem need be investigated and appropriate methods of compaction or stabilization adopted to achieve N more than 15.

Seismic Co-Efficients for Different Zones (Fig. 2.2)

Refer Clauses 3 4.1.1.,3.4.2.1. & 4.1. of IS 1893—197 0
(Second Revision)
Para 2.7.2.

ZONE NO	HORIZONTAL SEISMIC COEFFICIENT
V	0·08
IV	0·05
III	0·04
II	0·02
I	0·01

Depending upon the soil foundation systems on which the structure is founded, the horizontal seismic coefficient given above shall be multiplied by factor β.

Values of β for Different soil foundation system

Sl No. Type of soil mainly Constituting the foundation	Values of β for			
	Bearing piles resting on soil type I or raft foundation	Friction piles combined or Isolated RCC footings with TIE-Beams	Isolated footings without tie beams or unreinforced strip foundations	Well foundations
(i) Type I Rock or Hard soil.	1·0	1·0	1·0	1·0
(ii) Type II medium soils	1·0	1·0	1·2	1·2
(iii) Type III soft soils	1·0	1·2	1·5	1·5

Buildings provided or accommodating essential services which will be of post earthquake importance, e.g. hospitals emergency relief stores etc. shall be designed for one and a half times the seismic coefficients given above

* Refer IS 1498—1959

** Refer IS 2131—1963 (F. Revision)

For various loading classes as specified in IS 875—1964 (revised) the horizontal earthquake force shall be calculated for the full dead load and the percentage of live loads as given below :

Load Class	Percentage of Design Live load.
200,250,300 Stairs & ba con es	25
400,500,750 and 1000, Garage, Light and heavy	50

NOTE 1 : The percentage of Live Load given above shall also be used or calculating stresses due to vertical loads for combining with those due to earthquake forces. Under the earthquake conditions, the whole frame may be assumed loaded with Live Load except the roof.

NOTE 2 : If the Live Load is assessed instead of taking the above proportions for calculating horizontal earthquake force, only that par of the Live Load shall be considered which possesses mass. Earthquake force shall not be applied on impact effects.

For calculating the earthquake force on roofs, the live load may not be considered.

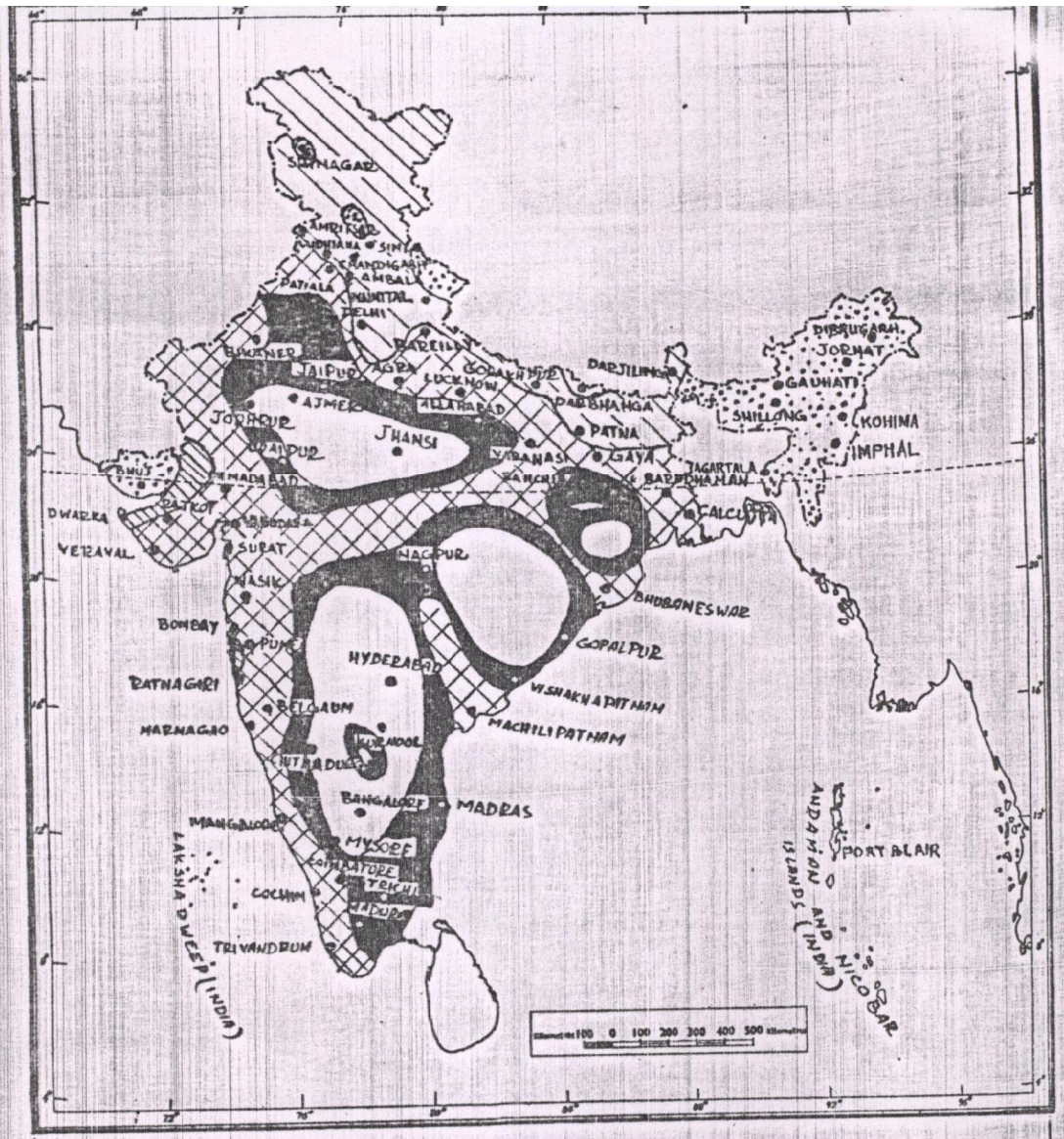
APPENDIX 2.3

Seismic coefficients for some important towns

TOWN	ZONE	HORIZONTAL SEISMIC COEFFICIENT	TOWN	ZONE	HORIZONTAL SEISMIC COEFFICIENT
Agra	III	0.04	Calcutta	III	0.04
Ahemedabad	III	0.04	Calicut	III	0.04
Ajmer	I	0.01	Chandigarh	IV	0.05
Allahabad	II	0.02	Chitradurga	I	0.01
Almora	IV	0.05	Coimbatore	III	0.04
Ambala	IV	0.05	Cuttack	III	0.04
Amritsar	IV	0.05	Darbhanga	V	0.08
Asansol	III	0.04	Darjeeling	IV	0.05
Aurangabad	I	0.01	Dehradun	IV	0.05
Bahriacin	IV	0.05	Delhi	IV	0.05
Bangalore	I	0.01	Durgapur	III	0.04
Barauni	IV	0.05	Gangtok	IV	0.05
Bareilly	III	0.04	Gauhati	V	0.08
Baroda	III	0.04	Gaya	III	0.04
Bhatinda	III	0.04	Gorakhpur	IV	0.05
Bhilai	I	0.01	Hyderabad	I	0.01
Bhopal	II	0.02	Imphal	V	0.08
Bhubaneswar	III	0.04	Jaipur	II	0.02
Bhuj	V	0.08	Jamashedpur	II	0.02
Bikaner	III	0.04	Jhashi	I	0.01
Bokar[illegible]	III	0.04	Jodhpur	I	0.01
Bomb[illegible]lista[illegible]	III	0.04	Jorhat	V	0.08
Bur[illegible]dwan	III	0.04	Jabalpur	III	0.04

TOWN	ZONE	HORIZONTAL SEISMIC COEFFICIENT	TOWN	ZONE	HORIZONTAL SEISMIC COEFFICIENT
Kanpur	III	0.04	Raipur	I	0.01
Kathmandu	V	0.08	Rajkot	III	0.04
Kohima	V	0.08	Ranchi	II	0.02
Kurnool	I	0.01	Roorkee	IV	0.05
Lucknow	III	0.04	Rourkela	I	0.01
Ludhiana	IV	0.05	Sadiya	V	0.08
Madras	II	0.02	Simla	IV	0.05
Mandurai	II	0.02	Sironj	I	0.01
Mandi	V	0.08	Srinagar	V	0.08
Mangalore	III	0.04	Surat	III	0.04
Monghyr	IV	0.05	Tezpur	V	0.08
Moradabad	IV	0.05	Thanjavur	II	0.02
Mysore	I	0.01	Tiruchirapalli	II	0.02
Nagpur	II	0.02	Trivendrum	III	0.04
Nainital	IV	0.05	Udaipur	II	0.02
Nasik	III	0.04	Varanasi	III	0.04
Nellore	II	0.02	Vijaywada	III	0.04
Panjim	III	0.04	Visakhapatnam	II	0.02
Patiala	III	0.04			
Patna	IV	0.05			
Pilibhit	IV	0.05			
Pondicherry	II	0.02			
Pune	III	0.04			

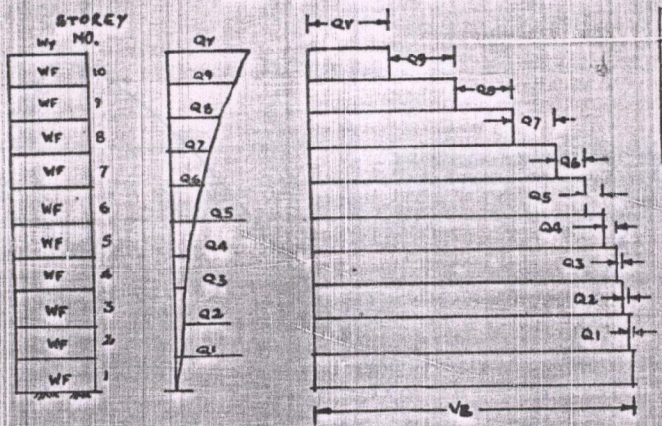
Note— The coefficients given are according to 3.4.2, 3.4.4. 4.3 and 7.1 and should be 3.4.1. and should be suitably modified for important structures according to 3.4.2, 3.4.4,4.3, and 7.1 and should be read along with others provisions of the standard.



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

MAP OF INDIA SHOWING SEISMIC ZONES.

ZONE	HORIZONTAL SEISMIC COFF.
V	0.08
IV	0.05
III	0.04
II	0.02
I	0.01



2A_FRAME 2B_DISTRIBUTION OF FORCES 2C_ DISTRIBUTION OF SHEARS

$$V_B = C.\alpha_h.\beta.(W_r + 9W_f)$$

$$Q_i = V_B \frac{W_i.h_i^2}{\sum_{i=1}^{i=n} W_i.h_i^2} \qquad V_j = \sum_{i=j}^{i=n} Q_i$$

WHERE V_j = SHEAR IN j^{th} STOREY

FORCE AND SHEAR DISTRIBUTION FOR TEN-STOREY BUILDING

FIG. 2.2

9—1 GPWD(ND)75

REFERENCES

1. IS : 875—1964, Indian Standard Code of Practice for Structural Safety of Buildings : Loading Standards (Revised), Reprinted January 1970.

2. IS : 1893—1970, Indian Standard Criteria for Earthquake Resistant Design of Structures (Second Revision), October 1971.

3. IS : 456—1964, Indian Standard Code of Practice for Plain & Reinforced Concrete (Second Revision), Second Reprint December 1970 (Incorporating Amendment No. 1,2,3 & 4).

4. IS : 3534—1965, Indian Standard Outline dimensions for Electric Lifts, July 1966.

CHAPTER 3

PRELIMINARY DESIGN

3.1 Introduction

3.1.1. The analysis of moments and forces in the various members of a multi-storeyed framed structure requires the prior knowledge of cross sectional dimensions so that the stiffnesses of the members can be computed before analysis is commenced. A preliminary estimate of approximate sizes of members is, therefore, to be made, based on judgement of the designer or on the basis of approximate calculations and thumb rules. As fairly good approximation of necessary sizes of the members is required, the priliminary design requires considerable labour and exercise of good judgement.

3.1.2. It is not possible to prescribe any firm rules for the preliminary determination of cross sectional dimensions because reinforced concrete frame-works can be of unlimited variety. However, the guidelines given in subsequent paras may be helpful. These may be modified by designer for the special conditions of any particular structure.

3.2. Slabs

3.2.1. The thickness of a slab is primarily determined from considerations of moment of resistance and limitation to deflection. However, the thickness of solid slab should not be less than 10 cm. For normal superimposed loads i.e. loads other than self-weight of slab, the thickness for different spans and boundary conditions is shown in Tables at Annexure 3-1 (for mild steel reinforcement) and Annexure 3-2 (for high yield strength reinforcement).

3.2.2. Slabs Spanning in one Direction

3.2.2.1. Simply supported one way slabs are designed for the specified uniformly distributed load for a positive bending moment of $\frac{Wl}{8}$ near midspan where W is the total uniformly distributed load over the span and l is the effective span. In addition, where a slab is built into brick or masonry walls which develop only partial restraint, the slab shall be designed to resist a negative moment of $\frac{Wl}{24}$ at the face of the support.

3.2.2.2. The $\frac{\text{Span}}{\text{depth (thickness)}}$ ratio of a simply supported slab may be limited to 26 if mild steel reinforcement is used and 20 for high-yield strenght bars.

3.2.3. Continuous Slab Spanning in one Direction

3.2.3.1. The bending moments in uniformly loaded slabs spanning in one direction continuous over three or more approximately equal spans (two spans may be considered approximately equal when they do not differ by 15% of the longer one) may be assumed to have the following values for preliminary design purposes.

	Moment due to Dead Load	Moment due to Live Load
(i) Near Middle of end span	$+\frac{W_d l}{12}$	$+\frac{W_s l}{10}$
(ii) A support next to the end support.	$-\frac{W_d l}{10}$	$-\frac{W_s l}{9}$
(iii) At other interior supports	$-\frac{W_d l}{12}$	$-\frac{W_s l}{9}$
(iv) At middle of interior spans	$+\frac{W_d l}{24}$	$+\frac{W_s l}{12}$

Where W_d = Total dead load per span.
W_s = Total uniformly distributed live load per span.
l = Effective span.

Positive and negative signs indicate sagging & hogging moments respectively.

3.2.3.2. In case of continuous slabs spanning in one direction, the span/depth ratio may be limited to 36 for mild steel and 28 for high yield strength bars.

3.2.4. TWO-WAY SLABS

3.2.4.1. The slabs will be assumed as spanning in two directions when they are supported on all four sides and the length is equal to or less than twice the width of slab. In other cases the slab will be assumed as spanning in one direction.

3.2.4.2. The amount of load that is transmitted in each direction of a slab panel will depend upon the relative length of the sides of the panel and the conditions of continuity that exist at the four edges.

3.2.4.3. The bending moments in two-way slabs may be found out as per procedure given in Annexure 2.3. The slabs should then be designed for these bending moments.

3.2.4.4. The span/Depth ratio in case of two-way slabs may be kept within the following limits:

Condition of support	Span/depth ratio	
	For M.S. bars	For High yield strength bars.
Simply supported	35	27
Continuous	40	31

In slabs spanning in two directions, the shorter of the two spans should be used to calculate span/depth ratio.

3.3 **Beams**

3.3.1. The section of a beam is determined from consideration of moment of resistance, resistance to shearing force and to deflection limitation. The ratio of length between lateral restraints and breadth (compression face) of a beam should not exceed 30 for simply supported beams and 12 cantilever beams. If these limits are exceeded, the beams will have to be designed as slender beams in which case permissible stresses will have to be reduced.

3.3.2. In building frames, the sizes of beams are usually governed by the negative moments and the shears at the supports, where their operative section is rectangular due to the fact that the flanges of the T beam are not effective.

3.3.3. SIMPLY SUPPORTED BEAMS

3.3.3.1. Simply supported beams are designed in a way similar to that adopted for design of simply supported slabs spanning in one direction.

3.3.3.2. The span/depth ratio of simply supported beams may be limited to 20 for M.S. and 16 for high yield strength bars.

3.3.4. CONTINUOUS BEAMS

3.3.4.1. The bending moments in uniformly loaded beams continuous over three or more approximately equal spans may be worked out in a way similar to that adopted for working out moments for continuous slabs spanning in one direction.

3.3.4.2. The shearing forces at the supports may be assumed to have the following values :

Location	Shearing force
(i) At exterior face of support next to end support	0·6 W
(ii) At faces of all other supports	0·5 W

3.3.4.3. The span/depth ratio in case of continuous beams may be limited to 25 for M.S. and 21 for high yield strength bars.

3.3.5. Loads on Beams Supporting Two-way Slabs

The loads on the beams supporting two-way slabs may be assumed to be in accordance with the following figure for calculating the reactions on supporting columns.

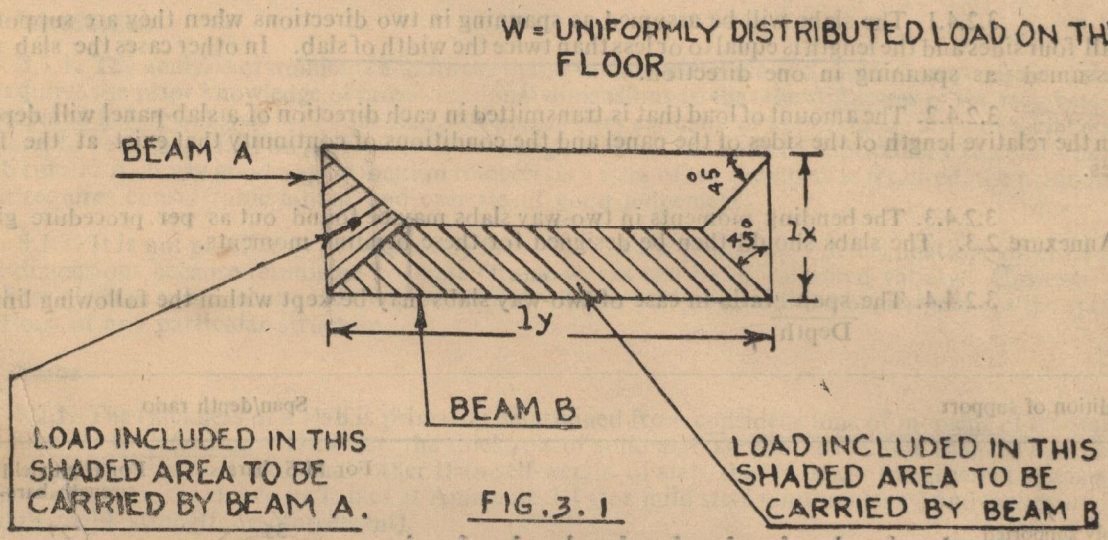


FIG. 3.1

$$\text{Total load on Beam A} = \frac{W.\,1x^2}{4}$$

$$\text{Total load on Beam B} = \frac{W.1x.1y}{2} - \frac{W1^2x}{4}$$

} For S.F. on beams and reactions on columns.

For computation of B.M. in beams an equivalent uniform load per lineal metre of beam may be takeen as :

$$\text{for Short span} = \frac{W.1x}{3}$$

$$\text{for long span} = \frac{W.1x}{3} \times \frac{(3 - m^2)}{2}$$

$$\text{where } m = \frac{\text{Short span}}{\text{Long span}}$$

3.3.6. Choice of Dimensions of Beams

Generally for determining a trial section for rectangular or T beams total depth may be taken as 1/12 the clear span. The breadth of a rectangular beam or the breadth of the rib of a T-beam generally varies from 1/3 to 2/3 times the total depth. Factors such as clearance below beams, cross-section areas required for resistance to shearing sizes of columns and architectural requirements may affect the sizes of the beams. However, it is desirable not to reduce the section keeping the beam somewhat under reinforced to avoid sudden failure due to crushing of concrete.

3.4. Flat slab Construction

3.4.1. The term flat slab means a reinforced concrete slab with or without drops, supported, generally without beams, by columns with or without flared column heads. A flat slab may be a solid slab or may have recesses formed on the soffit so that the soffit comprises of a series of ribs in two directions. The recesses may be formed by removable or permanent filler blocks. Flat plate is a special type of flat slab having neither drop panels nor column capitals.

3.4.2. Flat slabs may be designed :

(a) as continuous frames satisfying the principles of statics and continuity.

(b) by the empirical method as given in Para 13.10 of IS 456-1964 (Second Revision).

3.4.3. The ratio of average length of a panel to total thickness of a flat slab should not exceed the following values :

Panels	Ratio of average length of panel to thickness of slab	
	Mild steel bars.	High yield strength bars.
1. End panels without drops	32	25
2. Interior panels, fully continuous without drops and for end panels with drops	36	28
3. Interior panels, fully continuous, with drops	40	31

3.4.4. The critical sections for shear are at a distance from a column, column head, or drop equal to half the overall depth of slab or drop. The minimum thickness of flat slab should be 12.5 cm. Generally the thickness of flat plate is governed by considertion of resistance to shearing force.

3.5. Columns

3.5.1. For calculating the loads on columns the equivalent distributed load on beams resting on the supporting columns as mentioned in Para 3.3.5 may be taken.

3.5.2. A penultimate column support may be assumed to receive 0.5 times the load on each of the beams which connect it to the interior columns and 0.6 times the load on the beams which connect it with external columns. An external column may be assumed to receive 0.5 times the load on each of the beams connected with it.

3.5.3. The reduction in assumed total live loads on floors may be made in designing columns as per Appendix 2.1 of Chapter 2.

3.5.4. The bending moments in columns caused by vertical loads may be calculated for a few typical columns by formulae given in Annexure 3.4. For this purpose approximate sizes of columns may be worked out considering following thumb rule :—

> Cross-section areas of columns may be worked out taking 12 sq. cm and 15 sq. cm for every 1 Sq.m of floor supported for interior columns and exterior columns respectively for concrete of grade M 250.

3.5.5. The bending moments caused by horizontal loads may be found out for the same typical columns as selected in 3.5.4. by using approximate methods as indicated in Para 3.8 for analysis of frames for horizontal loads. These typical columns may then be designed for the direct loads and bending moments caused by horizontal and vertical loads.

3.5.6. It is preferable to use richer mixes like M 250 or even higher if controlled concrete is used in columns in order to effect economy in steel.

3.6. Shear Walls

3.6.1. Where wind forces predominate, the total horizontal shear force at the bottom of the building may be calculated for wind effects. However, where the earthquake force is controlling the design, the total approximate load coming on all columns at the base level is worked out. The base shear is then calculated using the formula given in Para 2-7-3 of Chapter 2.

3.6.2. Assuming that the whole base shear is resisted by the shear walls alone the area of the shear walls in the direction under consideration is equal to the base shear divided by the permissible shear stress in the concrete. This area of shear walls should be provided when the walls are placed symmetrically with respect to both horizontal axes of the structure. When the walls are not symmetrically placed, the area of shear walls should be recalculated taking into account the effect of torsion arising out of eccentricity.

3.7. Foundation

3.7.1. The size of footing to be adopted may be calculated taking the maximum calculated column load including the weight of the footing divided by the assumed bearing capacity of soil. If there is overlapping of adjacent footings, then combined or strip footings may be resorted to. When these are also not feasible then raft or pile foundation may be provided. While computing bending moments and shears only that part of the upward pressure is considered which is caused by the column load. In the absence of the sizes of the footings, weights of the footings may be taken as 5 per cent to 10 per cent of the column load in case of isolated footings, 10 to 15% in case of combined footings and 15 to 20% in case of raft foundation. The lower values are applicable to the soils having higher bearing capacity.

3.7.2. CHOICE OF TYPE OF FOUNDATION

3.7.2.1. The choice of type of foundation depends primarily on the depth at which the bearing stratum lies and the safe bearing pressure which determines the area of foundation. The choice also depends upon the loading encountered and limitations on the area available on one or more sides.

3.7.2.2. *Isolated Footings.*—An isolated R.C.C. column footing is most economical for concentric or slightly eccentric loads if the area exceeds about 1 Sq. m. For footings having base area less than 1 Sq.m a rectangular block of plain concrete is probably more economical. These footings may be square, rectangular or round in plan.

3.7.2.3. *Combined Footings.*—Two or more columns are provided with a common foundation known as combined footing under following circumstances :

(a) when two columns are so closely spaced that separate footings would overlap.

(b) An exterior column footing which cannot be made symmetrial because of property line limitations.

3.7.2.4. *Cantilever Footing or Strap Footing.*—When it is not possible to place an adequately sized footing centrally under a column or other load owing to restrictions of the site, and when under such conditions the eccentricity would result in inadmissible ground pressures, a cantilever footing is recommended. This case is common in the external columns of buildings on sites in built up areas.

3.7.2.5. *Raft Foundation.*—When the columns or other supports of a structure are closely spaced in both directions or when the column loads are so high and the safe ground pressure is low that it becomes impossible to accommodate the group of independent bases, a single raft foundation may be provided. Where basement has to be constructed, raft foundation is often preferable for the following reasons :—

(i) Basement can be more easily waterproofed.

(ii) Permissible settlement for raft being larger than that in case of isolated footings a higher bearing capacity can be adopted for the soil underneath:—

(iii) Raft helps in resisting uplift pressure of sub-soil water.

(iv) Design co-efficient for earthquake forces stipulated for the design of structure is lower in case of raft foundation.

However, the advantages gained by having raft foundation are offset by the extra cost due to the need for reinforcement at both faces of the raft.

3.7.2.6. *Pile Foundations.*—When the loading is heavy and the bearing strata lies far below the usual footing level, pile foundation is provided. Piles are effective in one of the following three ways :—

(i) Transferring the load through soft upper strata to end bearing on hard sub-strata.

(ii) As friction piles in their lower portions in transferring the load through soft upper strata into stiffer strata below.

(iii) As pure friction piles for their full length.

3.7.2.7. *Under-Reamed Piles.*—Under-reamed piles may be provided for foundation of structures in expansive soils or in poor soils overlying firm soil strata. The principle of this type of foundation is to anchor the structure at a depth where ground movements due to changes in mositure contents or consolidation of poor strata are negligible. Single under-reamed piles may be provided for foundations of lighter structures (upto two storeys) and multiple under-reamed piles for heavier structures in shallow as well as in deep layer of expansive soils.

3.7.3. PRELIMINARY DESIGN OF FOUNDATIONS

3.7.3.1. *Isolated Footings*

(a) The columns may be grouped into different groups depending upon the loads on them. The variation in column loads of a particular group may not be more than 10%. The footing may be designed for the maximum column load of that group.

(b) Divide the maximum column load at the footing plus the self weight of the footing by the permissible bearing capacity of soil to get the area of the footing. In the case of footings subjected to heavy moments, the area should be so proportioned as to limit the maximum stress in the soil within the permissible limit.

(c) The minimum thickness of the footing at the edge shall not be less than 15 cms. for footings on soils, nor less than 30 cms. for footings supported on piles.

(d) For the bending moment and shear force as a measure of diagonal tension and bond calculations, the net soil pressure which is caused by the column load alone should be considered.

(e) For footings supported on soils, the critical section for shear as a measure of diagonal tension will be at a distance equal to the effective depth of the footing from the face of the column. For footings supported on piles the critical section will be at a distance equal to half the effective depth of the footing from the face of the column.

(f) After locating the critical section, the footing may be designed assuming the width at the top of the critical section to be effective.

3.7.3.2. *Combined Footings*

(a) When more than one column or load is carried on a single base, the centre of gravity of the several loads should, if possible, coincide with the centre of the area of the base, in which case the pressure under the base is uniformly distributed. This is essential in order that, if any settlement should occur, it will be as nearly uniform as the character of the soil will allow. Footings may be rectangular or trapezoidal. While making the combination of two or more columns for a combined footing, it should be so planned that columns of similar type, such as corridor columns or outer columns are combined. A combination of columns on which the proportion of live load to dead load may differ widely should be avoided as far as possible. Combined footings may be of either beam and slab or solid slab construction. Beam and slab construction is generally more economical.

(b) Find out the area of the footing required by dividing the sum of the total column loads and weight of the footing by the safe allowable bearing pressure of the soil.

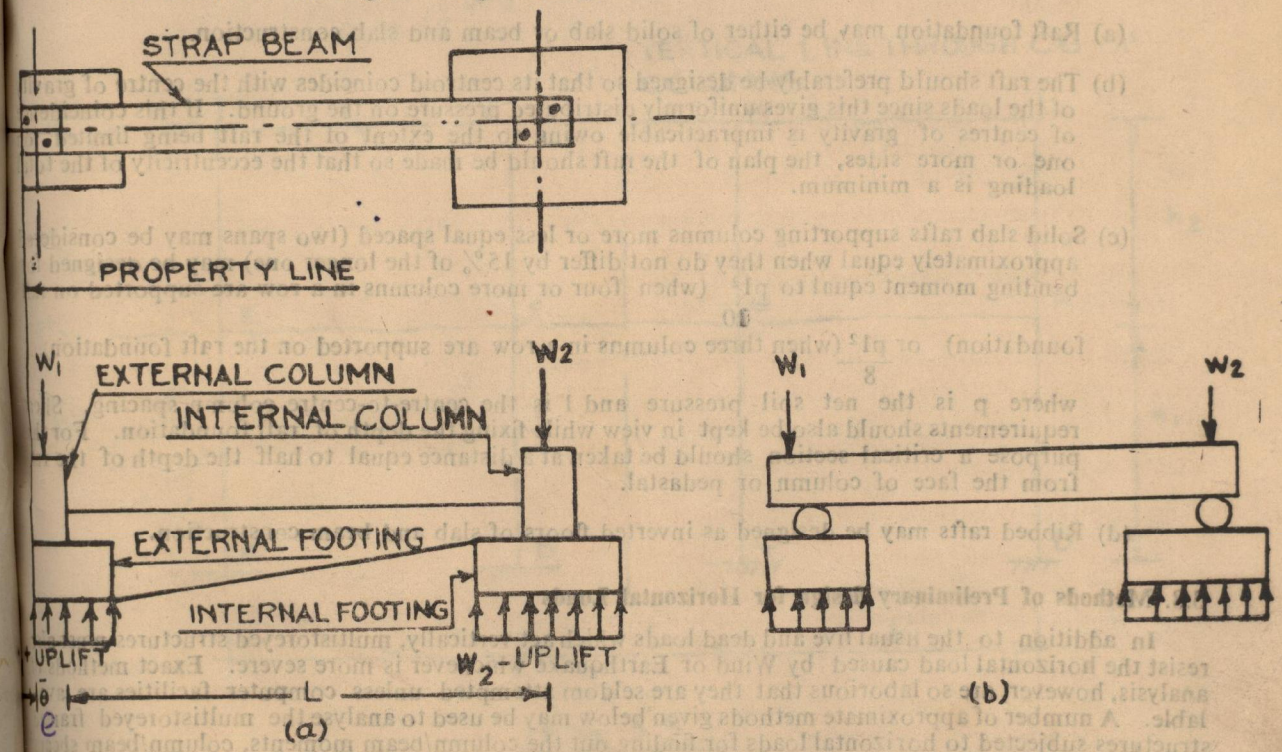
(c) Divide the shape of the footing required depending upon the spacing of columns and site restrictions.

(d) Find out the dimensions of the footing for the condition that the centre of gravity of the loads coincides with the centre of gravity of the footing to ensure uniform soil pressure under the footing.

(e) Find out the B.M. at each column face and at the point of zero shear by simple statics. Also find out the shear at the faces of columns.

(f) Calculate the depth of the footing taking maximum B.M. and shear into consideration.

3.7.3.3. *Cantilever Footing or Strap Footings*



AGRAM ILLUSTRATING (a) FORCES ACTING ON CANTILEVER FOOTING;
(b) PRINCIPLE OF CANTILEVER FOOTING.

FIG. 3.2

(a) A cantilever footing or a strap footing is a construction connecting the footings of an interi and an exterior column, the latter because of obstructions or load conditions being such as t have its C.G. eccentric with the C.G. of the column. A strap or beam is provided connecti the two footings to transfer the uplift caused by cantilevering the exterior column beyond th centre of the footing.

(b) The uplift at the footing for the interior column is :

$$\text{Uplift} = \frac{W_1 e}{L}$$

where W_1 = Load on the exterior column

and e = eccentricity of exterior column with respect to centre of gravity of its footing

L = Distance between centre of gravity of footings.

(c) The value of the uplift calculated above is the vertical shear carried by the strap. The total load on the exterior footing is increased and the total load in the interior footing is decreased by this amount. The total load to be provided for in the exterior footing will be (W_1 + uplift) and for the interior footing (W_2—uplift).

The max. moment in the strap beam = W_1X eccentricity.

The value of shear in the strap = Uplift.

The strap should be designed to resist the B.M. caused by the eccentricity and the maximum shear.

(d) The section of the strap at the exterior column will be determined by shear. A nominal amount of reinforcement should be placed in the bottom of the strap.

3.7.3.4. *Raft Foundation*

(a) Raft foundation may be either of solid slab or beam and slab construction.

(b) The raft should preferably be designed so that its centroid coincides with the centre of gravity of the loads since this gives uniformly distributed pressure on the ground. If this coincidence of centres of gravity is impracticable owing to the extent of the raft being limited on one or more sides, the plan of the raft should be made so that the eccentricity of the total loading is a minimum.

(c) Solid slab rafts supporting columns more or less equal spaced (two spans may be considered approximately equal when they do not differ by 15% of the longer one) may be designed for bending moment equal to $\frac{pl^2}{10}$ (when four or more columns in a row are supported on raft foundation) or $\frac{pl^2}{8}$ (when three columns in a row are supported on the raft foundation)

where p is the net soil pressure and l is the centre-to-centre column spacing. Shear requirements should also be kept in view while fixing the depth of raft foundation. For this purpose a critical section should be taken at a distance equal to half the depth of the raft from the face of column or pedastal.

(d) Ribbed rafts may be designed as inverted floors of slab and beam construction.

3.8. Methods of Preliminary design for Horizontal Loads

In addition to the usual live and dead loads which act vertically, multistoreyed structures must also resist the horizontal load caused by Wind or Earthquake whichever is more severe. Exact methods of analysis, however, are so laborious that they are seldom attempted unless computer facilities are available. A number of approximate methods given below may be used to analyse the multistoreyed framed structures subjected to horizontal loads for finding out the column/beam moments, column/beam shears and direct forces in columns.

(a) Approximate methods :

(i) Cantilever method.

(ii) Portal method.

(b) Improved methods :

(i) Bowman's method.

(ii) Factor method.

(iii) Muto's 'D' Value method.

In case of analysis by approximate methods, prior knowledge about sizes of columns and beams is not necessary. Hence these methods may be used in the initial stage in analysing the frames for horizontal loads. These methods are explained in Paras 3.8.1. and 3.8.2. Improved methods will be explained in Chapter 5.

3.8.1. (i) CANTILEVER METHOD

In the cantilever method, the following assumptions are made in order to render the building frames statically determinate :

(i) Points of inflection are located at the mid-span of girders;

(ii) Points of inflection are located at the mid-height of columns; and

(iii) Unit direct stresses in the columns vary as the distance of the columns from the centre of gravity of the bent. (If all the columns in a storey are of equal area, then the total axial forces in the columns will vary as the distances from the centre of gravity of the bent).

Step 1 : *Column Direct Stresses*

Find out the centre of gravity of the frame. When all the columns in a storey are of equal area, by assumption (iii), direct forces in columns will vary as their distances from the centre of gravity of the frame. Consider a buliding frame of figure 3.3 having all the columns in a storey of equal area.

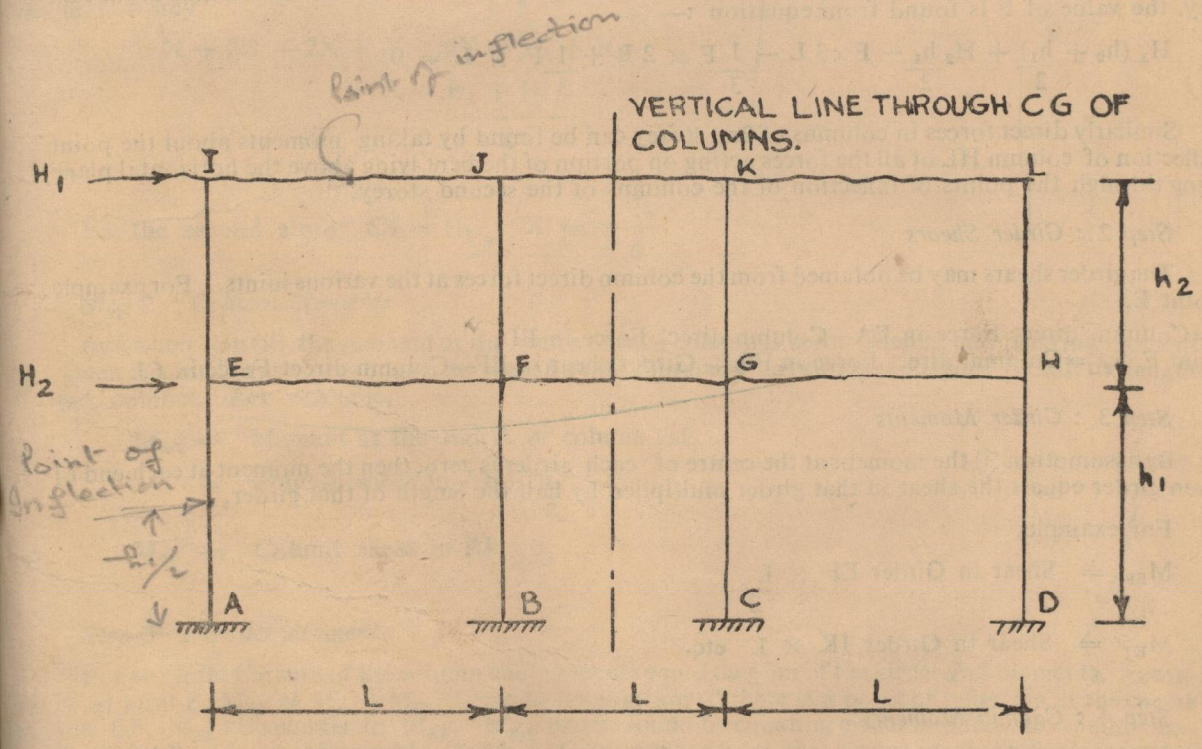


FIG. 3.3

For the first storey, if the axial force in column AE is denoted by F, then, by assumption (iii) the axial forces in columns BF, CG & DH will be $\frac{1}{3}$ F, (—) $\frac{1}{3}$ F and (—) F respectively. Taking moments about a, the point of inflection of column DH (Fig. 3.4), of all the forces acting on part of the

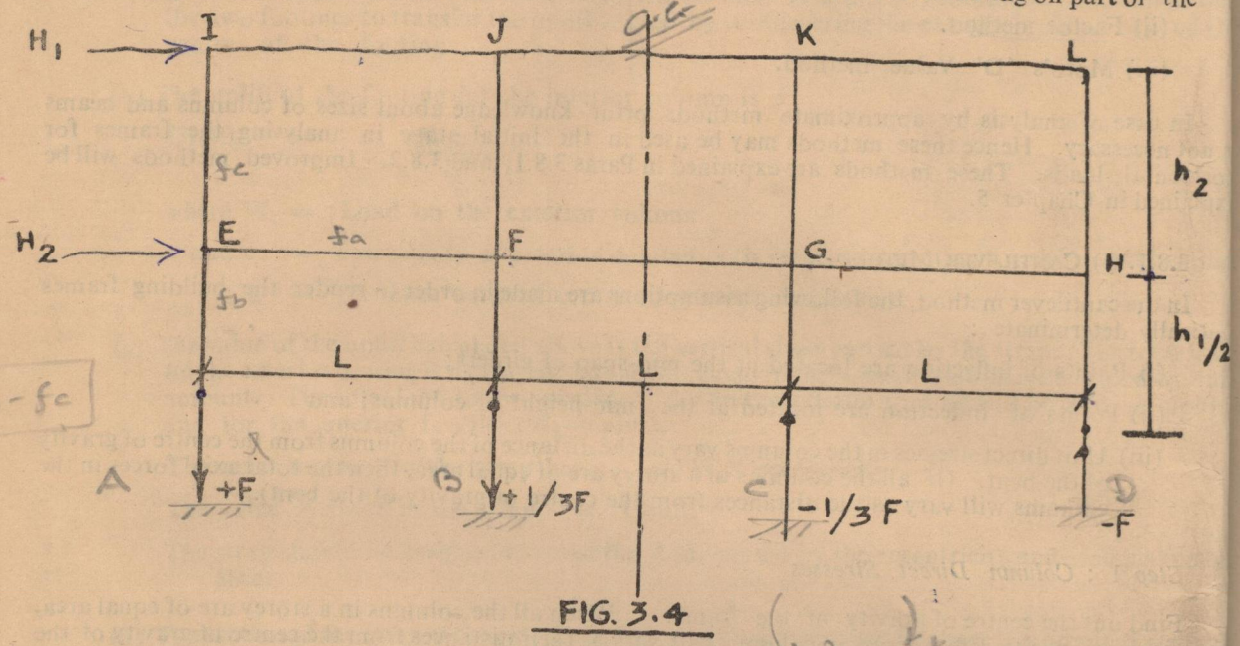


FIG. 3.4

frame laying above the horizontal plane passing through the points of inflection of the columns of the first storey, the value of F is found from equation :—

$$H_1\left(h_2 + \frac{h_1}{2}\right) + H_2\frac{h_1}{2} - F\times 3L - \frac{1}{3}F\times 2L + \frac{1}{3}F\times L = 0$$

Similarly direct forces in columns of 2nd storey can be found by taking moments about the point of inflection of column HL of all the forces acting on portion of the bent lying above the horizontal plane passing through the points of inflection of the columns of the second storey.

Step 2 : *Girder Shears*

The girder shears may be obtained from the column direct forces at the various joints. For example at joint E,

S_{EF}=Column direct Force in EA—Column direct Force in EI
at joint F, S_{FG}=Column direct Force in FB + Girder shear in EF—Column direct Force in FJ.

Step 3 : *Girder Moments*

By assumption (i) the moment at the centre of each girder is zero, then the moment at each end of a given girder equals the shear in that girder multiplied by half the length of that girder.

For example,

$$M_{EF} = \text{Shear in Girder EF} \times \frac{L}{2}$$

$$M_{KJ} = \text{Shear in Girder JK} \times \frac{L}{2} \text{ etc.}$$

Step 4 : *Column Moments*

Column moments are determined by begining at the top of each column stack and working progressively toward its base. For example, at joint J, the column moment equals the sum of the girder moments whence $M_{JF} = M_{JI} + M_{JK}$ etc.
Since, there is a point of inflection at the centre of FJ, M_{FJ} also equals M_{JF}. At joint F,

$M_{FB} + M_{FJ} = M_{FE} + M_{FG}$ from which M_{FB} can be found out. Further $M_{BF} = M_{FB}$ by assumption (ii).

Step 5 : *Column Shear*

In accordance to assumption (ii), shear in column AE = M_{AE} divided by $\frac{h_1}{2}$,

The sequence in which various values are found is as below :

(a) Column direct stresses/forces.

(b) Girder shears.

(c) Girder moments.

(d) Column moments and

(e) Column shears.

The application of the above method to a seven storeyed four bay frame will be explained in detail in Chapter 5.

3.8.2. (ii) PORTAL METHOD

In the PORTAL METHOD, the following assumptions are made :

(i) points of inflection are located at the mid span of girders;

(ii) Points of inflection are located at the mid height of columns; and

(iii) For any storey the shears in each of the interior columns are euqal. Similarly, the shears in the two exterior columns are equal. Shear in each exterior column is equal to half the shear in any of the interior columns of that storey.

Step 1 : *Column Shear*

Consider a building frame of Fig. 3.3. In accordance with assumption (iii), let X = shear in each exterior column of a given storey : then 2X = shear in each interior column of the same storey. For the first storey ;

$$X + 2X + 2X + X = 6X = H_1 + H_2$$

$$X = \frac{H_1 + H_2}{6}$$

For the second storey $6X = H_1$, $X = \frac{H_1}{6}$

Step 2 : *Column Moments*

By assumption (ii), the moment at the centre of each column is zero. Hence, end bending moment for a given column in any storey equals the shear on that column in that storey multiplied by half the length of that column. For example,

M_{AE} = Moment at the end A of column AE

= Column shear in AE $\times \frac{h_1}{2}$

M_{FJ} = Column shear in FJ $\times \frac{h_2}{2}$

Step 3 : *Girder Moments*

For any joint the sum of the column end moments equal the sum of the girder end moments. Accordingly at joint E, $M_{EF} = M_{EA} + M_{EI}$. Since by assumption (i) there is a point of inflection at the centre of girder EF, M_{FE} also equals to M_{EF}. M_{FG} can be found by equating girder moments to column moments at joint F, $M_{FG} + M_{FE} = M_{FB} + M_{FJ}$. Continuing across the girders of the first floor in this manner, all the end moments in the girders of the first floor will be found equal to M_{EF}. Girder end moments in the roof may be determined in a similar manner.

Step 4 : *Girder Shears*

By assumption (i) the moment at the centre of each girder is zero. Hence girder shear in EF = $\frac{M_{EF}}{L/2}$.

Step 5 : Direct Forces in Columns

The axial forces in the columns may be obtained by summing up, from the top of the column, the reactions applied to tne column by the girder shears.

Thus, Direct force in column EI = Shear in Girder IJ

Direct force in column AE = Girder shear in EF + column direct force in EI etc.

The sequence in which various values are found is as below :

(a) Column shears

(b) Column moments

(c) Girder moments

(d) Girder shears and

(e) Column direct forces.

The application of the above method to a seven storeyed four bay frame will be explained in detail in chapter 5.

ANNEXURE 3.1

Clause 3.2.1

Minimum slab thickness in Cm (for Mild Steel)

Support condition	Short Spans (Metres)												
	2·0	2·5	3·0	3·5	4·0	4·5	5·0	5·5	6·0	6·5	7·0	7·5	8·0
1. Simply supported slabs spanning in one direction . .	10	10	12	14	16	18	20	22	24	25	27	29	31
2. Continuous slabs spanning in one direction :													
(a) one end continuous .	10	10	10	12	13	15	17	18	20	21	23	25	26
(b) both ends continuous .	10	10	10	10	12	13	14	16	17	19	20	21	23
3. Cantilever slabs . . .	16	20	24	27	31	35	39	43	47	50	54	58	62
4. Simply supported slabs spanning in two directions . . .	10	10	10	10	12	13	15	16	18	19	20	22	23
5. Continuous slabs spanning in two directions	10	10	10	10	10	12	13	14	15	17	18	19	20
6. Flat Slabs :													
(a) End panels, without drops	12·5	12·5	12·5	12·5	12·5	15	16	18	19	21	22	24	25
(b) Interior panels, fully continuous, without drops and for end panels with drops	12·5	12·5	12·5	12·5	12·5	12·5	14	16	17	19	20	21	23
(c) Interior panels, fully continuous, with drops .	12·5	12·5	12.5	12.5	12·5	12·5	12·5	14	15	17	18	19	20

ANNEXURE 3.2.

Clause 3.2.1.

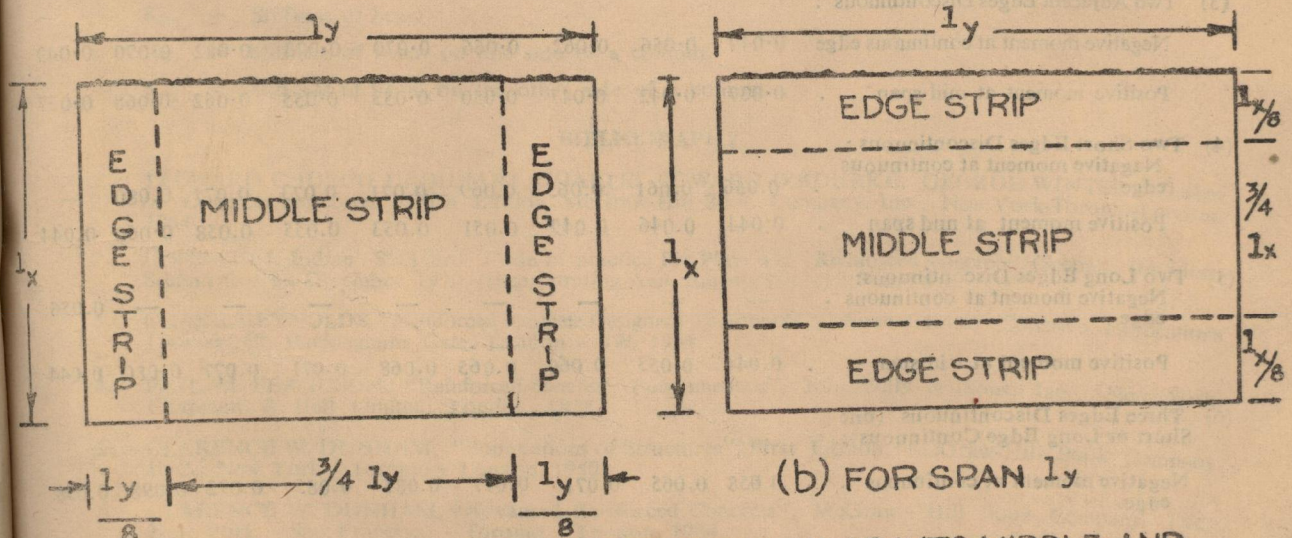
Minimum slab thickness in CM (for torsteel)

Support condition	Short Spans (Metres)												
	2·0	2·5	3·0	3·5	4·0	4·5	5·0	5·5	6·0	6·5	7·0	7·5	8·0
1. Simply supported slabs spanning in one direction . . .	10	13	15	18	20	23	25	28	30	33	35	38	40
2. Continuous slabs spanning in one direction :													
(a) one end continuous . .	10	11	13	15	17	19	21	23	25	28	30	32	34
(b) both ends continuous .	10	10	11	13	15	17	18	20	22	24	25	27	29
3. Cantilever slabs . . .	20	25	30	35	40	45	50	55	60	65	70	75	80
4. Simply supported slabs spanning in two directions . . .	10	10	12	13	15	17	19	21	23	25	26	28	30
5. Continuous slabs spanning in two directions	10	10	10	12	13	15	17	18	20	21	23	25	26
6. Flat Slabs :													
(a) End panels without drops .	12·5	12·5	12·5	14	16	18	20	22	24	26	28	30	32
(b) Interior panels, fully continuous, without drops and for end panels with drops . .	12·5	12·5	12·5	12·5	15	17	18	20	22	24	25	27	29
(c) Interior panels, fully continuous, with drops . .	12·5	12·5	12·5	12·5	13	15	17	18	20	21	23	25	26

ANNEXURE 3.3

Bending Moments in Two-way Slabs

(a) Slabs are considered as being divided in each direction into middle strips and edge strips as shown in Fig. 3.5, below, the middle strips having a width of three-quarters the width of the slab and each edge strip having a width of one-eighth of the width of the slab, with the proviso that no edge strip should exceed $\frac{l_x}{2}$ in width.



DIVISION OF A SLAB INTO MIDDLE AND EDGE STRIP

FIG. 3.5

(b) The maximum bending moments per unit width in the middle strip of a slab are given by the following equations :

$M_x = Z^1_x \, w l^2 x$, and

$My = Z^1y^2 \, w \cdot lx^2$ $Z^1y \, w l_x^2$

where

Z^1x, Z^1y = coefficients shown in table below :

Bending moment coefficients for rectangular panels supported on four sides with provision for torsion at corners

Clause 3.2.4.3.

This refers to Cl. C-3.1 of IS 456—1964 (Second Revision).

Case No.	Type of Panel and Moments	Bending Moment Coefficient Z'x For Short Span For Values of $\frac{l_y}{l_x}$						Bending Moment Coefficient Z'y For Long Span for All values of $\frac{l_y}{l_x}$		
		1·0	1·1	1·2	1·3	1·4	1·5	1·75	2·0 or more	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
(1)	Interior Panels :									
	Negative moment at continuous edge	0·033	0·040	0·045	0·050	0·054	0·059	0·071	0·083	0·033
	Positive moment at mid-span .	0·025	0·030	0·034	0·038	0·041	0·045	0·053	0·062	0·025
(2)	One Short or Long Edge Discontinuous :									
	Negative moment at continuous edge	0·041	0·047	0·053	0·057	0·061	0·065	0·075	0·085	0·041
	Positive moment at mid-span .	0·031	0·035	0·040	0·043	0·046	0·049	0·056	0·064	0·031
(3)	Two Adjacent Edges Discontinuous :									
	Negative moment at continuous edge	0·049	0·056	0·062	0·066	0·070	0·073	0·082	0·090	0·049
	Positive moment at mid-span .	0·037	0·042	0·047	0·050	0·053	0·055	0·062	0·068	0·037
(4)	Two Short Edges Discontinuous : Nagative moment at continuous edge .	0.056	0.061	0.065	0.069	0.071	0.073	0.077	0.080	—
	Positive moment at mid span .	0·044	0.046	0.049	0.051	0.053	0.055	0.058	0·060	0.044
(5)	Two Long Edges Discontinuous: Negative moment at continuous edge .	—	—	—	—	—	—	—	—	0.056
	Positive moment at mid span .	0.044	0.053	0.060	0.065	0.068	0.071	0.077	0.080	0.044
(6)	Three Edges Discontinuous (one Short or Long Edge Continuous):									
	Negative moment at continuous edge. .	0.058	0.065	0.071	0.077	0.081	0.085	0.092	0.098	0.058
	Positive moment at mid span .	0.044	0.049	0.054	0.058	0.061	0.064	0.069	0.074	0.044
(7)	Four Edges Discontinuous : . Positive moment at mid span.	0.050	0.057	0.062	0.067	0.071	0.075	0.081	0.083	0.050

ANNEXURE 3.4

Moment in Columns

Clause 3.5.4.

This refers to Cl. 11.3.1. of IS 456—1964 (2nd Revision)

LOCATION FOR MOMENT	MOMENTS FOR FRAMES OF ONE BAY	MOMENTS FOR FRAMES OF TWO OR MORE BAYS
(1)	(2)	(3)
External (and similarly loaded) columns :		
Moment at foot of upper column	$M_e \frac{K_u}{K_1+K_u+0.5K_b}$	$M_e \frac{K_u}{K_1+K_u+K_b}$
Moment at head of lower column	$M_e \frac{K_1}{K_1+K_u+0.5K_b}$	$M_e \frac{K_1}{K_1+K_u+K_b}$
Internal columns :		
Moment at foot of upper column	—	$M_{es} \frac{K_u}{K_1+K_u+K_{b1}+K_{b2}}$
Moment at head of lower column	—	$M_{es} \frac{K_1}{K_1+K_u+K_{b1}+K_{b2}}$

NOTE 1.—For the purpose of this table, 'stiffness' of a member may be obtained by dividing the moment of inertia of a cross section by the length of the member, provided that the member is of constant cross section throughout its length.

NOTE 2.—The equation for the moment at the head of the lower column may be used for columns in a topmost storey by taking K_u as zero.

NOTE 3.— M_e = Bending moment at the end of beam framing into a column assuming fixity at connection.

M_{es} = The maximum difference between the moments at the ends of two beams framing into opposite sides of a column, each calculated on the assumption that the ends of the beams are fixed and that one of the beams is not loaded.

K_u = Stiffness of upper column.

K_1 = Stiffness of lower column.

K_b = Stiffness of beam.

K_{b1} = Stiffness of beam on one side of a column.

K_{b2} = Stiffness of beam on the other side of a column.

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CHAPTER 4

METHODS OF ANALYSIS OF FRAMED STRUCTURES FOR VERTICAL LOADS

4.0 Notation

C	Carry-over factor
D	Distribution factor
δ	Displacement of a member.
E	Modulus of elasticity of concrete
h_{ab}	Height of the column AB.
I	Moment of Inertia of a member.
K_a	Twice the sum of the stiffnesses of all the members meeting at a joint A.
K_{ab}	Stiffness of a member AB.
~~l,r~~ l, r	Subs cripts indicating the left and right ends of a member e.g. D_l = Distribution factor at the left end of the member.
l_{ab}	Length of a member AB.
M_{ab}	End moment for the end A of the member AB.
M_{ba}	End moment for the end B of the member AB.
$\bar{M}_{ab}$	Fixed-end moment produced by the given exernal loading at the end A of member AB.
$\bar{M}_{ba}$	Fixed-end moment produced by the given external loading at the end B of member AB.
$\bar{M}_a$	Sum of the fixed-end moments at joint A.
M'_{ab}	Moment produced by the rotation of end A.
M'_{ba}	Moment produced by the rotation of end B.
M''_{ab}	Moment produced due to the liner displacement of one end of a member with respect to the other.
m_a	rotation moment due to θ_a.
m_b	rotation moment due to θ_b.
$\bar{M}_{ab}$	Displacement moment.
$\triangle m_{ab}$, $\triangle m_{ba}$	The terms of correction for end moments and expressed as functions of the changes in the joint rotation angles and of the joint displacement angles.
Q_{ab}	Shear force in a column AB.
T	Twice the stiffnesses of all the columns of a particular storey.
t_{67}	3/2 times the stiffness of a column 67 divided by the sum of the stiffnesses of all the columns of the storey containing column 67.
θ_a	Rotation of the end A.
θ_b	Rotation of the end B.
γ_{ab}	Stiffness of a member AB divided by sum of the stiffnesses of all the members meeting at A.

4.1 General

4.1.1 The skeleton frame work of a multistoreyed R.C.C. framed structure is made up of a system of columns, beams and slabs. It is presumed that the reinforcements are always so arranged that all joints of the frame are monolithic.

In view of the uncertain property of material, creep, shrinkage and a number of approximate simplifying assumptions made in the detailed analysis of multistoreyed framed structures (such as conditions of end restraints etc.) it is considered sufficient to obtain reasonable accuracy of analysis for the design of structure. If the normal moment distribution is applied to all joints, the work involved is enormous. However with certain assumptions, it is possible to analyse the frames and get results which will be adequate for design purposes.

To simplify analysis three dimensional multistoreyed R.C.C. framed structures are considered as combinations of planar frames in two directions. It is assumed that each of these planer frames act independently or other frames.

4.1.2. Degree of Accuracy in Calculations

The design of R.C.C. structures does not require a high degree of precision in calculations. We allow certain variations in the strength of concrete test cubes and reinforcing bars, form work and placing of reinforcement. Moreover, the computations are done with the help of a 10″ slide rule with which we can read upto three significant figures. When numbers are substracted, significant figures are often lost. The time and efforts of the designers need not be wasted in striving for a high degree of a mathematical precision by carrying figures to an excessive number of places. It is, therefore, recommended that figures be carried to three significant places or to the extent of a 10″ slide rule.

There is no point in computing loads to a fine precision only to lose results in a moment computation; nor is it logical to carry moments to the suggested 3 significant numbers when the loads are estimated to one figure precision. The following table is suggested as a rough guide to give some indication of a satisfactory procedure.

Record values to the following precision :

Loads to nearest 5 Kg/Sq. m; 10 Kg/m; 50 Kg concentration. Span lengths to 0.01m.

Total loads and reactions to 0.05 Tonnes or 3 significant figures.

Moments to nearest 1 Tcm or to 3 significant figures.

Reinforcement bar areas to 0.05 cm^2.

Member sizes to 1 cm.

Effective beam depth to 0.1 cm.

4.2 Methods of Analysis

Analysis of large framed structures became too cumbersome with the classical methods of structural analysis such as Clapeyron's theorem of three moments, Castigliano's theorem of least work. Poison's method of virtual work and Maney's method of slope deflection. Therefore, it became necessary to evolve simpler methods. Some of these are :

(a) Hardy cross method of moment distribution,

(b) Two cycle method of moment distribution,

(c) Kani's method of iteration,

(d) Takabeya's method of iteration,

(e) Kloucek's method of distribution of deformation.

Certain basic concepts required in these methods are explained in Paras 4.3 to 4.7. The methods are explained in Paras 4.8 to 4.12.

4.3 Stiffness

The stiffness of a member is defined as the value of the moment required to rotate the near end of the member through unit angle. The stiffness of a beam is :

$K = 4EI/L$ with far end fixed.

$K = 3EI/L$ with far end simply supported or hinged.

where K = Absolute stiffness

E = Modulus of elasticity

I = Moment of inertia of the cross-section about its neutral axis.

L = Length of the member.

In order to determine the distribution of moments at a joint of a structure it is only relative values of the stiffnesses of members that are required. Therefore, for the analysis of a frame we calculate the relative stiffnesses of members.

While calculating moment of intertia of the cross-section of a member we consider only concrete section neglecting the steel reinforcement. The moment of inertia thus arrived is multiplied by a factor to account for the steel reinforcement in the case of columns. For normal cases this factor may be taken as 1.1. In case of L & T beams the moment of intertia of the concrete section can be obtained by using Table 4.1. The usual practice is to calculate the moment of inertia of rectangular sections and to multiply it by 1.5 and 2 in case of L and T beam respectively.

4.4 Distribution Factor

It is the ratio of the stiffness K of the member under consideration to the sum of the stiffnesses of all members meeting at the joint. This is expressed as :

$$\text{Distribution factor} = DF = K/\Sigma K$$

Where K is the stiffness of the particular member and ΣK is the sum of all member stiffnesses. In the above expression, the relative stiffness factors may be used. The sum of all distribution factors for the members meeting at a joint must be 1.

4.5 Carry-over Factor

When a moment M_a is applied to one end of a beam fixed at the other end, there will be a moment M_b called the carry-over moment, induced at the far end. The ratio of this carry-over moment to the moment applied at the near end is called the carry-over factor.

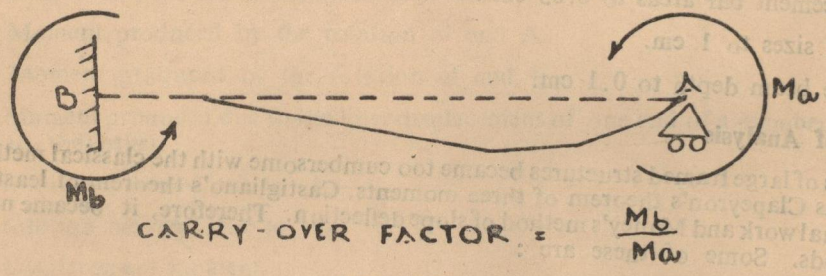


FIG 4.1

When the beam is of uniform cross-section, the carry-over factor is $\frac{1}{2}$. In the case of beam with far end hinged there will be no carry-over if the relative stiffness is taken as $3/4 \frac{EI}{l}$.

4.6 Fixed-End-Moments

The fixed end moments are the moments which must be applied simultaneously at both ends of a span in order to prevent the rotation of those ends when the span is loaded. These moments, for all usual cases of loading, and for prismatic beams can be obtained from the formulae given in Annexure 4.1.

4.7 Sign Convention

The following sign convention has been adopted in the manual.

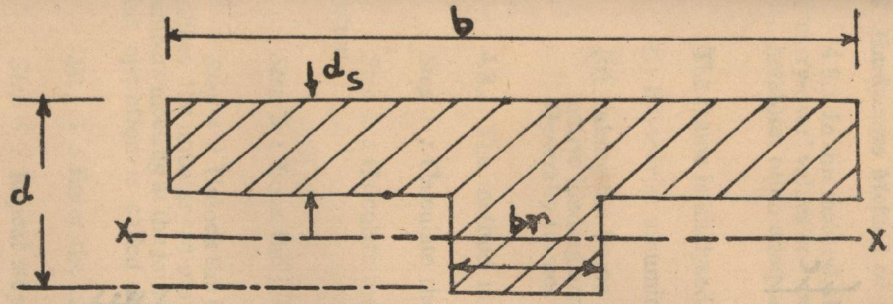
4.7.1. End Moments

A moment acting on a joint is positive in the counter clock-wise direction and negative in the clock-wise direction. For example, in Fig. 4.2 the moment of 400 acting on the joint B is negative and the moment of 300 is positive.

TABLE 4.1

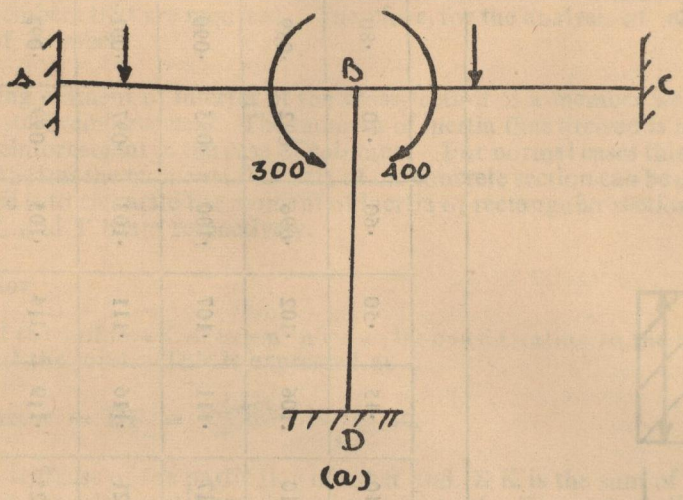
MOMENT OF INERTIA OF TEE SECTION.
(ALSO APPLICATION TO ELL SECTION & INVERTED CHANNEL)
MOMENT OF INERTIA ABOUT AXIS X—X
THROUGH CENTLODD

$$I_{xx} = Cb_r d^3$$

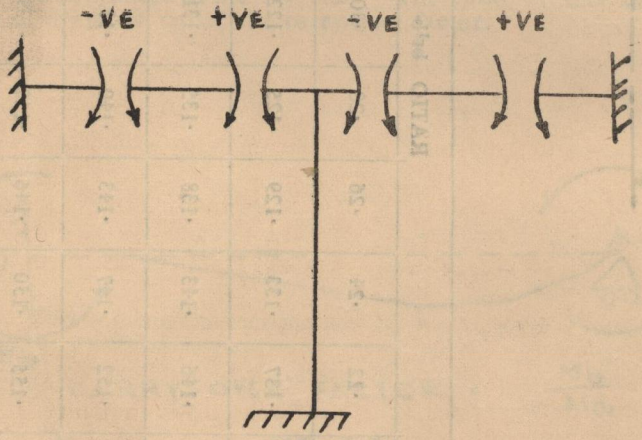


VALUES OF C ARE TABULATED

RATIO ds/d	RATIO br/b (b'/b)																
	·12	·14	·16	·18	·20	·22	·24	·26	·28	·30	·35	·40	·45	·50	·60	·70	·80
0·10	·170	·161	·154	·147	·142	·137	·133	·129	·125	·122	·115	·110	·106	·102	·096	·092	·089
0·15	·180	·172	·165	·158	·152	·146	·143	·138	·135	·131	·124	·117	·111	·107	·100	·095	·090
0·20	·183	·175	·169	·163	·157	·152	·147	·143	·140	·137	·128	·121	·116	·111	·103	·097	·091
0·30	·184	·176	·170	·164	·159	·155	·150	·146	·142	·139	·132	·125	·119	·114	·105	·099	·093
0·40	·190	·180	·173	·165	·160	·155	·151	·146	·142	·139	·132	·125	·119	·115	·106	·099	·093
0·45	·197	·185	·176	~~·166~~ 0.169	·162	·156	·152	·147	·143	·140	·132	·125	·119	·115	·106	·099	·093
0·50	·208	·195	·183	·174	·167	·160	·155	·150	·146	·141	·133	·126	·120	·115	·106	·099	·093



(a)



(b)

FIG 4.2

4.7.2. Span Moments

Span moment is considered positive when the top of the beam over which it is acting is in the compression.

4.7.3 Moments Due to Sinking or Lateral Sway

Counter clockwise rotation of the chord connecting the ends of the member causes positive moments at both ends of the member. Clockwise rotation of the chord connecting ends of the member causes negative moments at both ends of the member. Thus in case of a beam if the left hand side support of the beam is lower than its right hand side support, the moments caused due to this relative movement are positive and if the left hand side support of a beam is higher than its right hand side support, the moments caused due to this relative movement are negative.

4.8 Hardycross Method of moment distribution

4.8.1 In this method, the 'balancing' and 'carry-over' constitute one cycle and it has been found that the 'carry-over' values converge fast enough to become quite insignificant after four cycles of operation. It is, therefore, often adequate to stop the computations after four cycles.

The frame is analysed by this method either :

(i) Floor-wise assuming the columns to be fixed for ends or

(ii) Taking the frame as a whole. The whole frame analysis can be carried out for several alternative loading arrangements for obtaining maximum positive and negative bending moments. Generally frames are analysed floor-wise for the worst conditions of loading.

4.8.2. The method is described in the following steps :

Step 1 : Calculate the stiffnesses of all members. Enter them in the calculation scheme.

Step 2 : Calculate the distribution factors at all joints from the stiffnesses. Enter them into the scheme.

Step 3 : Lock the joints and calculate the fixed-end moments. Enter them into the scheme.

Step 4 : Unlock the joints one by one by applying imaginary external moment at each joint which nullifies the unbalanced moment at the joint. Distribute the imaginary external moment among all members meeting at the joint in proportion to their relative stiffnesses and enter these values in the scheme. This operation is called balancing.

Step 5 : Enter the carry-over moments at the far ends in the scheme.

Step 6 : Repeat steps 4 & 5 till the carry-over moments become insignificant.

Step 7 : Balance the unbalanced moments obtained from the last carry-over operation.

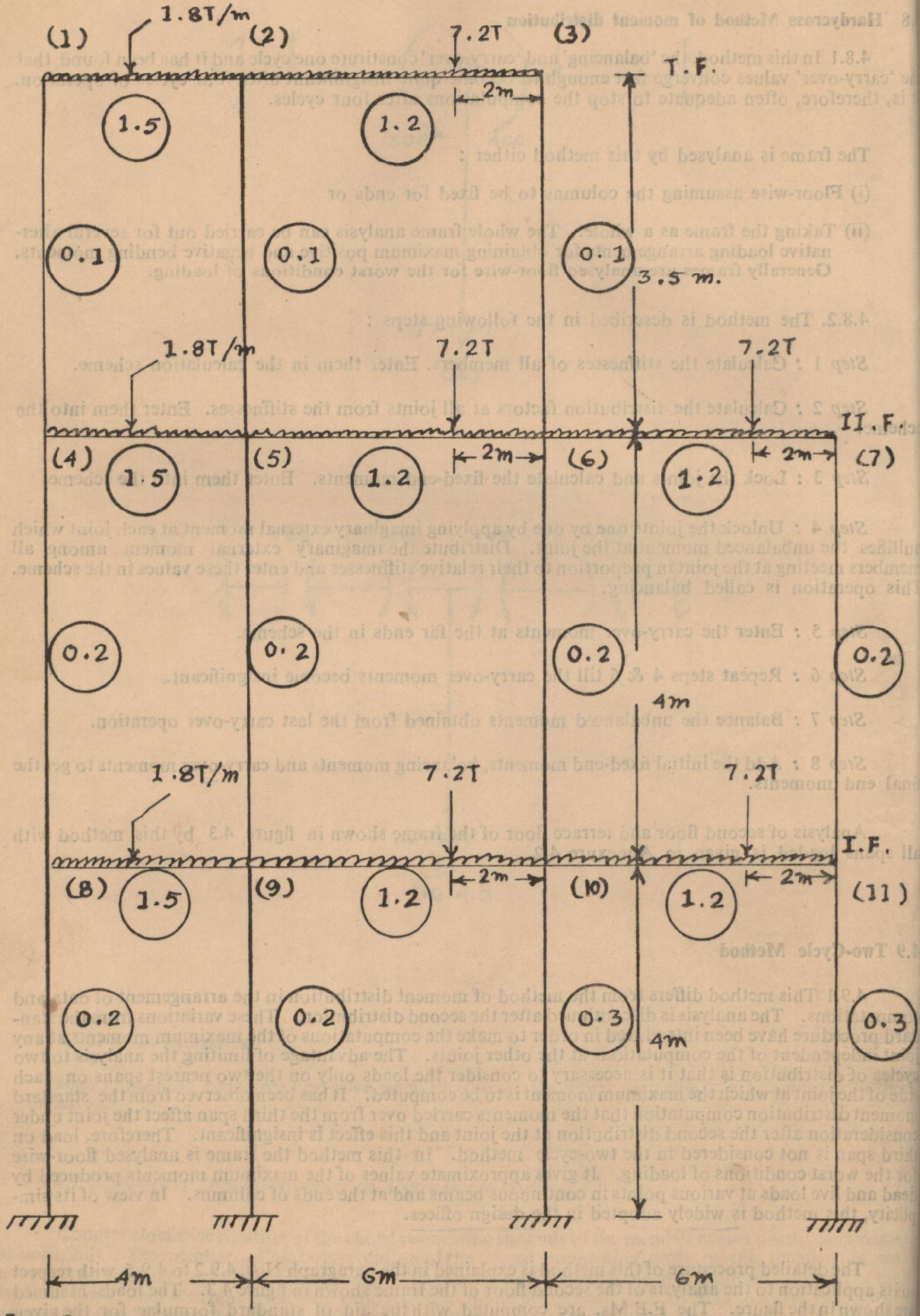
Step 8 : Add the initial fixed-end moments, balancing moments and carry-over moments to get the final end moments.

Analysis of second floor and terrace floor of the frame shown in figure 4.3 by this method with all spans loaded is given in Annexure 4.2.

4.9 Two-Cycle Method

4.9.1 This method differs from the method of moment distribution in the arrangement of data and computations. The analysis is discontinued after the second distribution. These variations from the standard procedure have been introduced in order to make the computations of the maximum moments at any joint independent of the computations at the other joints. The advantage of limiting the analysis to two cycles of distribution is that it is necessary to consider the loads only on the two nearest spans on each side of the joint at which the maximum moment is to be computed. It has been observed from the standard moment distribution computation that the moments carried over from the third span affect the joint under consideration after the second distribution at the joint and this effect is insignificant. Therefore, load on third span is not considered in the two-cycle method. In this method the frame is analysed floor-wise for the worst conditions of loading. It gives approximate values of the maximum moments produced by dead and live loads at various points in continuous beams and at the ends of columns. In view of its simplicity, this method is widely adopted in the design offices.

The detailed procedure of this method is explained in the paragraph Nos. 4.9.2 to 4.9.5. with respect to its application to the analysis of the second floor of the frame shown in figure 4.3. The loads assumed are shown in the figure. The F.E.Ms. are computed with the aid of standard formulae for the given type of loading.



NOTE: STIFFNESS VALUES OF THE MEMBERS ARE WRITTEN AT THE CENTRE OF THE CORRESPONDING MEMBERS IN CIRCLES

FIG 4.3

4.9.2 Maximum Moments at Joints

4.9.2.1. In order to make clear the method of computations at every joint, calculations of the moments at the joints in 2nd floor (Fig. 4.4) are shown in table 4.2. It has been shown later that these calculations can be combined in a single table.

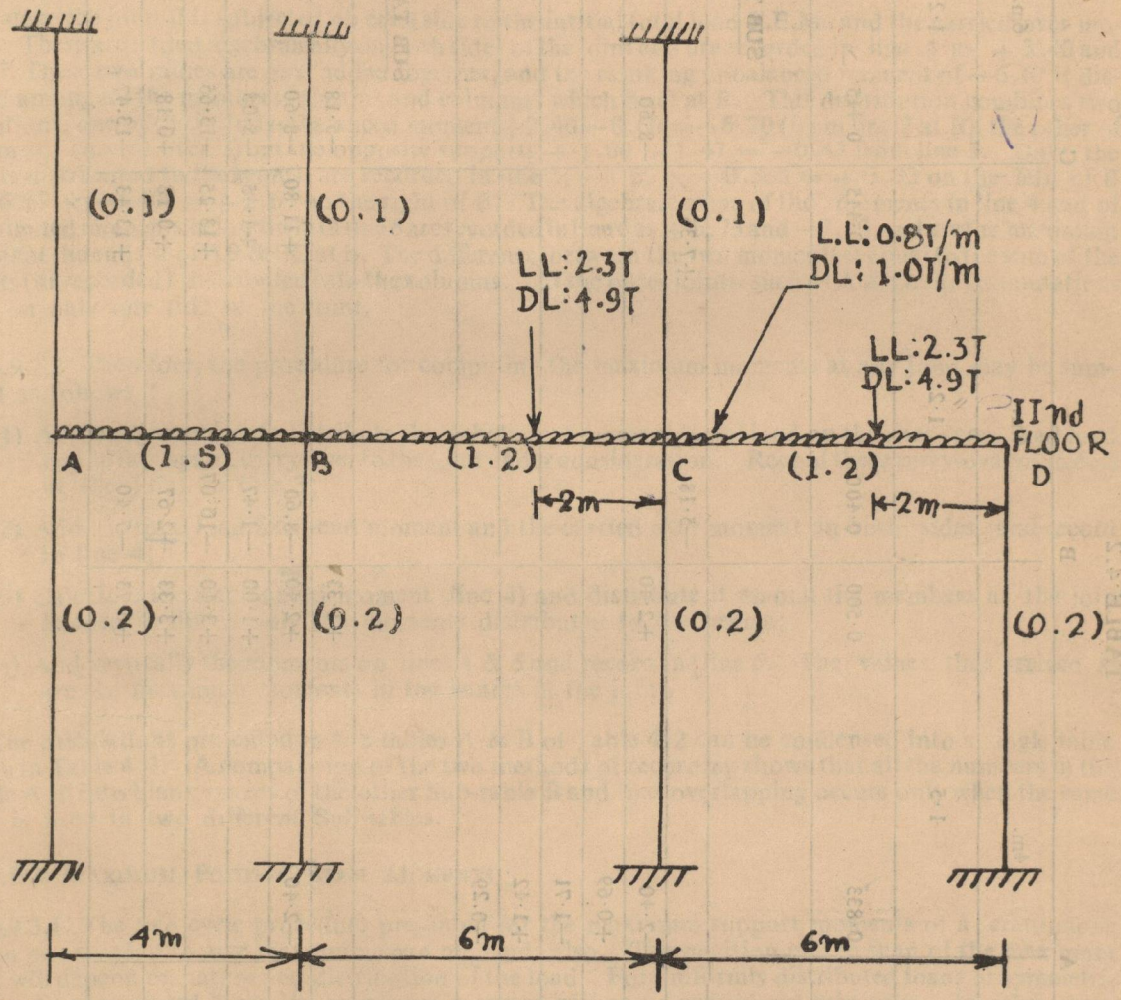


FIG 4.4

Line 1 in the table contains the dead load fixed-end moments and line 2 the total (dead and live) load fixed-end moments. Only those values are shown in the table which correspond to the loading on the two nearest spans on each side of the near joint and produce the maximum moment at the joint. For instance, the maximum moment at B occurs when the spans AB and BC are loaded with total load and the span CD is having only dead load. Therefore, the fixed end moments pertinent to the computation of the maximum moment at B are :

	A	B	C	D
LINE 1 D.L. F.E.M.				−5.18
LINE 2 T.L. F.E.M.		−2.40+2.40	−8.60+11.80	—

TABLE 4.2

	A		B		C	D
SPAN L	4m.		6m.		6m.	
K RELATIVE STIFFNESS	1·5		1·2		1·2	
DISTRIBUTION FACTOR	0·833	0·500	0·400	0·445	0·445	0·857
MAXIMUM-VE MOMENT AT A & D.					**SUB TABLE 'A'**	
LINE 1. D.L. F.E.M.			—5·18	+7·36		
,, 2. T.L. F.E.M.	—2·40	+2·40			—8·60	+11·80
,, 3. DISTRIBUTE + CARRY OVER	+0·69					+0·28
,, 4. ADD 2 & 3.	—1·71					+12·8
,, 5. DISTRIBUTE	+1·42					—10.35
,, 6. ADD 4 & 5.	—0.29					—1.73
MAXIMUM-VE MOMENT AT B & C.					**SUB TABLE 'B'**	
LINE 1. D.L. F.E.M.		+1·33			—5·18	
,, 2. T.L. F.E.M.	—2·40	+2·40	—8.60	+11·80	—8·60	—11.80
,, 3. DISTRIBUTE & CARRY OVER		+1·00	—1·47	+1·45	—5·05	
,, 4. ADD 2 + 3.		+3·40	—10·07	+13·25	—13·65	
,, 5. DISTRIBUTE		+3·33	+2·67	+0·18	+0·18	
,, 6. ADD 4 & 5.		+6·73	—7·40	+13·43	—13.47	

The joints opposite the near joint i.e. A and C are unlocked and the unbalanced moments at these joints are distributed and carried over to the near joint i.e. B. Only the carried-over moments are recorded in line 3 and the distributed moments at A & C are not shown. Thus, the moments carried over to the locked joint B are

$$\text{from A} : -\tfrac{1}{2} \times 0.833 \times 2.40 = +1.00$$

$$\text{from C} : -\tfrac{1}{2} \times 0.445 \times (+11.80 - 5.18) = -1.47$$

At this stage the joint B is subjected on each side to the intitial total load F.E.Ms and the carried-over moments. These are added algebraically on both sides of the joint and are recorded in line 4 as $+3.40$ and -10.07. These two values are next added together, and the resulting unbalanced moment of -6.67 is distributed among all the members (beams and columns) which meet at B. This distribution combines two distributions, one of the initial unbalanced moment $+2.40 - 8.60 = -6.20$ (from line 2 at B), the other of the moments carried over from the opposite supports $+ 1.00 - 1.47 = -0.47$ from line 3. Only the moments distributed to the beams are recorded in line 5, $-6.67 \times -0.500 = + 3.33$ on the left of B and $-6.67 \times -0.400 = + 2.67$ on the right of B. The algebraic sums of the moments in line 4 and of the distributed moments below them in line 5 are recorded in line 6 as $+6.73$ and -7.40, and are the maximum moments at the ends a of AB & BC at B. The difference between the two moments is equal to the sum of the moments (unrecorded) distributed into the columns. At the outer joints, such as A & D, the computations appear on only one side of the joint.

4.9.2.2. Therefore, the procedure for computing the maximum moments at any joint may be summarized as follows :

(1) At the adjacent joints distribute the unbalanced moment (total load on the near span, dead load on the far span), carry-over to the joint under consideration. Record these carry-over moments at the joint in line 3;

(2) Add the total load fixed-end moment and the carried over moment on both sides and record in line 4;

(3) Calculate the unbalanced moment (line 4) and distribute it among the members at the joint Record in line 5 only the moments distributed to the beams;

(4) Add vertically the moments on lines 4 & 5 and record in line 6. The values thus arrived at are the maximum moments in the beams at the joint.

The calculations presented in sub-tables A & B of Table 4.2 can be condensed into a single table as shown in Table 4.3. A comparision of the two methods of recording shows that all the numbers in the sub-table A fit into blank spaces of the other Sub-table B and that overlapping occurs only when the same number is used in two different Sub-tables.

4.9.3. Maximum Positive Span Moments

4.9.3.1. The two cycle procedure presented for the maximum support moments of a continuous beam can be extended to give maximum span moments also. The position on the span of the maximum moment will depend on nature and distribution of the load. For uniformly distributed loads or symmetrically placed concentrated loads the maximum moment will occur near about mid span. In other cases the position of maximum moment will have to be worked out. The procedure for determining the span moments is as follows :

(1) Place the live load on the span under consideration and alternate spans;

(2) compute the end moments and span moments in a fixed beam;

(3) compute changes (with proper signs) in the end moments caused by releasing the ends of the beam;

(4) compute the ordinates of the trapezoid which has for its vertical sides the moment changes computed in (3); and

(5) add algebraically the trapezoid ordinates of 4 to the ordinates of (2).

For example, the changes in the initial fixed-end moments in the span BC can be determined as given in Table 4.4.

4.9.3.2. Mid Span Moment

The steps for determining mid span moments may be summarised as follows :—

(i) Compute the intermediate point factor Q_1 & Q_r for mid point from the formulae given in table 4.5.

TABLE 4.3

	A	B		C		D
DISTRIBUTION FACTORS	·833	.500	·400	·445	·445	·857
1. DL. F.E.M.	—1·33	+1·33	—5·18	+7·36	—5·18	+7·36
2. T.L. F.E.M.	—2·40	+2·40	—8·60	+11·80	—8·60	+11·80
3. DISTRIBUTE AND CARRY OVER	+0·69	+1·00	—1·47	+1·45	—5·05	+0·28
4. ADD 2 & 3	—1·71	+3·40	—10·07	+13·25	—13·65	+12·08
5. DISTRIBUTE	+1·42	+3·33	+2·67	+0·18	—0·18	—10·35
6. ADD 4 & 5 MAXIMUM MOMENT	—0·29	+6·73	—7·40	+13·43	—13·47	+1·73

TABLE 4.4

	A	B		C		D
DISTRIBUTION FACTOR	0·833	0·500	0·400	0·445	0·445	0·857
1 D.L. F.E.M.	—1·33	+1·33			—5·18	+7·36
2 T.L. F.E.M.			—8·60	+11·80		
3 DISTRIBUTE	+1·10		+2·91	—2·95		—6·31
4 CARRY OVER		+0·55	—1·47(C_1)	(C_r) +1·45	—3·16	
5 DISTRIBUTE			+0·37	+0·76		
6 CHANGE IN T. L. F F MS.			+1·81	—0·74		

TABLE 4.5

POINT	$_rL$	$_rR$	Intermediate Point factors for Q_l	Intermediate Point factors for Q_r
Left ¼-point	¼	¾ —	$\frac{3D_l - 1}{4}$	$\frac{D_r + 5}{4}$
Right ¼-point	¾	¼ —	$\frac{D_l + 5}{4}$	$\frac{3D_r - 1}{4}$
Left 1/3-point	1/3	2/3 —	$\frac{2}{3}D_l$	$\frac{D_r}{3} + 1$
Right 1/3-point	2/3	1/3 —	$\frac{D_l}{3} + \frac{D_l}{3} + 1$	$\frac{2}{3}D_r$
Midpoint	½	½ —	$\frac{D_l + 1}{2}$	$\frac{D_r + 1}{2}$

D_l = Distribution factor at left of mid span.

D_r = Distribution factor at Right of mid span.

(ii) Record these factors as shown in table 4.6 Q_l being written above Q_r.

(iii) At mid span compute the fixed beam moments due to total load and record in line 2 with their signs based on sign convention given in Para 4.7.

(iv) Find the products of carry-over moments C_l/C_r as given in table 4.4 and the corresponding intermediate point factors Q_l/Q_r and record in line 3.

This is illustrated as follows :

For span BC,

$C_l = -1.47$ $\quad C_r = +1.45$ (refer Table 4.4)

Coefficients of $Q_l = -\frac{D_l+1}{2} = -\frac{0.400+1}{2} = -0.700$

,, ,, $Q_r = +\frac{D_r+1}{2} = +\frac{0.44+1}{2} = +0.723$

Product $= -1.47 \times -0.700 = +1.03$

,, $= +1.45 \times (+0.723) = +1.05$

The values + 1.03 and + 1.05 are written in line 3.

(v) Find the sum of fixed beam moment in line 2 and of the two corresponding moments in line 3 and record it in line 6 as the maximum moment at the mid point.

The computations of the maximum moments at mid points and at supports can be combined in the same table as shown in Table 4.6.

4.9.4. Minimum Positive & Maximum Negative Span Moments

In most cases the dead load present on all spans prevents the live load from creating negative moments in the span. The usual effect of placing the live load on the side spans of a beam is to reduce the positive moments in the mid span. Sometimes, even negative moments are produced in the span. In such a case reinforcement should be provided at the top to take care of the negative moments. This case normally arises when the adjacent spans are much longer than the span under consideration. In an office the corridor span is usually smaller than the adjacent spans.

Table 4.7 shows the computations of the minimum positive and maximum negative moment at mid-points of the beams shown in Fig 4.4. The method adopted is similar to the one followed in Table 4.6 except for the following :

(i) The fixed beam moments at mid points are computed for dead loads;

TABLE 4.6

Maximum Moments in Beams (Support and Mid Span Moments)

	A		B			C		D
LENGTH K DISTRIBUTION FACTORS	·833	4 m 1·5	·500	·400	6 m 1·2	·445	·445	6 m 1·2	·857
MID POINT Q_1 FACTOR FOR Qr		—0·916 +0·750			—0·700 +0·723			—0·722 +0·928	
1. D.L. F.E.M.	—1·33		+1·33	—5·18		+7·36	—5·18		+7·36
2. T.L. F.E.M.	—2·40	+1·20	+2·40	—8·60	+5·10	+11·80	—8·60	+5·10	+11·80
3. DISTRIBUTION & CARRY OVER (T.L.ON SPAN, D.L. ON ADJACENT SPAN)	+0·69	—0·63· +0·75	+1·00	—1·47	+1·03 +1·05	+1·45	—5·05	+3·65 +0.26	+0·28
4. ADD 2 & 3.	—1·71		+3·40	—10·07		+13·25	—13·65		+12·08
5. DISTRIBUTE	+1·42		+3·33	+2·67		+0·18	+0·18		—10·35
6. MAXIMUM MOMENTS*	—0·29	+1·32	+6·73	—7·40	+7·18	+13·43	—13·47	+9·01	+1·73

* FOR END MOMENTS ADD 4 & 5; FOR MID SPAN ADD 2 AND 3.

(ii) The first unbalanced moment distributed at the end of a span is taken as the algebric sum of the dead load moment in the span and the total load moment in the adjacent span and

(iii) The first summation (line 4) performed at the end of a span is of the dead load moment and the carried-over moment (i.e. line 1 + line 3 values).

4.9.5 Maximum Moments in Columns

The maximum moments in the columns at their junctions with the beams may be obtained by applying live load on the alternate spans of beams. Since two sets of alternate loading are possible on the beams causing a possible reversal of bending in every column, two sets of maximum moments are computed at the ends of all the columns framing into the beams. Usually the larger moment in a column is produced by taking live load on the longer adjacent span and other alternate spans. For a column framing into beams both top and bottom, the beams produced at the two ends of columns four moments, any one of which may be maximum. The column is designed for the axial load and maximum moment.

The terrace floor of the frame has also been analysed by two cycle method and the calculation tables are given in Annexure 4.3.

4.10 Dr. Gaspar Kani's Method of Iteration

4.10.1 Basic Principle

When load is applied to a structure, joints of the structure undergo rotations. Due to rigidity of the joints these rotations produce bending moments at the ends of members meeting at the joints. Moments at the ends of the members produced by the loading of the members are obtained by the superimposition of the following 3 steps :

(i) The ends of a loaded member AB are fixed and fixed end moments at the ends A & B are worked out.

(ii) The end A is released, and is allowed to undergo rotation of joint A while the end B is fixed.

(iii) The end B is released, and is allowed to undergo rotation of joint B while the end A is fixed.

Thus the end moment for the end A of the member AB is composed of

(i) $\bar{M}_{ab}$ —Fixed-end moment (produced by the given external loading),

(ii) $2M'_{ab}$ —Produced by the rotation of its own end, and

(iii) M'_{ba} —Produced by the rotation of the other end of the member.

$$M_{ab} = \bar{M}_{ab} + 2m'_{ab} + M'_{ba} \quad \ldots\ldots\ldots\ldots\ldots\ldots 4.1$$

The moment M'_{ab} which is produced by the rotation of end A is proportional to the rotation θA and corresponding K value of the member. This moment will be called rotation contribution of end A. (The complete expression for the rotation contribution of the end A is $M'_{ab} = 2.EK.\theta A$.). Similarly, the moment M'_{ba} is proportional to θB and the K value of the member and will be designated as the rotation contribution of the end B. If the rotation contributions are known the end moment M_{ab} can be determined by the summation of

(i) the fixed-end moment,

(ii) twice the rotation contribution of its own end A,

(iii) the rotation contribution of the other end of the same member.

When a joint rotates, the members connected at this joint undergo the same rotation. The rotation contribution depends on the angle of rotation and the stiffness factor K of the member. Thus, for the members meeting at any joint, the rotation being same, the rotation contributions will be proportional to the K-values of the members. Therefore, if the sum of the rotation contributions at a joint is known, the rotation contribution of ends of members meeting at the joint can be obtained by distributing their sum in proportion to their K-values.

Designating the end of the member connected to the joint considered as the near end and the other end of the member as the far end, it follows that for each joint there are as many far ends as near ends. A connected cantilever is considered as a member whose far end has moved to infinity.

For equilibrium at joint A, $\Sigma M_{ab} = 0$.

TABLE 4.7

Minimum + VE and Maximum —VE Moments at mid Span of Beams

		A			B			C		D
	DISTRIBUTION FACTOR	·833	—0·916 / +0·750	·500	·400	—·700 / +·722	·445	·445	—·722 / +·928	·857
1.	D.L. F.E.M.	—1·33	+0·67	+1·33	—5·18	+3·13	+7·36	—5·18	+3·13	+7·36
2.	T.L. F.E.M.	—2·40		+2·40	—8·60		+11·80	—8·60		+11·80
3.	DISTRIBUTE & CARRY OVER (D.L. ON SPAN AND T. L. ON ADJACENT SPANS)	+1·82	—1·67 / +0·41	+0·55	+0·28	—0·20 / +0·40	+0·56	—3·16	+2·28 / —1·36	—1·47
4.	ADD (1) & (3)	+0·49		+1·88	—4·90		+7·92	—8·34		+5·89
5.	DISTRIBUTE	—0·41		+1·51	+1·21		+0·19	+0·19		—5·05
6.	ADD (4) & (5)	+0·08	—0·59	+3·39	—3·69	+3·33	+8·11	—8·15	+4·05	+0·84

TABLE 4.8

TABLE 4.8

Maximum Moments in Columns

DISTRIBUTION FACTOR		TOP COL. BOT. COL.	0·055 0·112		0·033 0·067		0·37 0·074		.. 0·143
1.	TOTAL OR D. L.	F.F.M.	—2·40	+2·40	—5·18	+7·36	—8·60		+11·80
2.	DISTRIBUTE & CARRY OVER		+0·69	+1·00	+0·28	+0·56	—5·05		+0·28
3.	ADD 1 & 2		—1·71	+3·40	—4·90	+7·92	—13·65		+12·08
4.	DISTRIBUTE	TOP COL. BOT. COL.	+0·09 +0·19		+0·05 +0·10		+0·21 +0·42		— —1·73
1.	T.L. OR D.L.	F.E.M.	—1·33	+1·33	—8·60	+11·80	—5·18		+7·36
2.	DISTRIBUTE & CARRY OVER		+1·82	+0·55	—1·47	+1·45	—3·16		—1·47
3.	ADD (1) & (2)		+0·49	+1·88	—10·07	+13·25	—8·34		+5·89
4.	DISTRIBUTE	TOP COL. BOT. COL.	—0·03 —0·05		+0·27 +0·54		—0·18 —0·36		— —0·84

Replacing the end moment M_{ab} by the expression from eduation 4·1, we obtain

$$\Sigma \bar{M}_{ab} + 2\Sigma M'_{ab} + \Sigma M'_{ba} = 0$$

$$\Sigma M'_{ab} = -\tfrac{1}{2}(\Sigma \bar{M}_{ab} + \Sigma M'_{ba}) \quad .. \quad .. \quad .. \quad .. \quad 4.2$$

The sum of the rotation contributions at the near ends of members at joint A equals minus half the sum of the fixed-end moments at the joint and the rotation contributions at the far ends of the joint.

Thus, if the rotation contributions of the far ends of the members at a joint are known, the rotation contributions of the near ends of members can be immediately determined. In the beginning the rotation contributions of far ends may be assumed to be zero (where no better ones are available). This will be the first approximation and the successive operation will produce more accurate results.

The proceaure to be adopted in the analysis is to find out the sum of the fixed-end moments at the joint, and the rotation contributions of the far ends of the joints. The figure thus obtained is divided by (—)2 and distributed proportional to the K-values of the members. This can be further simplified by having the concept of ROTATION FACTORS which are obtained by distributing $(-\tfrac{1}{2})$ in the ratio of the K-values of the members meeting at the joint.

The rule for the basic operation in calculating the rotation contributions can be stated as follows:

"Sum the fixed-end moments of the joint and all the rotation contributions of the far ends of this joint. Multiply this sum by the respective rotation factors of this joint. The values obtained in this manner are the rotation contributions sought".

By repeated application of this single calculation and proceeding from joint to joint in an arbitrary sequence, all the rotation contributions may be obtained to any desired degree of accuracy.

4.10.2 Effect of Side Sway

For a member AB subjected to end rotations as well as lateral displacement of one end with respect to the other, the equation for the final end moment of the member at A can be expressed as

$$M_{ab} = \bar{M}_{ab} + 2M'_{ab} + M'_{ba} + M''_{ab} \quad .. \quad .. \quad .. \quad 4.3$$

The additional contribution M''_{ab} due to the linear displacement of the member is termed as the liner displacement contirbution.

Substituting the expression 4·3 in the equilibrium condition $\Sigma M_{ab} = 0$ for any joint A, one obtains

$$\Sigma \bar{M}_{ab} + 2\Sigma M'_{ab} + \Sigma M'_{ba} + \Sigma M''_{ab} = 0 \qquad \text{or}$$

$$\Sigma M_{ab} = -\tfrac{1}{2}\left\{\Sigma \bar{M}_{ab} + \Sigma (M'_{ba} + M''_{ab})\right\}$$

The rule for the basic operation to determine the rotation contributions will be the same as in the case of non-translatory joints. But in addition to fixed-end moments $\bar{M}_{ab}$ and the rotation contributions of the ends, the displacement contribution M''_{ab} of all bars of the corresponding joint are to be considered.

From the consideration of the equilibrium conditions, an equation for the determination of the linear displacement contributions can be worked out as shown below.

If a horizontal cut is made through all the columns of any storey, from consideration of equilibrium it follows that the sum of all the shear forces at the columns of this storey is equal to zero. This equilibrium condition, which must be satisfied at each storey and which is satisfied by horizontal relative displacements of the members, serves to determine the linear displacement contributions. If the storey 'r' contains COLUMNS OF EQUAL LENGTH, then this equilibrium condition, together with equation 4.3, by substituting the expression for the shear force of a column AB,

$$\theta_{ab} = -\frac{M_{ab} + M_{ba}}{h_{ab}} \qquad \text{gives}$$

$$\Sigma_{(r)} \theta_{ab} = \frac{-1}{h_{ab}} \Sigma (2M'_{ab} + M'_{ba} + M''_{ab} + 2M'_{ba} + M'_{ab} + M''_{ba}) = 0$$

and from this $\Sigma_{(r)} M''_{ab} = -\tfrac{2}{3} \Sigma_{(r)} (M'_{ab} + M'_{ba}) \quad .. \quad .. \quad .. \quad .. \quad 4.4$

The sum of the displacement contributions of all columns of a storey 'r' can, therefore, be determined from the rotation contributions of the column ends of this storey.

Since the magnitude of the linear displacement contributions depends only on the displacement (which is equal for all the columns of a particular storey) and on the corresponding $\frac{K}{h}$ and is, moreover, proportional to these values (The exact expression for the dispalcement contribution is

$M''_{ab} = 6EK\frac{\delta}{h}$ where δ stands for the relative transverse displacement of the member end and h for the member length) the linear displacement contributions of the columns of the story 'r' will be proportional to the corresponding values of $\frac{K}{h}$ during such a displacement i.e. they will be proportional to the stiffnesses since all the columns of the storey 'r' were assumed to be of equal length.

To make the calculations more convenient and in analogy to the rotation factors, we may now introduce the displacement factors, which are obtained by distributing the values $(-)\frac{3}{2}$, in proportion to the K-values of the members among all the columns of the storey.

The rule for the determination of the linear-displacement contributions can be summarised as follows:

> *"Sum the rotation contributions of all column ends of the storey in question. Multiply the sum so obtained by the linear displacement factors of the columns one after the other to obtain the linear displacement contributions".*

From the rotation contributions we calculate the displacement contributions, and from these again the rotation contribution of the following approximation, etc. until results of desired accuracy are reached.

4.10.3 PROCEDURE FOR ANALYSIS

4.10.3.1 The process of analysis can be summarised thus (For frames having non translatory joint)

(i) Find out the F.E.Ms due to external loads.

(ii) Find out K-values of all members.

(iii) Find out the rotation factors of each member at every joint (i.e.$= -\frac{1}{2} \times \frac{K_{ab}}{\Sigma K_{ab}}$) and write out these at the corresponding members as shown in Fig 4.5.

(iv) Find out the algebraic sum of the fixed end moments at each joint and write out the same at the centre of each joint.

(v) Start from the joint where you expect maximum change in moment.

(vi) Find out the sum of the fixed-end moments at the joint and rotation contribution of the far ends meeting at the joint. At the beginning the rotation contribution of far ends can be assumed as zero.

(vii) Multiply the sum obtained in (vi) by the rotation factors and write out against the corresponding member ends.

(viii) Proceed from joint to joint and repeat the operation at every joint till the required degree of accuracy is obtained.

(ix) The final moments at the ends of members are then calculated by the formula $M_{ab} =$ Fixed end moment $\bar{M}_{ab}$ + twice rotation contribution of near end A (M'_{ab}) + rotation contribution of the far end B(M'_{ba}).

(x) Check whether the sum of the final end moments at the joint is zero i.e. $\Sigma M_{ab}=0$.

4.10.3.2. The process of analysis of frames with translatory joints can be summarised thus :

Items (i) to (vii) same as in the case of non-translatory joints.

(viii) Find out the sum of the rotation contributions of all column ends of a storey and multiply with linear displacement factors to obtain the linear displacement contributions. This procedure is done for all storeys.

(ix) Find out the second approximations of rotation contributions. For this sum up the fixed end moment + first approximations of rotation contributions of the farther ends of members meeting at the joint + first approximation of linear displacement contributions of the columns meeting at the joint. Multiply this sum with corresponding rotation factors.

(x) Calculate the displacement contributions from the rotation contributions and from these again the rotation contribution of the following approximation etc. until results of desired accuracy are reached.

(xi) The final end moments of the members are then calculated by the formula

$M_{ab}=\bar{M}_{ab}+2M'_{ab}+M_{ba}+M''_{ab}$;

(xii) Check whether the sum of the final end moments at each joint is zero i.e. $\Sigma M_{ab}=0$.

(xiii) Check whether the sum of column shears in each storey is zero i.e.

$$\Sigma \frac{M_{ab}+M_{ba}}{h_{ab}} = 0$$

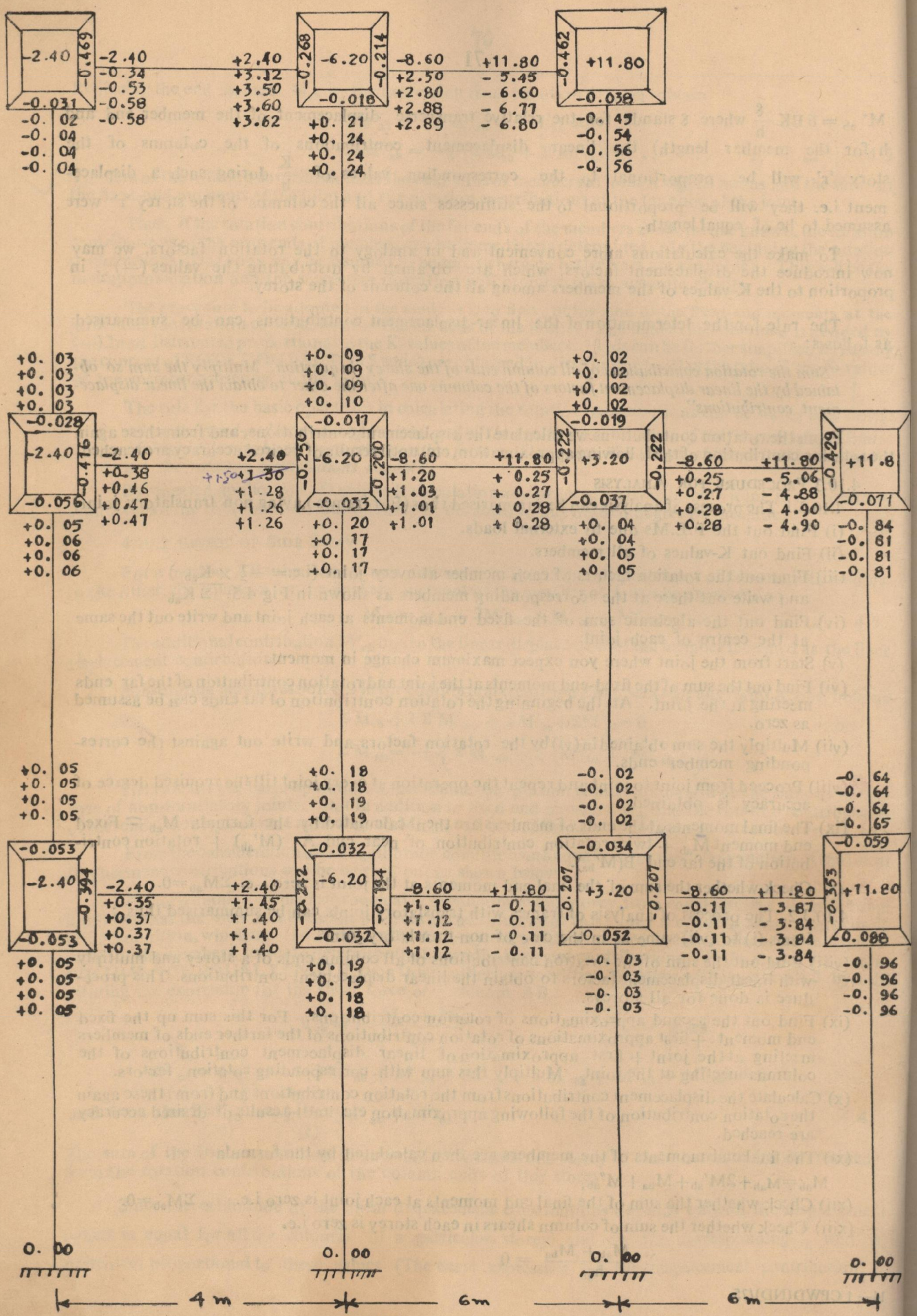


FIG 4.5

NOTE: FOR JOINT NUMBERS SEE FIG 4.3

4.10.4 The application of Kani's method for analysis of the frame shown in Fig 4.3 is explained in following steps, separatelv (i) neglecting the displacement of joints and, (ii) taking into account the displacement of joints.

Step 1 : Firstly the F.E.Ms for the given loading are computed using standard formulae and entered in the scheme and are written at the corresponding bar ends above the girder line.

For example, the F.E. Ms for the girder 2-3 with L=6 m and W=1.8T/m uniformly distributed load and Wc=7·20T concentrated load are as follows :

At the left end : $\bar{M}_{2-3} = -\left\{ \frac{WL^2}{12} + \frac{W_c ab^2}{L^2} \right\}$

$$= -\left\{ \frac{1.8\times 6^2}{12} + \frac{7.2\times 4\times 4}{36} \right\}$$

$$= -\ (5.4+3.2) = -8.60 \text{Tm.}$$

At the right end : $\bar{M}_{3-2} = \frac{WL^2}{12} + \frac{W_c a^2 b}{L^2}$

$$= +(5.4+ \frac{7.2\times 2\times 16}{36}$$

$$= +(5.4+6.4) = +11.80 \text{Tm}$$

Step 2 : Next, the restraint moments are written at the centre of the joint. The restraint moment i.e. the joint moments, which maintain the state of complete fixation, are always equal to the sum of the fixed end moments of all the members at the joint considered. For example for joint (2) M_2=restraint moment=+2.40 — 8.60= —6·20 Tm.

Step 3 : Now follows the calculation of rotation factors.

Example at joint (10) (Rotation Factor $= \frac{1}{2} \times \frac{K}{\Sigma K}$

For member 10—9 $= -\frac{1}{2} \times \frac{1.2}{2.9} = -0.207$

,, 10—6 $= -\frac{1}{2} \times \frac{0.2}{2.9} = -0.034$

,, 10—11 $= -\frac{1}{2} \times \frac{1.2}{2.9} = -0.207$

,, 10—C $= -\frac{1}{2} \times \frac{0.3}{2.9} = -0.052$

Check whether the sum of the rotation factors of the joint equals(—)0·5. Similarly rotation factor for members of other joints can be calculated. These rotation factors are entered in Fig 4.5 at each joint.

Step 4 : Next rotation contributions are calculated.

Rotation contribution=(Restraint moment of the joint plus all rotation contributions of the far ends meeting at the joint) multiplied by the respective rotation factors.

NOTE : at the beginning the rotation contributions of the far ends may be assumbed as zero.

The process of calculating rotation contributions is proceeded from joint to joint in any arbitrary sequence. The sequence in which one proceeds from joint to joint does not influence the result but only the speed of convergence. The most rapid convergence is obtained if one considers always that joint at which largest changes are expected. In our case, this should be joint (3). Therefore, in calculating the rotation contributions, the start is made at joint (3) whereby the rotation contributions of the distant bar ends i.e. member 2—3 at joint (2) and member 6—3 at joint (6) are assumed to be zero.

Joint (3)

rotation contribution 3—2 $= -0.462\times(+11.80+0+0)$

$= -5.45$

rotation contribution 3--6$= -0.038\times(+11.80+0+0)$

$= -0.45$

These approximate values of the rotation contribtions obtained are indicated at the corresponding bar ends (for girders, below the bar axis).

To clarify the general sequence of the calculation, the analysis of the example in fig 4.5 will be continued with the evaluation of the fourth approximation values of rotation contributions.

For joint (3)

rotation contribution $M'(4)3\text{-}2=(+11.80+2.88+0.02) \times -0.462=-6.80$

rotation contribution $M'(4)3\text{-}6=(+11.80+2.88+0.02) \times -3.038=-0.56.$

These values are immediately entered in the scheme. Since the rotation factors are negative, the rotation contributions and the sum from which they are obtanined always have opposite signs.

Now we proceed with the calculation of rotation contributions at joint (2) (fourth approximations).

For joint (2).

$M'_{(4)}2\text{—}1 = (-6.20-0.58-6.80+0.09)\times(-0.268) = +3\cdot62$

$M'_{(4)}2\text{—}3 = (-6.20-0.58-6.80+0.09)\times(-0.214) = +2\cdot89$

$M'_{(4)}2\text{—}5 = (-6.20-0.58-6.80+0.09)\times(-0.018) = +0\cdot24$

These values are entered immediately in the scheme. This process is continued till the desired accuracy is achieved.

The calculation of the rotation contributions proceeds in such a way that from approximate values for the rotation contributions, which originally represent very rough estimates, the next approximation is obtained with considerably better approximate values. If an error is made in the calculation, the rotation contributions so obtained may not be considered wrong, but merely as more or less close approximate values. But since we terminate the calculation when the approximation yields the same results as the preceding one, and moreover, we may assume that in the various approximations the same error is not committed repeatedly at the same place, the final result is free from errors.

Step 5 : After the final rotation contributions are determined, the final end moments may be obtained from the equation.

$$M_{ab}=\overline{M}_{ab}+2M'_{ab}+M'_{ba}$$

Example : $M_{1\text{-}2}=-2.40+2(-0.58)+3.62$

$=-2.40-1.16+3.62=+0.06$

and $M_{2\text{-}1}=+2.40+2(3.62)+(-0.58)$

$=+2.40+7.24-0.58=+9.06$

and so on.

In order not to crowd Fig 4.5 the calculation of the final end moments is indicated separately in Fig 4.6. First all the fixed-end moments and the final rotation contributions, indicated at the corresponding members have been carried over from Fig. 4.5. In accordance with equation 4.1 for obtaining the end moments in addition to the above two fgues we require at each bar end sum of the rotation contributions of the two ends of the same bar. This means that at both ends of a bar the same value, namely, the sum of the rotation contributions at both ends of the bar, must be added. In each case the sum of these three values (for column ends only two, because there are no fixed end moments) yields the end moment, which is entered at each bar end.

Step 6 : From considerations of equilibrium the sum of the end moments around a joint must be equal to zero. This check should be carried out at the end to see the correctness of the calculations.

EXAMPLE :

At joint 5 of Fig 4.6 the sum of the end moments around the joint is $=+5.39+0.42-6.32+0.52 =+0.01$. As this is negligible, it may be presumed that the calculations for the joint 5 are correct.

Should the necessity arise, at the end of the analysis, the change the bar dimensions or the loading at some places, then it is not necessary to repeat the whole analysis. After the corresponding numerical values have been changed in the old calculation scheme, the rotation contributions already calculated may

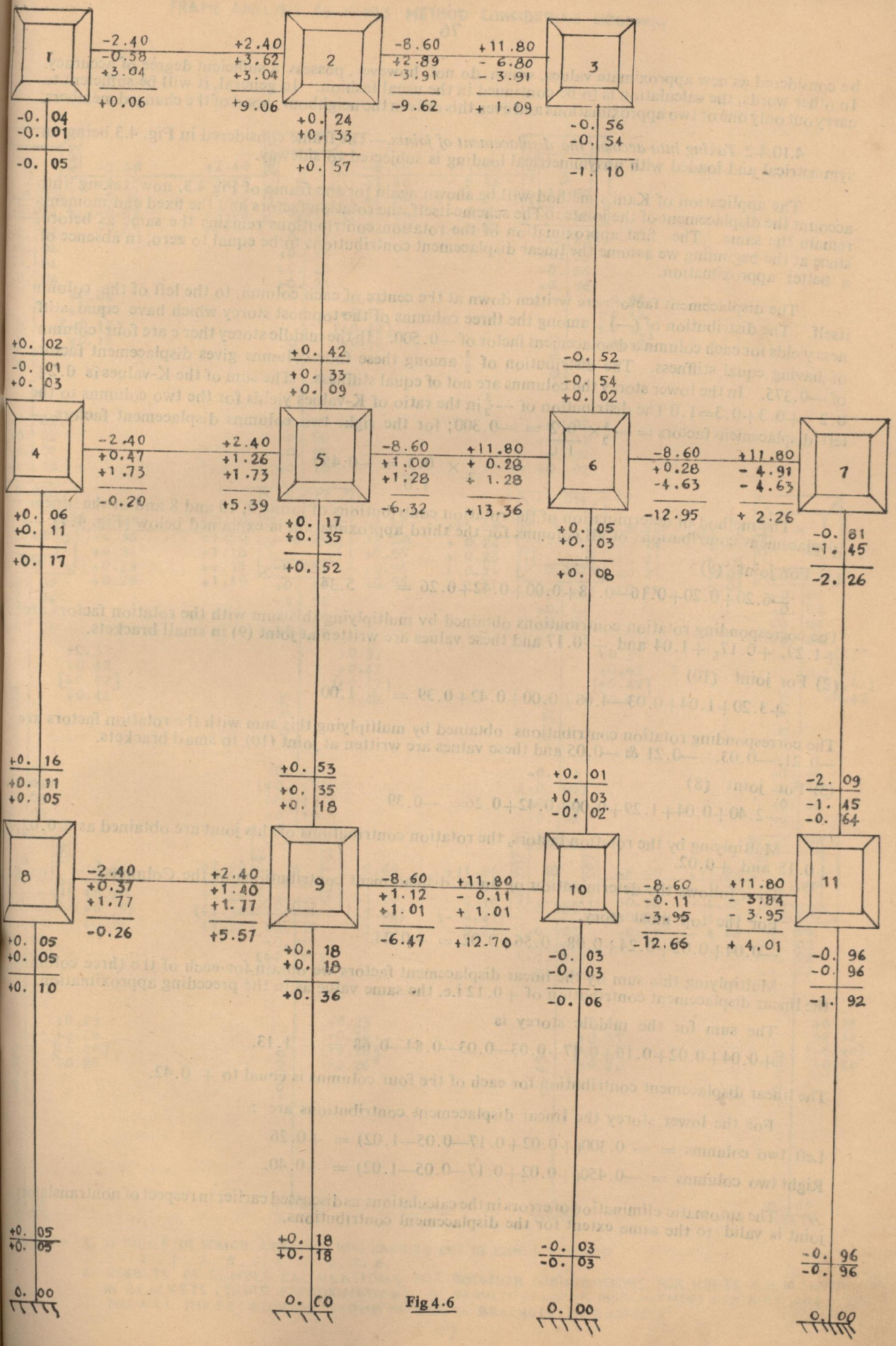

1
2
3
4
5
6
7
8
9
10
11
-2.40
-0.58
+3.04
+0.06
+2.40
+3.62
+3.04
+9.06
-8.60
+2.89
-3.91
-9.62
+11.80
- 6.80
- 3.91
+ 1.09
-0. 04
-0. 01
-0. 05
+0. 24
+0. 33
+0. 57
-0. 56
-0. 54
-1. 10
+0. 02
-0. 01
+0. 03
+0. 42
+0. 33
+0. 09
-0. 52
-0. 54
+0. 02
-2.40
+0.47
+1.73
-0.20
+2.40
+1.26
+1.73
+5.39
-8.60
+1.00
+1.28
-6.32
+11.80
+ 0.28
+ 1.28
+13.36
-8.60
+0.28
-4.63
-12.95
+11.80
- 4.91
- 4.63
+ 2.26
+0. 06
+0. 11
+0. 17
+0. 17
+0. 35
+0. 52
+0. 05
+0. 03
+0. 08
-0. 81
-1. 45
-2. 26
+0. 16
+0. 11
+0. 05
+0. 53
+0. 35
+0. 18
+0. 01
+0. 03
-0. 02
-2. 09
-1. 45
-0. 64
-2.40
+0.37
+1.77
-0.26
+2.40
+1.40
+1.77
+5.57
-8.60
+1.12
+1.01
-6.47
+11.80
- 0.11
+ 1.01
+12.70
-8.60
-0.11
-3.95
-12.66
+11.80
- 3.84
- 3.95
+ 4.01
+0. 05
+0. 05
+0. 10
+0. 18
+0. 18
+0. 36
-0. 03
-0. 03
-0. 06
-0. 96
-0. 96
-1. 92
+0. 05
+0. 05
0. 00
+0. 18
+0. 18
0. 00
-0. 03
-0. 03
0. 00
-0. 96
-0. 96
0. 00

Fig 4.6

be considered as new approximate values, which, do not, however, possess a sufficient degree of accuracy. In other words, the calculation is to be continued in the usual fashion. In general, it will be sufficient to carry out only one or two approximations and even this only in the neighbour hood of the changed members.

4.10.4.2 *Taking into account the displacement of joints.*—The frame considered in Fig. 4.3 being un symmetrical and loaded with unsymmetrical loading is subjected to sideway.

The application of Kani's method will be shown again for the frame of Fig 4.3, now, taking into account the displacement of the joints. The scheme itself, the rotation factors and the fixed end moments remain the same. The first approximation of the rotation contributions remains the same as before, since at the beginning we assume the linear displacement contributions to be equal to zero, in absence of a better approximation.

The displacement factors are written down at the centre of each column, to the left of the column itself. The distribution of $(-)\frac{3}{2}$ among the three columns of the topmost storey which have equal stiffness yields for each column a displacement factor of —0.500. In the middle storey there are four columns of having equal stiffness. The distribution of $\frac{3}{2}$ among these four columns gives displacement factors of —0.375. In the lower storey the columns are not of equal stiffness. The sum of the K-values is 0.2+0.2 +0.3+0.3=1.0 The distribution of $-\frac{3}{2}$ in the ratio of K-values yields for the two columns to the left displacement factors $= -\frac{3}{2} \times \frac{0.2}{1.0} = -0.300$; for the right two columns displacement factors —

$$= -\frac{3}{2} \times \frac{0.3}{1.0} = -0.450$$

The method of determination of the rotation contributions of joints 9, 10 and 8 and of the linear displacement contributions of all columns for the third approximation is explained below (Fig. 4.7):—

(1) For joint (9)

$$-6.20+0.20+0.16-0.18+0.00+0.42+0.26 = -5.34$$

The corresponding rotation contributions obtained by multiplying this sum with the rotation factors are +1.29, +0.17, +1.04 and + 0.17 and these values are written at joint (9) in small brackets.

(2) For joint (10)

$$+3.20+1.04+0.03-4.08+0.00+0.42+0.39 = +1.00$$

The corresponding rotation contributions obtained by multiplying this sum with the rotation factors are —0.21, —0.03. —0.21 & —0.05 and these values are written at joint (10) in small brackets.

(3) For joint (8)

$$-2.40+0.04+1.29+0.00+0.42+0.26 = -0.39$$

Multiplying by the rotation factors, the rotation contributions of this joint are obtained as +0.02, +0.15 and +0.02.

Then follows the determination of linear displacement contributions of the Columns.

For the top most story,

$$-0.04+0.02+0.24+0.08-0.56+0.02 = -0.24.$$

Multiplying this sum by the linear displacement factors we obtain for each of the three columns the linear displacement contribution of +0.12 i.e. the same value as in the preceding approximation.

The sum for the middle storey is

$$+0.04+0.02+0.16+0.17+0.03-0.03-0.84-0.68 = -1.13.$$

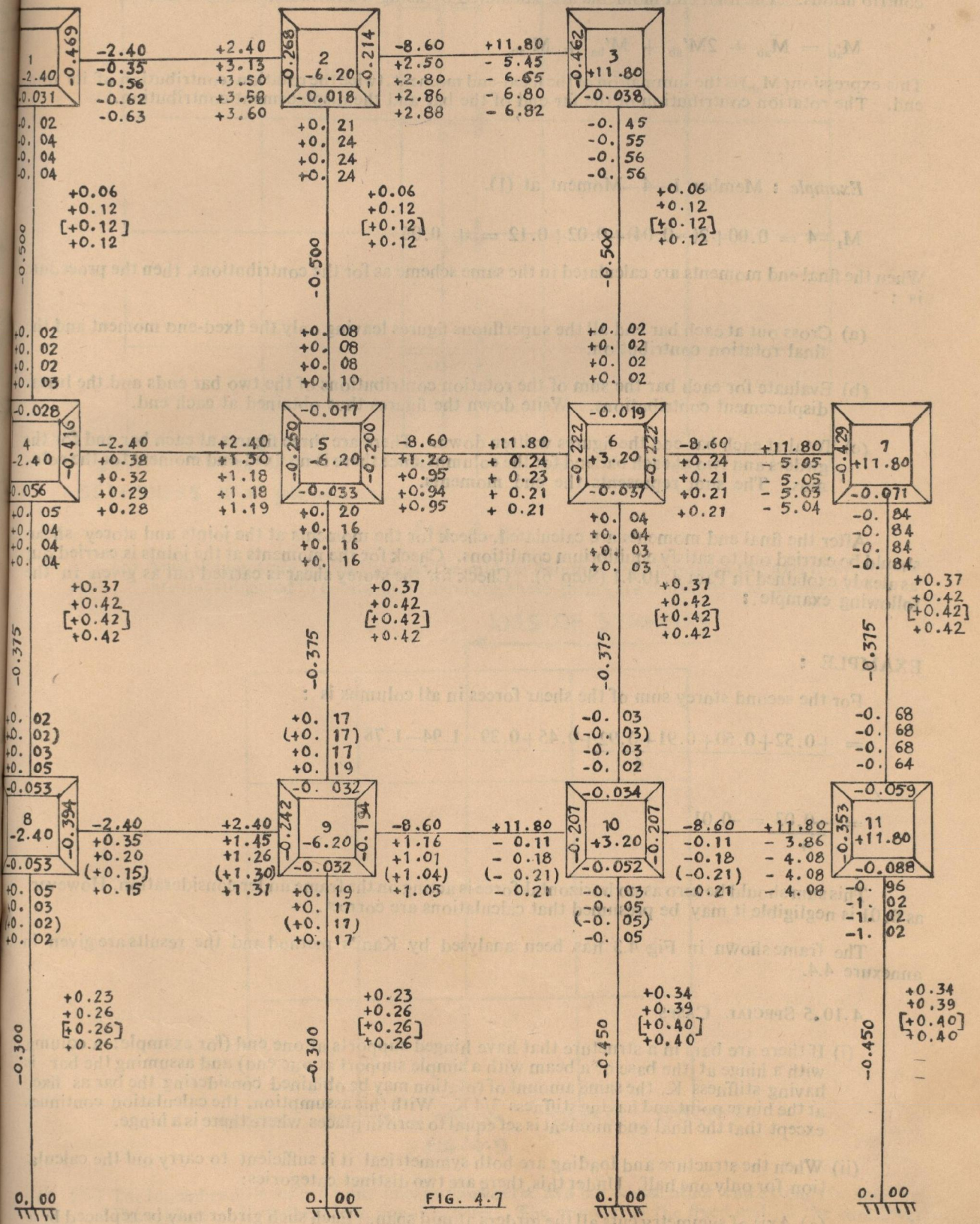
The linear displacement contribution for each of the four columns is equal to + 0.42.

For the lower storey the linear displacement contributions are :

Left two columns $= -0.300(+0.02+0.17-0.05-1.02) = +0.26$

Right two columns $= -0.450(+0.02+0.17-0.05-1.02) = +0.40.$

The automatic elimination of errors in the calculations as discussed earlier in respect of nontranslator joint is valid to the same extent for the displacement contributions.



NOTES: – 1. SEQUNCE IN WHICH ANALYSIS WAS CARRIED OUT IN KANI'S METHOD
3, 2, 1, 7, 5, 6, 4, 11, 9, 10, 8.
2. RESULTS OF SAMPLE CALCULATIONS FOR ROTATION CONTRIBUTIONS FOR JOINTS 8,9,10, SHOWN IN BRACKETS (THIRD APPROXIMATION). 3. RESULTS OF LINEAR DISPLACEMENT CONTRIBUTIONS FOR ALL THREE STOREYS SHOWN IN SQUARE BRACKETS (THIRD APPROX)

This process is repeated till the desired accuracy is achieved in the values of rotation and displacement contrib utions. The final end moments are calculated by using the formula.

$$M_{ab} = \bar{M}_{ab} + 2M'_{ab} + M'_{ba} + M''_{ab}$$

This expression(M_{ab}) is the summation of the fixed-end moment, twice the rotation contribution of its own end. The rotation contribution of the far end of the bar and the displacement contribution.

Example : Member 1—4—Moment at (1),

$$M_1-4 = 0.00+2(-0.04)+0.02+0.12 = +\ 0.06$$

When the final end moments are calculated in the same scheme as for the contributions, then the procedure is :

(a) Cross out at each bar end all the superfluous figures leaving only the fixed-end moment and the final rotation contribution.

(b) Evaluate for each bar the sum of the rotation contributions of the two bar ends and the linear displacement contributions. Write down the figures thus obtained at each end.

(c) Total at each bar end the figures written down. There are three figures at each bar end for the girders and two at each bar end for the columns since there is no fixed-end moment for the columns. The sum represents the end moments.

After the final end moments are calculated, check for the moments at the joints and storey shear should be carried out to satisfy equilibrium conditions. Check for the moments at the joints is carried out as aleady explained in Para 4.10.4.1 (Step 6). Check for the storey shear is carried out as given in the following example :

EXAMPLE :

For the second storey sum of the shear forces in all columns is :

$$= \frac{+0.52+0.50+0.91+0.92+0.45+0.39-1.94-1.78}{4}$$

$$= \frac{-0.03}{4} = -0.01$$

This sum should be zero as no horizontal force is acting on the frame under consideration. However as 0.01 is negligible it may be presumed that calculations are correct.

The frame shown in Fig 4.3 has been analysed by Kani's method and the results are given in annexure 4.4.

4.10.5 Special Cases

(i) If there are bars in a structure that have hinged supports at one end (for example, a column with a hinge at the base or a beam with a simple support at one end) and assuming the bar is having stiffness K, the same amount of rotation may be obtained considering the bar as fixed at the hinge point and having stiffness 3/4 K. With this assumption, the calculation continues except that the final end moment is set equal to zero in places where there is a hinge.

(ii) When the structure and loading are both symmetrical it is sufficient to carry out the calculation for only one half. Under this, there are two distinct categories:

(a) Axis of symmetry cuts all the girders at mid span. Each such girder may be replaced by a bar which is clamped at the symmetry axis and which has a stiffness $K' = \frac{1}{2}K$.

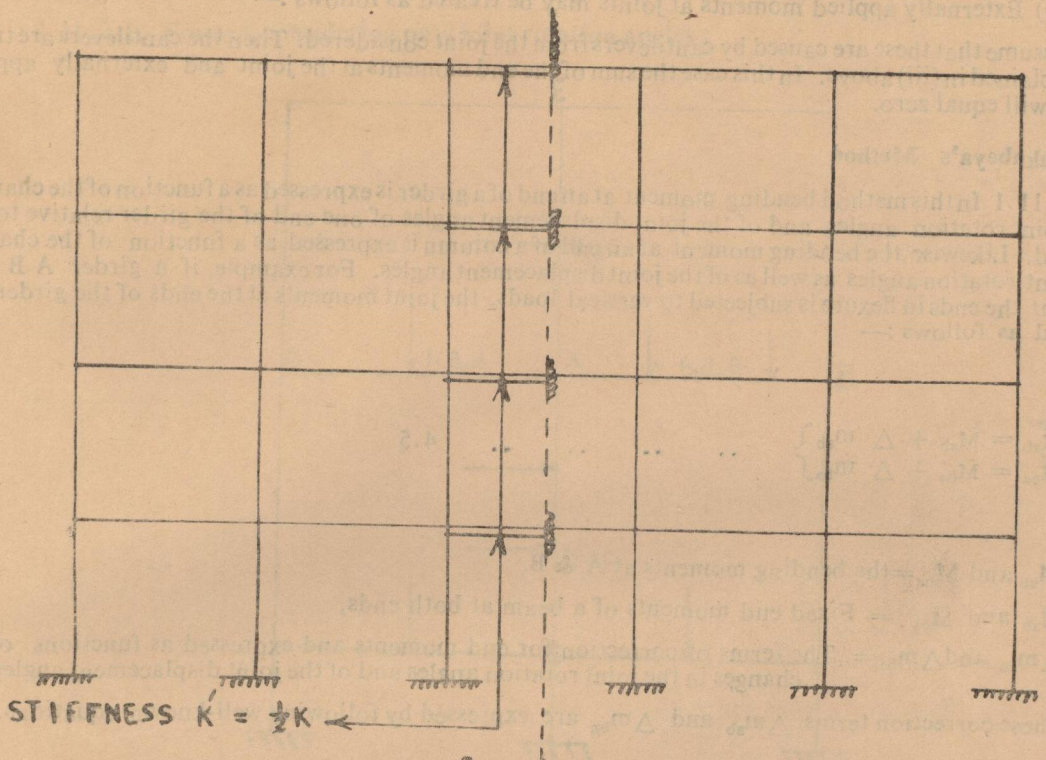


FIG. 4.8

(b) Axis of symmetry passes through the middle column. All joints on the axis of symmetry are not subjected to rotations. Therefore, these joints are considered as clampled.

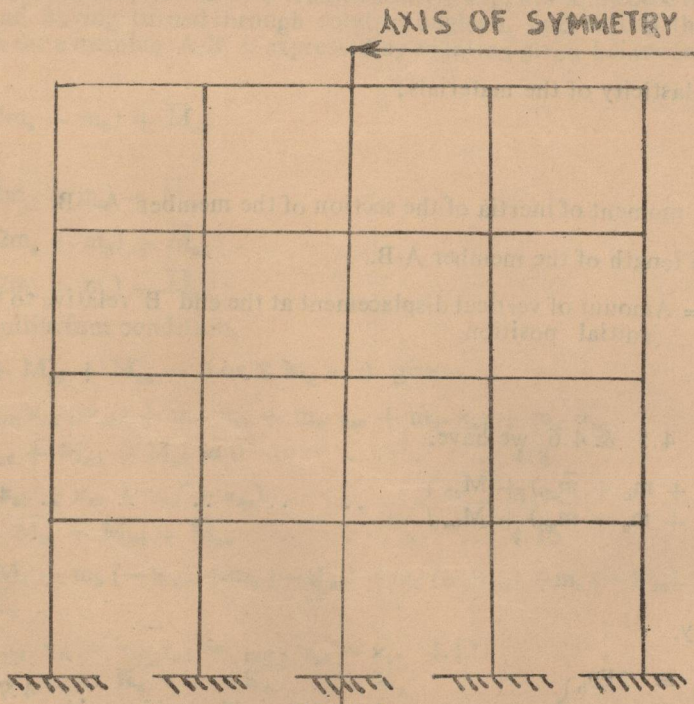


FIG. 4.9

(iii) The loaded cantilever in a joint may be considered as a bar whose other end extends to infinity. The stiffness K is, therefore, equal to zero. The moment at the fixed end of the cantilever should be determined and treated like the F.E.M. of any other bar.

(iv) Externally applied moments at joints may be treated as follows :—

Assume that these are caused by cantilevers from the joint considered. Then the cantilevers are treated as explained in (iii) above. In this case the sum of the end moments at the joint and externally applied moment will equal zero.

4.11 Takabeya's Method

4.11.1 In this method bending moment at an end of a girder is expressed as a function of the changes in the joint rotation angles and of the joint displacement angles of one end of the girder relative to the other end. Likewise the bending moment at an end of a column is expressed as a function of the changes in the joint rotation angles as well as of the joint displacement angles. For example, if a girder A-B restrained at the ends in flexure is subjected to vertical loads, the joint moments at the ends of the girder are expressed as follows :—

$$\left.\begin{aligned} M_{ab} &= \bar{M}_{ab} + \triangle m_{ab} \\ M_{ba} &= \bar{M}_{ba} + \triangle m_{ba} \end{aligned}\right\} \quad .. \quad .. \quad .. \quad .. \quad 4.5$$

where M_{ab} and M_{ba} = the bending moments at A & B,

$\bar{M}_{ab}$ and $\bar{M}_{ba}$ = Fixed end moments of a beam at both ends,

$\triangle m_{ab}$ and $\triangle m_{ba}$ = The terms of correction for end moments and expressed as functions of the changes in the joint rotation angles and of the joint displacement angles.

These correction terms $\triangle m_{ab}$ and $\triangle m_{ba}$ are expressed by following well-known equations.

$$\left.\begin{aligned} \triangle m_{ab} &= 2E\kappa_{ab}(2\theta_a + \theta_b - 3R_{ab}) \\ \triangle m_{ba} &= 2E\kappa_{ab}(2\theta_b + \theta_a - 3R_{ab}) \end{aligned}\right\} \quad .. \quad .. \quad .. \quad .. \quad .. \quad 4.6$$

where E = modulus of elasticity of the materials;

$\kappa_{ab} = \frac{I_{ab}}{l_{ab}}$, I_{ab} = moment of inertia of the section of the member A—B.

l_{ab} = length of the member A-B.

$R_{ab} = \frac{\delta_{ab}}{l_{ab}}$, δ_{ab} = Amount of vertical displacement at the end B relative to the end A from the initial position.

From equations 4.5 & 4.6 we have,

$$\left.\begin{aligned} M_{ab} &= \kappa_{ab}(2m_a + m_b + \bar{m}_{ab}) + \bar{M}_{ab} \\ M_{ba} &= \kappa_{ab}(2m_b + m_a + \bar{m}_{ab}) + \bar{M}_{ba} \end{aligned}\right\} \quad .. \quad .. \quad .. \quad .. \quad .. \quad 4.7$$

in which we denote by,

$$\left.\begin{aligned} m_a &= 2E\theta_a, \quad m_b = 2E\theta_b \\ \bar{m}_{ab} &= -6ER_{ab} \end{aligned}\right\} \quad .. \quad .. \quad .. \quad .. \quad .. \quad .. \quad 4.7a$$

m_a = rotation moment due to θ_a.

m_b = rotation moment due to θ_b.

and $\bar{m}_{ab}$ = acement moment due to R_{ab}.

4.11.2 Equation of Rotation Moment & Displacement Moment

4.11.2.1. Frames, considering only joint rotation angles :

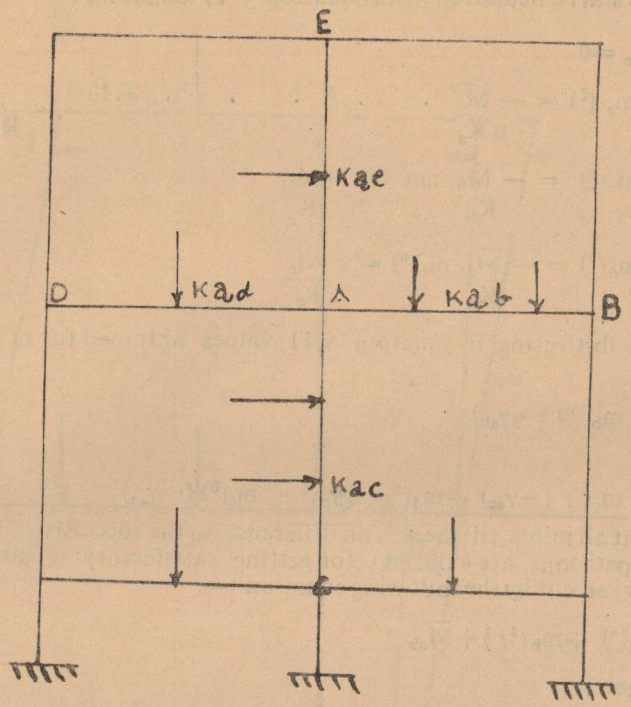


FIG. 4.10

Let us consider a member A—B of a frame shown in Fig. 4.8 subjected to vertical and horizontal loads; member A end having turned through rotation angles θ_a and end B through θ_b. The end moment M_{ab} at joint A for a member A-B, is expressed by equation given below (neglecting joint displacement angle).

$$M_{ab} = k_{ab}\,(2m_a + m_b) + \bar{M}_{ab}$$

in a similar way,

$$M_{ac} = \varkappa_{ac}\,(2m_a + m_c) + \bar{M}_{ac}$$

$$M_{ad} = \varkappa_{ad}\,(2m_a + m_d) + \bar{M}_{ad}$$

$$M_{ae} = \varkappa_{ae}\,(2m_a + m_e) + \bar{M}_{ae}$$

Under the equilibrium condition,

$M_{ab} + M_{ac} + M_{ad} + M_{ae} = \theta$ or $\Sigma M_a = 0$ gives

$$m_a \cdot 2(\varkappa_{ab} + \varkappa_{ac}\,\varkappa_{ad} + \varkappa_{ae}) + m_b \cdot \varkappa_{ab} + m_c \cdot k_{ac} + m_d \cdot \varkappa_{ad} + m_e \cdot \varkappa_{ae}$$
$$+ \bar{M}_{ab} + \bar{M}_{ac} + M_{ad} + M_{ae} = 0 \quad . \quad . \quad . \quad . \quad 4.8$$

$$\text{Let } K_a = 2\,(\varkappa_{ab} + \varkappa_{ac} + \varkappa_{ad} + \varkappa_{ae}) \quad . \quad . \quad . \quad . \quad 4.9$$

$$\bar{M}_a = \bar{M}_{ab} + \bar{M}_{ac} + \bar{M}_{ad} + \bar{M}_{ae} \quad . \quad . \quad . \quad . \quad 4.10$$

$$\therefore m_a = -\frac{\bar{M}_a}{K_a} + m_b\,(-\gamma_{ab}) + m_c\,(-\gamma_{ac}) + m_d\,(-\gamma_{ad}) + m_e\,(-\gamma_{ae}) \quad 4.11$$

$$\text{where } \gamma_{ab} = \frac{\varkappa_{ab}}{K_a},\ \gamma_{ac} = \frac{\varkappa_{ac}}{K_a},\ \gamma_{ad} = \frac{k_{ad}}{K_a},\ \gamma_{ae} = \frac{\varkappa_{ae}}{K_a} \quad 4.12$$

$$\text{and } \Sigma\gamma_a = \gamma_{ab} + \gamma_{ac} + \gamma_{ad} + \gamma_{ae}$$
$$= \frac{1}{K_a}\,(\varkappa_{ab} + \varkappa_{ac} + \varkappa_{ad} + \varkappa_{ae})$$
$$= \tfrac{1}{2}$$

The equation 4.11 is defined by the author as "*moment equation of joint rotation*" and is important to determine rotation moment at the joint.

As the first step, we start calculation from equation 4.11 assuming :

$$m_b = m_c = m_d = m_e = 0$$

and we get $m_a = m_a\,(^0) = -\frac{\bar{M}_a}{K_a}$ 4.13

In a similar way, $m_b(^0) = -\frac{\bar{M}_b}{K_b}$, $m_c(^0) = -\frac{\bar{M}_c}{K_c}$,

$$m_d(^0) = -\frac{\bar{M}_d}{K_d},\quad m_e(^0) = -\frac{M_e}{k_e}$$

In the second step substituting in equation 4.11 values obtained for m $(^0)$ in the place of m we get—

$$m_a(^1) = -\frac{M_a}{K_a} + m_b\,(^0)\,(-\gamma_{ab})$$

$$+ m_c(^0)\,(-\gamma_{ac}) + m_d(^0)\,(-\gamma_{ad}) + m_e(^0)\,(-\gamma_{ae})$$

This process is repeated at all joints till there is no difference in the successive rotation moments. Generally three or four repetitions are sufficient for getting satisfactory results. For example design moment M_{ab} is to be worked out by the following equation :—

$$M_{ab}(^4) = k_{ab}\,(2m_a(^4) + m_b(^4)) + \bar{M}_{ab}$$

Likewise we get :

$$M_{ac}(^4) = k_{ac}\,(2m_a(^4) + m_c(^4)) + \bar{M}_{ac}$$
$$M_{ad}(^4) = k_{ad}\,(2m_a(^4) + m_d(^4)) + \bar{M}_{ad}$$
$$M_{ae}(^4) = k_{ae}\,(2m_a(^4) + m_e(^4)) + \bar{M}_{ae}$$

Next, let us assume that four times repetition does not satisfy the equilibrium condition :—

$$\Sigma M_a(^4) = 0$$

Let the discrepancy be $\pm\Delta m$, so that

$$\Sigma M_a(^4) = \pm\,\Delta\,m$$

We distribute this unbalanced moment in proportion to the distribution factors of members meeting at joint A and the total amount of design moments becomes as follows :—

$$M_{ab} = M_{ab}(^4) \pm \Delta\,m \left(\frac{k_{ab}}{k_{ab} + k_{ac} + k_{ad} + k_{ae}}\right)$$

$$M_{ac} = M_{ac}(^4) \pm \Delta\,m \left(\frac{k_{ac}}{k_{ab} + k_{ac} + k_{ad} + k_{ae}}\right)$$

$$M_{ad} = M_{ad}(^4) \pm \Delta\,m \left(\frac{k_{ad}}{k_{ab} + k_{ac} + k_{ad} + k_{ae}}\right)$$

$$M_{ae} = M_{ae}(^4) \pm \Delta\,m \left(\frac{k_{ae}}{k_{ab} + k_{ac} + k_{ad} + k_{ae}}\right)$$

4.11.2.2 *Frames, considering Joint Rotations and Joint Displacements.*—All joints of a multi-storied rigid frame subjected to horizontal joint loads and vertical loads, as shown in Fig. 4.11 displace horizontally. We get the following displacement moments for columns from equation 4.7a :—

For the first storey : $\bar{m}_{11} = \bar{m}_{11\,2} = \bar{m}_{111}{}^3 = +\,6EKR_2$ . . . 4.14

For the second storey : $\bar{m}_{16} = \bar{m}_{25} = \bar{m}^3{}_4 = +6EKR_2$ . . . 4.15

For the third storey : $m_{67} = m_{58} = +6EKR_1$ 4.16

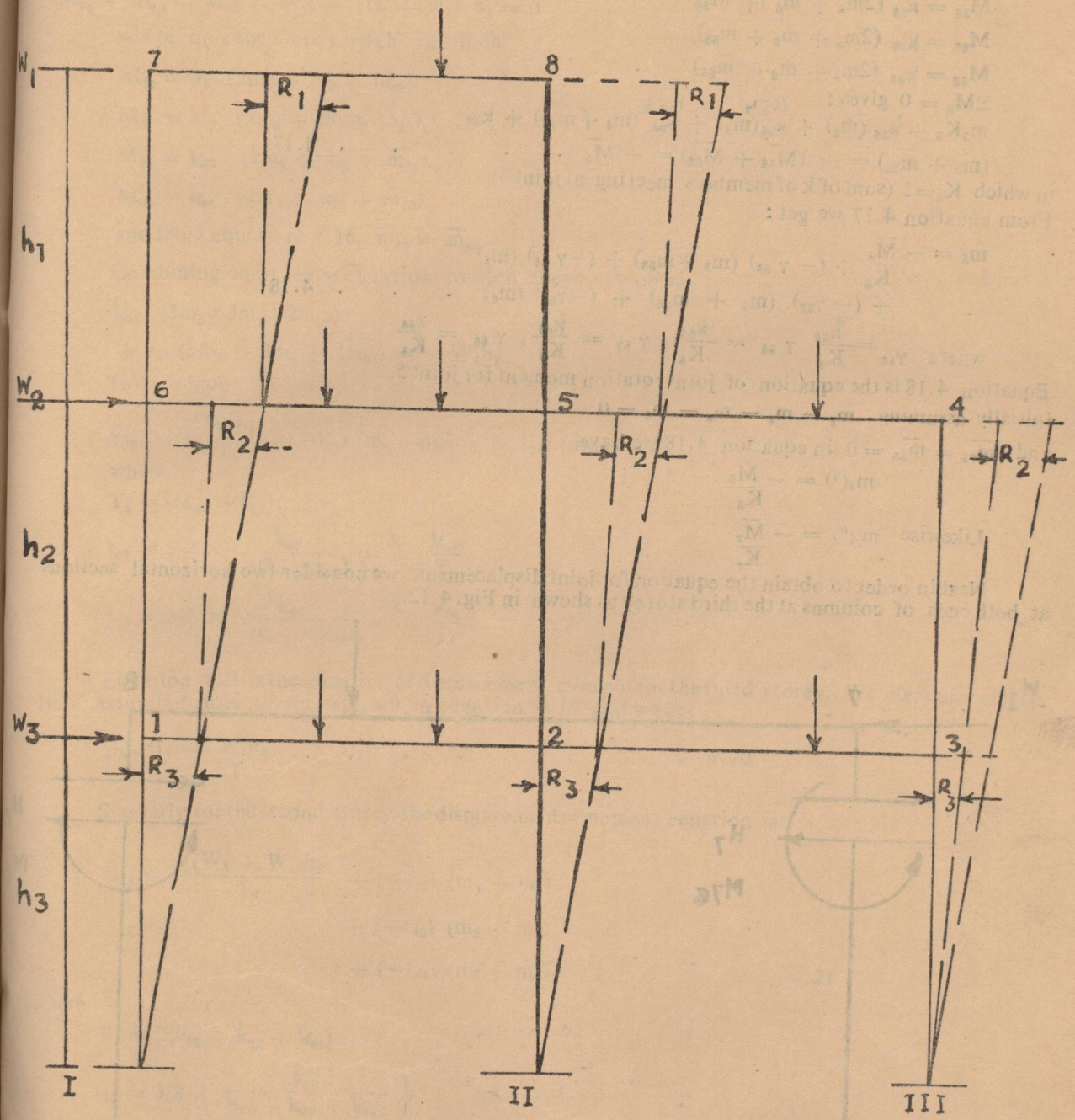


FIG. 4.11

IN ORDER TO OBTAIN THE EQUATION OF ROTATION MOMENT WE START FROM EQUATION 4.7

At joint (5) :

$M_{54} = k_{54}(2m_5 + m_4) + \bar{M}_{54}$

$M_{56} = k_{56}(2m_5 + m_6) + \bar{M}_{56}$

$M_{58} = k_{58}(2m_5 + m_8 + \bar{m}_{58})$,

$M_{52} = k_{52}(2m_5 + m_2 + \bar{m}_{25})$

$\Sigma M_5 = 0$ gives :

$$m_5 K_5 + k_{56}(m_6) + k_{54}(m_4) + k_{58}(m_8 + \bar{m}_{58}) + k_{52}(m_2 + \bar{m}_{25}) = -(\bar{M}_{54} + \bar{M}_{56}) = -\bar{M}_5 \quad . \quad . \quad . \quad . \quad . \quad 4.17$$

in which $K_5 = 2$ (sum of k of members meeting at joint 5)

From equation 4.17 we get :

$$m_5 = -\frac{\bar{M}_5}{K_5} + (-\gamma_{58})(m_8 + \bar{m}_{58}) + (-\gamma_{54})(m_4) + (-\gamma_{52})(m_2 + \bar{m}_{25}) + (-\gamma_{56})(m_6) \quad . \quad . \quad . \quad 4.18$$

where $\gamma_{58} = \frac{k_{58}}{K_5}$, $\gamma_{54} = \frac{k_{54}}{K_5}$, $\gamma_{52} = \frac{k_{52}}{K_5}$, $\gamma_{56} = \frac{\chi_{56}}{K_5}$

Equation 4.18 is the equation of joint rotation moment for joint 5.

Initially assuming $m_2 = m_4 = m_6 = m_8 = 0$

and $\bar{m}_{25} = \bar{m}_{58} = 0$ in equation 4.18 we have

$$m_5(^0) = -\frac{\bar{M}_5}{K_5}$$

Likewise $m_r(^0) = -\frac{\bar{M}_r}{K_r}$

Next in order to obtain the equation for joint displacement, we consider two horizontal sections at both ends of columns at the third storey as shown in Fig. 4.12.

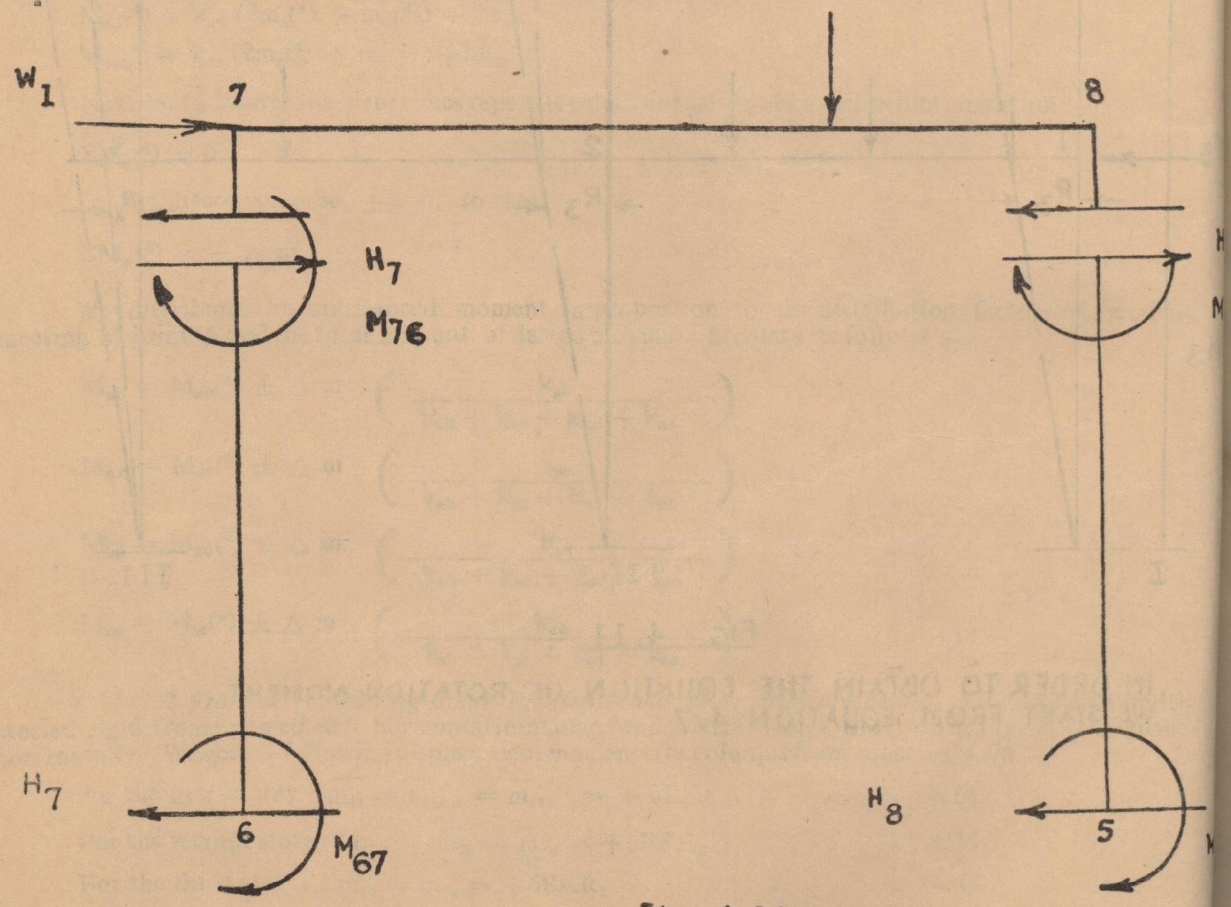


FIG 4.12

The equilibrium condition of the third storey as a whole gives :

$W_1 = H_7 + H_8$.

$M_{76} + M_{67} + M_{85} + M_{58} + (H_7 + H_8)\, h_1 = 0$

where h_1 is the storey height in which

$M_{76} = k_{67}\,(2m_7 + m_6 + \bar{m}_{67})$,

$M_{67} = k_{67}\,(2m_6 + m_7 + \bar{m}_{67})$,

$M_{85} = k_{58}\,(2m_8 + m_5 + \bar{m}_{58})$,

$M_{58} = k_{58}\,(2m_5 + m_8 + \bar{m}_{58})$,

and from equation 4.16, $\bar{m}_{67} = \bar{m}_{58}$.

Combining these seven equations written above, we obtain :

$k_{67}\,(3m_6 + 3m_7 + 2\bar{m}_{67})$

$+ k_{58}\,(3m_5 + 3m_8 + 2\bar{m}_{67}) = -W_1 h_1$

from which

$$\bar{m}_{67} = -\frac{W_1 h_1}{T_1} + (-t_{67})\,(m_6 + m_7) + (-t_{58})\,(m_5 + m_8) \quad . \quad . \quad . \quad 4.19$$

where

$T_1 = 2(k_{67} + k_{58})$

$$t_{67} = 3/2 \times \frac{k_{67}}{(k_{67} + k_{58})} = 3.\ \frac{k_{67}}{T_1}$$

$$t_{58} = 3/2 \times \frac{k_{58}}{(k_{67} + k_{58})} = 3.\ \frac{k_{58}}{T_1}$$

Equation 4.19 is the equation of displacement moment for the third storey. We start the calculation, assuming $m_5 = m_6 = m_7 = m_8 = 0$ in equation 4.19 and we get

$$\bar{m}_{67}^{(0)} = -\frac{W_1 h_1}{T_1} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad 4.20$$

Similarly for the second storey the displacement = moment equation is

$$\bar{m}_{16} = -\frac{(W_1 + W_2) h_2}{T_2} + (-t_{16})\,(m_1 + m_6) + (-t_{25})\,(m_2 + m_5) + (-t_{34})\,(m_3 + m_4) \quad . \quad . \quad . \quad . \quad 4.21$$

where

$T_2 = 2(k_{16} + k_{25} + k_{34})$

$$t_{16} = 3/2 \left\{ \frac{k_{16}}{k_{16} + k_{25} + k_{34}} \right\}$$

$$t_{25} = 3/2 \left\{ \frac{k_{25}}{k_{16} + k_{25} + k_{34}} \right\}$$

$$t_{34} = 3/2 \left\{ \frac{k_{34}}{k_{16} + k_{25} + k_{34}} \right\}$$

From Equation 4.21 we obtain assuming $m_1 = m_2 = m_3 = m_4 = m_5 = m_6 = 0$

$$\bar{m}_{16}^{(0)} = -\frac{(W_1 + W_2)\, h_2}{T_2}$$

Likewise we can determine all the displacement moments $\bar{m}^{(0)}$ for every storey.

Next we can obtain $m_5^{(1)}$ from equation 4.18

$$m_5^{(1)} = -\frac{\bar{M}_5}{K_5} + (-\gamma_{58})\ (m_8^{(0)} + \bar{m}_{58}^{(0)}) + (-\gamma_{54})\ (m_4^{(0)})$$

$$+ (-\gamma_{52})\ (m_2^{(0)} + \bar{m}_{25}^{(0)}) + (-\gamma_{56})\ (m_6^{(0)})$$

In a similar way from equation 4.19

$$\bar{m}_{67}^{(1)} = -\frac{W_1 h_1}{T_1} + (-t_{67})\ (m_6^{(1)} + m_7^{(1)}) + (-t_{58})\ (m_5^{(1)} + (m_8^{(1)})$$

and so on.

When the frame is subjected to vertical loads only, there are no horizontal loads acting at joint levels, equations 4.19 & 4.21 reduce to

$$\bar{m}_{67} = (-t_{67})\ (m_6 + m_7) + (-t_{58})\ (m_5 + m_8)$$
$$\bar{m}_{16} = (-t_{16})\ (m_1 + m_6) + (-t_{25})\ (m_2 + m_5) + (-t_{34})\ (m_3 + m_4)$$

respectively.

The process of calculating rotation moments and displacement moments is repeated till two successive values do not differ.

4.11.3 To illustrate the procedure, the frame shown in Fig. 4.3 has once again been considered and analysed by this method.

4.11.3.1 Example neglecting side sway

(Refer Fig. 4.13)

Step I : Calculation of $\bar{M}$

$\bar{M}_1 = -2.40$ tm.

$\bar{M}_2 = -6.20$ tm. $= \bar{M}_5 = \bar{M}_9$ (i.e. $-8.60 + 2.40$)

$\bar{M}_6 = +11.80$ tm. $= \bar{M}_7 = \bar{M}_{11}$

$\bar{M}_4 = -2.40$ tm $= \bar{M}_1 = \bar{M}_8$

$\bar{M}_6 = +11.80 - 8.60 = +3.20$ tm $= \bar{M}_{10}$

Step II : Calculation of K ($K = 2\Sigma k$)

$K_1 = 2(1.5 + 0.1) = 3.20$

$K_2 = 2(1.5 + 1.2 + 0.1) = 5.6$

$K_3 = 2(1.2 + 0.1) = 2.6$

$K_4 = 2(1.5 + 0.1 + 0.2) = 3.6$

$K_5 = 2(1.5 + 0.1 + 1.2 + 0.2) = 6.0$

$K_6 = 2(1.2 + 1.2 + 0.1 + 0.2) = 5.4$

$K_7 = 2(1.2 + 0.2) = 2.8$

$K_8 = 2(1.5 + 0.2 + 0.2) = 3.8$

$K_9 = 2(1.5 + 1.20 + 0.2 + 0.2) = 6.2$

$K_{10} = 2(1.2 + 1.2 + 0.2 + 0.3) = 5.8$

$K_{11} = 2(1.2 + 0.2 + 0.3) = 3.4$

FRAME ANALYSIS WITHOUT CONSIDERING SIDESWAY

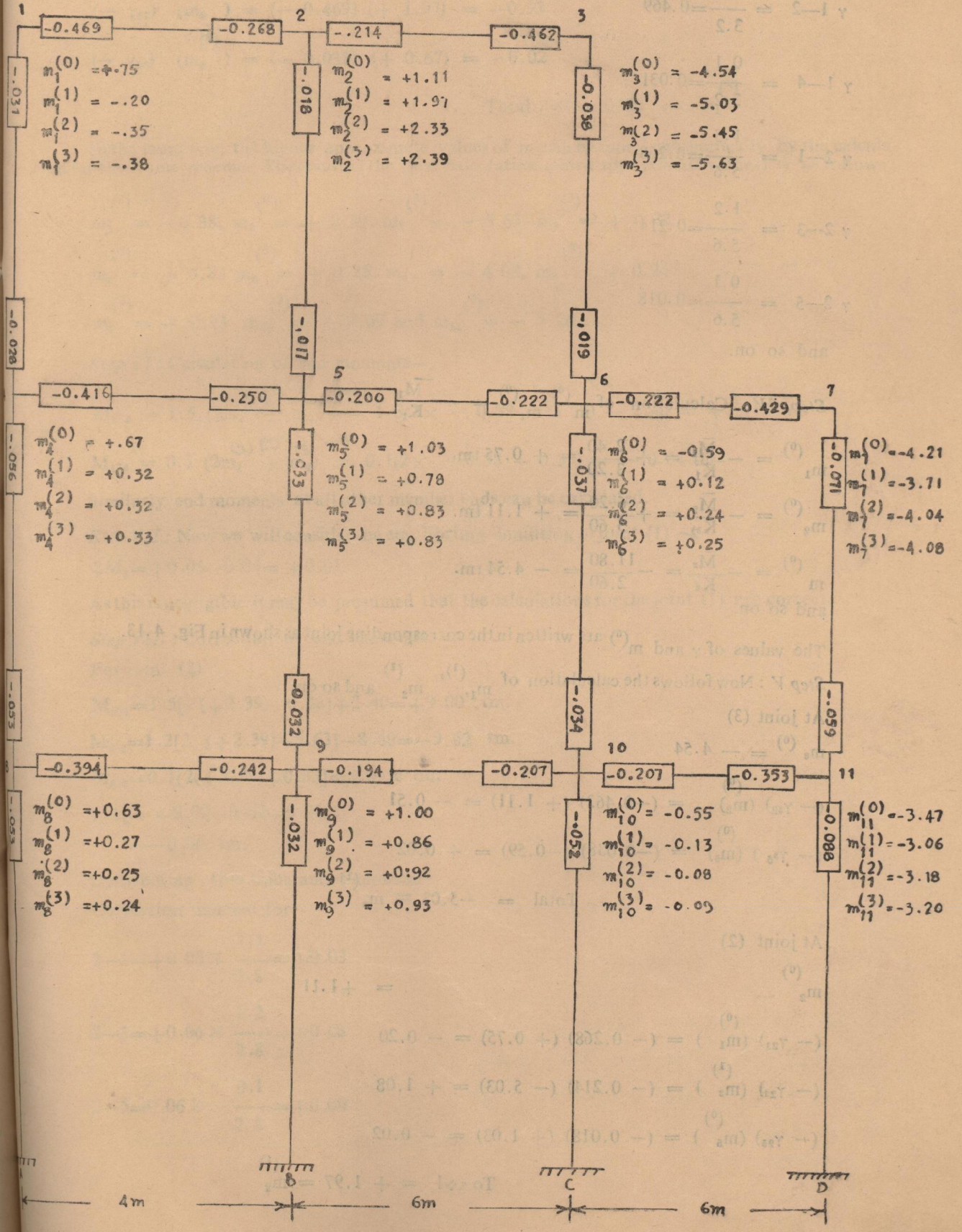


FIG. 4.13

Step III : Calculation γ ($\gamma ab = \frac{k_{ab}}{K_a}$)

$$\gamma\ 1—2 = \frac{1.5}{3.2} = 0.469$$

$$\gamma\ 1—4 = \frac{0.1}{3.2} = 0.031$$

$$\gamma\ 2—1 = \frac{1.5}{5.6} = 0.268$$

$$\gamma\ 2—3 = \frac{1.2}{5.6} = 0.214$$

$$\gamma\ 2—5 = \frac{0.1}{5.6} = 0.018$$

and so on.

Step IV: Calculation of $m^{(0)} \cdot m_1^{(0)} = -\frac{\bar{M}_1}{K_1}$

$$m_1^{(0)} = -\frac{\bar{M}_1}{K_1} = +\frac{2.40}{3.20} = +\ 0.75 \text{ tm.}$$

$$m_2^{(0)} = -\frac{\bar{M}_2}{K_2} = +\frac{6.20}{5.60} = +\ 1.11 \text{ tm.}$$

$$m^{(0)} = -\frac{\bar{M}_3}{K_3} = -\frac{11.80}{2.60} = -\ 4.54 \text{ tm.}$$

and so on.

The values of γ and $m^{(0)}$ are written in the corresponding joint as shown in Fig. 4.13.

Step V : Now follows the calculation of $m_1^{(1)}$, $m_2^{(1)}$ and so on.

At joint (3)

$m_3^{(0)} = -\ 4.54$

$(-\gamma_{32})\ (m_2^{(0)}) = (-0.462)\ (+\ 1.11) = -\ 0.51$

$(-\gamma_{36})\ (m_6^{(0)}) = (-0.038)\ (-0.59) = +\ 0.02$

Total $= -5.03 = m_3^{(1)}$

At joint (2)

$m_2^{(0)} = +1.11$

$(-\gamma_{21})\ (m_1^{(0)}) = (-\ 0.268)\ (+\ 0.75) = -\ 0.20$

$(-\gamma_{23})\ (m_3^{(1)}) = (-\ 0.214)\ (-\ 5.03) = +\ 1.08$

$(-\gamma_{25})\ (m_5^{(0)}) = (-\ 0.018)\ (+\ 1.03) = -\ 0.02$

Total $= +\ 1.97 = m_2^{(1)}$

At joint (1)

$m_1^{(0)} = +0.75$

$(-\gamma_{12})\ (m_2^{(1)}) = (-0.469)\ (+1.97) = -0.93$

$(-\gamma_{14})\ (m_4^{(0)}) = (-0.031)\ (+0.67) = -0.02$

Total $= -0.20 = m_1^{(1)}$

In the same way, the higher approximate values of m_γ can be found mechanically by the calculation on the frame scheme. The result of the final calculation (third approximate values) is as follows:

$m_1^{(3)} = -0.38$, $m_2^{(3)} = +2.39$, $m_3^{(3)} = -5.63$, $m_4^{(3)} = +0.33$

$m_5^{(3)} = +0.83$ $m_6^{(3)} = +0.25$ $m_7^{(3)} = -4.08$, $m_8^{(3)} = +0.24$

$m_9^{(3)} = +0.93$ $m_{10}^{(3)} = -0.09$ and $m_{11}^{(3)} = -3.20$

Step VI: Calculation of end moments—

$M_{1\text{-}2} = 1.5\ (2m_1^{(3)} + m_2^{(3)}) = 1.5(2\times -0.38 + 2.39) - 2.40 = +0.05$ tm.

$M_{1\text{-}4} = 0.1\ (2m_1^{(3)} + m_4^{(3)}) = 0.1(2\times -0.38 + 0.33) = -0.04$ tm.

Similarly end moments at all other member ends can be calculated.

Step VII: Now we will consider the equilibrium condition of joint (1)

$\Sigma M_1 = +0.05 - 0.04 = +0.01$

As this is negligible it may be presumed that the calculations for the joint (1) are correct.

Step VIII: Correction of end moments.—

For joint (2)

$M_{2\text{-}1} = 1.5[2\ (+2.39) - 0.38] + 2.40 = +9.00$ tm.

$M_{2\text{-}3} = 1.2[2\ (+2.39) - 5.63] - 8.60 = -9.62$ tm.

$M_{2\text{-}5} = 0.1[2(+2.39) + 0.83] = +0.56$ tm.

$\Sigma M_2 = +9.00 - 9.62 + 0.56$

$= -0.06$ tm.

Distributing this unbalanced moment

Correction moment for

$2\text{—}1 = +0.06 \times \frac{1.5}{2.8} = +0.03$

$2\text{—}3 = +0.06 \times \frac{1.2}{2.8} = +0.03$

$2\text{—}5 = 0.06 \times \frac{0.1}{2.8} = +0.00$

Final moments after correction

$M_2 = +9.00 + 0.03 = +9.03$ tm.

$M_2 = -9.62 + 0.03 = -9.59$ tm.

$M_2 = +0.56 + 0.00 = +0.56$ tm.

The amount of correction, however, is very small at joints 1, 5, 8, 9, 10 & 11 and may be disregarded for the purpose of designing sections.

4.11.3.2 *Example involving sidesway considerations* (Refer Fig. 4.14).—The calculations of $\bar{M}$ and $m^{(0)}$ (Steps I to IV) are the same as in the case of non-translatory joints.

Step V : Calculation of t:

Top most storey :

$$t_{14} = t_{25} = t_{36} = 3/2 \left(\frac{0.1}{0.3}\right) = 0.500$$

Middle storey :

$$t_{48} = t_{59} = t_{610} = t_{711} = 3/2 \left(\frac{0.2}{0.8}\right) = 0.375$$

Bottom storey left two columns

$$T_{8A} = t_{9B} = 3/2 \left(\frac{0.2}{1.0}\right) = 0.300$$

Right two columns

$$t_{10}C = t_{11}D = 3/2 \left(\frac{0.3}{1.0}\right) = 0.450$$

The values of γ, t, and $m^{(0)}$ are written in the corresponding joint and storey as shown in Fig. 4.13.

Step VI : Calculation of displacement moment $\bar{m}_u^{(0)}$ etc.

For the topmost storey :

$$m_T^{(0)} = (m_1^{(0)} + m_4^{(0)})(-t_{14}) + (m_2^{(0)} + m_5^{(0)})(-t_{25}) + (m_3^{(0)} + m_6^{(0)})(-t_{36})$$

$$= (+0.75 + 0.67)(-0.500) + (1.11 + 1.03)(-0.500) + (-4.54 - 0.59)(-0.500)$$

$$= +0.79 \text{ tm}$$

For the middle storey : $\bar{M}_m^{(0)}$

$$= (+0.67 + 0.63)(-0.375) +$$
$$(+1.03 + 1.00)(-0.375) +$$
$$(-0.59 - 0.55)(-0.375) +$$
$$(-4.21 - 3.47)(-0.375) = +2.06 \text{ tm}$$

For the bottom storey : $\bar{M}_b^{(0)}$

$$= (+0.63)(-0.300) + (+1.00)(-0.300) +$$
$$(-0.55)(-0.450) + (-3.47)(-0.450)$$
$$= +1.32 \text{ tm.}$$

FRAME ANALYSIS TAKING THE EFFECT OF SIDESWAY

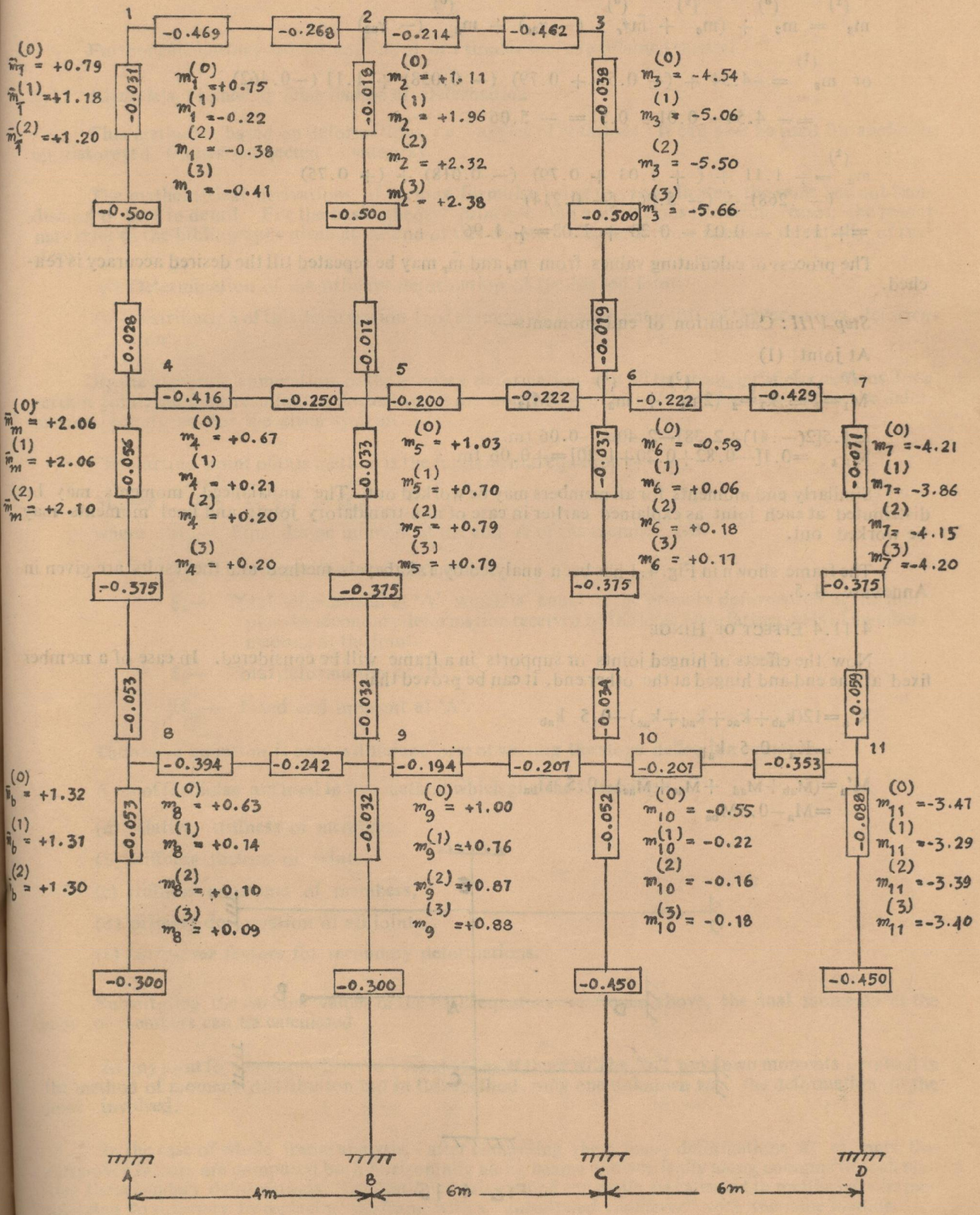


FIG. 4.14

Step VII : Calculation of $m_1^{(1)}$, $m_2^{(1)}$ etc.

At joint (3)

$$m_3^{(1)} = m_3^{(0)} + (m_6^{(2)} + \bar{m}_r^{(0)})(-\gamma_{36}) + m_2^{(0)}(-\gamma_{32})$$

or $$m_3^{(1)} = -4.54 + (-0.59 + 0.79)(-0.038) + 1.11(-0.462)$$

$$= -4.54 - 0.01 - 0.51 = -5.06$$

$$m_2^{(1)} = +1.11 + (+1.03 + 0.79)(-0.018) + (+0.75)(-.268) + (-5.06)(-0.214)$$

$$= +1.11 - 0.03 - 0.20 + 1.08 = +1.96$$

The process of calculating values from m_r and $\bar{m}_r$ may be repeated till the desired accuracy is reached.

Step VIII : Calculation of end moments—

At joint (1)

$$M_{1-2} = k_{1-2}(2m_1^{(3)} + m_2^{(3)}) - M_{12}$$

$$= 1.5[2(-.41) + 2.38] - 2.40 = -0.06 \text{ tm.}$$

$$M_{1-4} = 0.1[-0.82 + 0.20 + 1.20] = +0.06 \text{ tm.}$$

Similarly end moments for all members may be worked out. The unbalanced moments may be distributed at each joint as explained earlier in case of non-tranalatory joints and final moments may be worked out.

The frame shown in Fig. 4.1 has been analysed by Takabeya's method and the results are given in Annexure 4.5.

4.11.4 Effect of Hinge

Now the effects of hinged joints or supports in a frame will be considered. In case of a member fixed at one end and hinged at the other end, it can be proved that

$$K'_a = 12(k_{ab} + k_{ac} + k_{ad} + k_{ae}) - 0.5\ k_{ab}$$

$$= K_a - 0.5\ k_{ab}$$

$$M'_a = (\bar{M}_{ab} + \bar{M}_{ad} + \bar{M}_{ac} + \bar{M}_{ae}) - 0.5\ \bar{M}_{ba}$$

$$= M_a - 0.5 M_{ba}$$

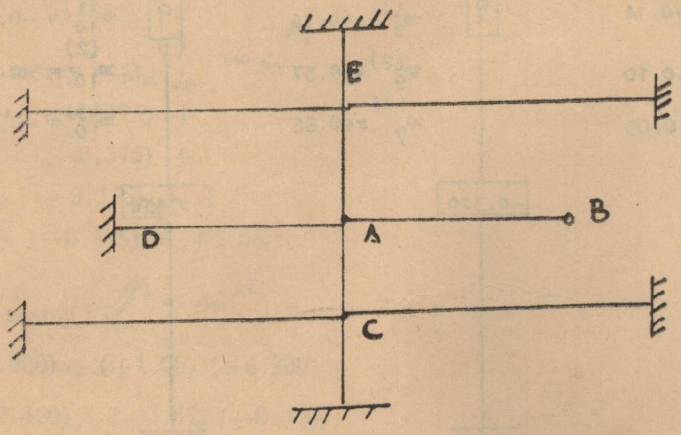


FIG 4.15

To start with, assume $m_c = m_e = m_d = 0$.

we get for hinged condition $m_a^{(0)} = \frac{M'_a}{K'_a}$

Further calculations can be continued in a similar way as explained earlier.

4.12 Kloucek's Method of Distribution of Deformation

This method is based on deformations *i.e.*, angles of rotations. It can also be used for analysing multistoreyed frames subjected to sidesway.

The mathematical derivations of various formulae being too complicated, the same has not been discussed here in detail. For the basic theory, principle and applications to various cases, the reader may refer to the bibliography given at the end of this chapter. The procedure consists essentially of two steps :

(i) Determination of the primary deformation of the loaded joint.

(ii) Distribution of this deformation (not moments), to adjacent joints as the secondary deformation.

By the algebraic summation of the primary deformation and all the secondary deformations for a certain joint, the same resultant deformation is obtained as is obtained by the solution of all the deformation equations for the given system.

The starting point of this method is the fundamental equation :—

$$M_{ab} = k_{ab} \ (2\delta_a + \delta_b) + \bar{M}_{ab}$$

where M_{ab}— Final design moment at the end A of the member A-B.

k_{ab}— relative stiffness of the member.

δ_a— Total deformation at 'A' which is equal to the primary deformation at the joint plus the secondary deformation received by the joint from farther ends of members meeting at the joint.

δ_b— Total deformation at 'B'.

$\bar{M}_{ab}$— Fixed end moment at 'A'.

The above equation is only a different form of writing the slope deflection equation.

A set of formulae are used in this method which give the following :—

(a) relative stiffness of members,

(b) stiffness factors of joint.

(c) stiffness constants of members,

(d) primary deformation of all joints,

(e) carry-over factors for secondary deformations.

Substituting the various values in the basic equation mentioned above, the final moments at the ends of members can be calculated.

At any joint for instance where "n" members meet there will be "n" unknown moments involved in the method of moment distribution but in this method only one unknown *viz.*, the deformation of the joint involved.

In the case of whole frame analysis, after computing the primary deformations at all jonts, the carry-over factors are computed both horizontally along beams and vertically along columns for calculating the secondary deformations. Application of this method can easily be extended to multi-storey frames subjected to sidesway by replacing the frame with a "*substituted cantilever*" with the same loading.

4.13 Relative Advantages of Different Methods

4.13.1 Floorwise Analyis

The main advantage of floorwise analysis of frames by Hardy Cross (moment distribution) method or two-cycle method is that floors can be isolated for design purposes while the construction is in progress and drawings being supplied as the work proceeds. Whenever the moments are likely to be affected due to large variations in the span lengths and loadings, floor-wise analysis of frames by moment distribution method has to be done for various alternate arrangements of live load. But if the ratio of live load to total load is low say 0.4, floor-wise analysis of frames by moment distribution method assuming all spans loaded with live load gives results which are sufficient for design purposes. Two-cycle method gives maximum moments for worst conditions of loading in one operation and is, therefore, a convenient method suitable for adoption in most cases.

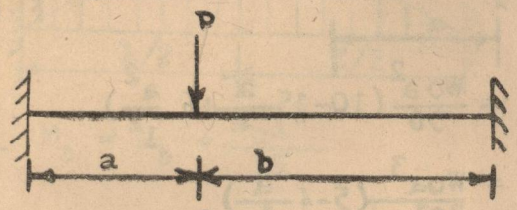
4.13.2 Whole Frame Analysis

Moment distribution method for analyising the whole frame for worst conditions of loading is very laborious and is, therefore, not used in design offices. But if the ratio of live load to total load is low, this method of analysis for the whole frame assuming all spans loaded with live load gives results which are sufficiently accurate for design purposes. It, however, suffers from one draw-back *i.e.* in case of unsymmetrical frames or unsymmetrical loading, calculations taking into account the effect of sidesway are extremely cumbersome. Kani's and Takabeya's methods for analysing whole frames are simple and accurate for all practical purposes. Takabeya's method requires less calculations than Kani's method and, therefore, requires less time for the analysis. But in Kani's method any error which may creep in during the analysis automatically gets eliminated during the process of analysis Kloucek's deformation distribution method is very laborious and is, therefore, not generally used for design purposes.

In Takabeya's method any such errors are also eliminated but only where sidesway effects are not taken into consideration.

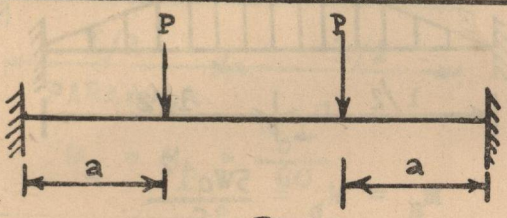
FIXED END MOMENTS FOR PRISMATIC BEAMS

1



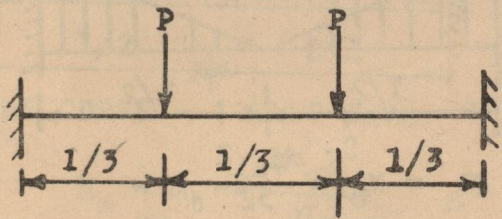
$M_a = \frac{Pab^2}{l^2}$ $M_b = \frac{Pba^2}{l^2}$

2



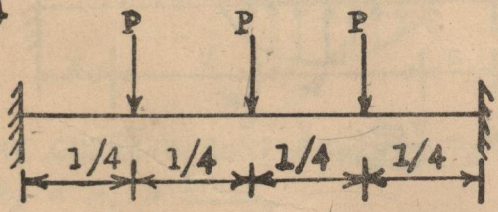
$M_a = M_b = \frac{Pa}{l}(1-a)$

3



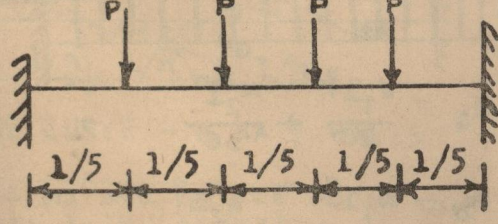
$M_a = M_b = \frac{2Pl}{9}$

4



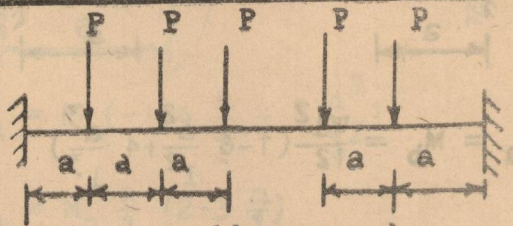
$M_a = M_b = \frac{5Pl}{16}$

5



$M_a = M_b = \frac{2Pl}{5}$

6

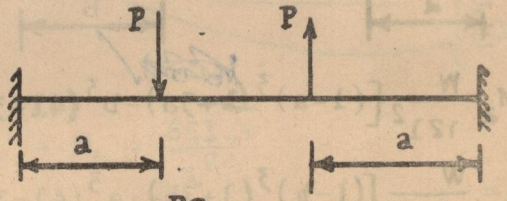


n loads, equally spaced

$a = \frac{l}{n+1}$

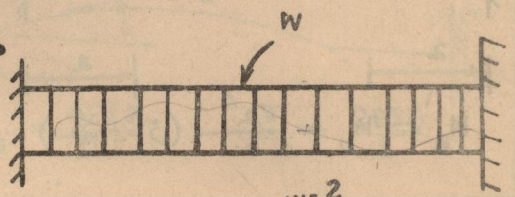
$M_a = M_b = \frac{Pa}{12} n(n+2)$

7



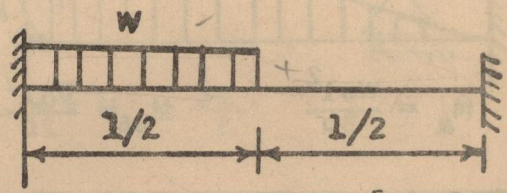
$M_a = M_b = \frac{Pa}{l^2}(1-a)(1-2a)$

8



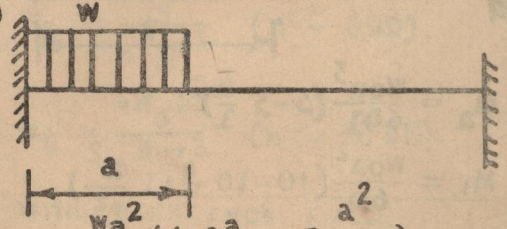
$M_a = M_b = \frac{Wl^2}{12}$

9



$M_a = \frac{11}{192} Wl^2$ $M_b = \frac{5}{192} Wl^2$

10

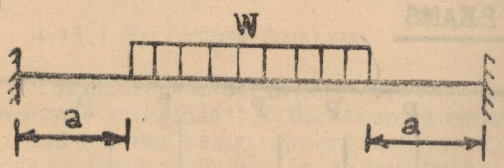


$M_a = \frac{Wa^2}{12}(6 - 8\frac{a}{l} + 3\frac{a^2}{l^2})$

$M_b = \frac{Wa^2}{12l}(4 - 3\frac{a}{l})$

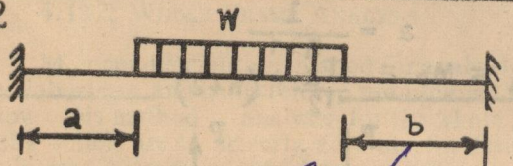
ANNEXURE 4.1 Contd...

11



$$M_a = M_b = \frac{Wl^2}{12}\left(1-6\frac{a^2}{l^2}+4\frac{a^3}{l^3}\right)$$

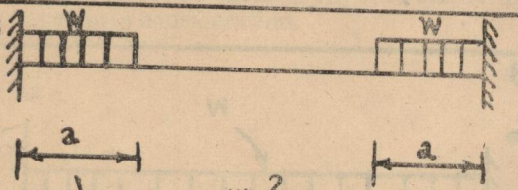
12



$$M_a = \frac{W}{12l^2}\left[(l-a)^3(l+3a)-b^3(4l-3b)\right]$$

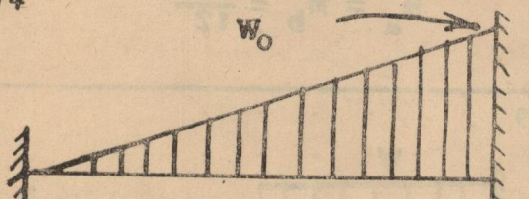
$$M_b = \frac{W}{12l^2}\left[(l-b)^3(l+3b)-a^3(4l-3a)\right]$$

13



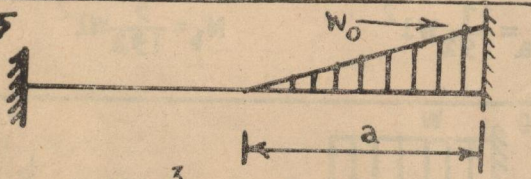
$$M_a = M_b = \frac{Wa^2}{6}\left(3-2\frac{a}{l}\right)$$

14



$$M_a = \frac{Wol^2}{30} \qquad M_b = \frac{Wol^2}{20}$$

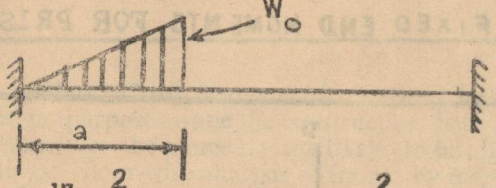
15



$$M_a = \frac{Woa^3}{60l}\left(5-3\frac{a}{l}\right)$$

$$M_b = \frac{Woa^2}{60}\left(10-10\frac{a}{l}+3\frac{a^2}{l^2}\right)$$

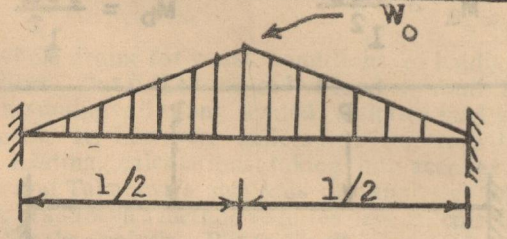
16



$$M_a = \frac{Woa^2}{30}\left(10-15\frac{a}{l}+6\frac{a^2}{l^2}\right)$$

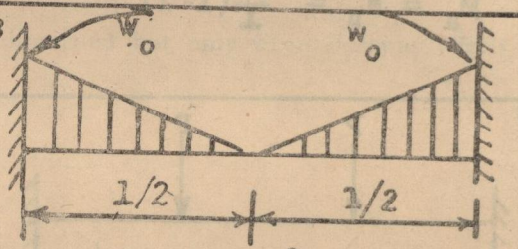
$$M_b = \frac{Woa^3}{20l}\left(5-4\frac{a}{l}\right)$$

17



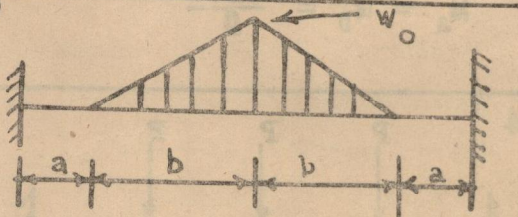
$$M_a = M_b = \frac{5Wol^2}{96}$$

18



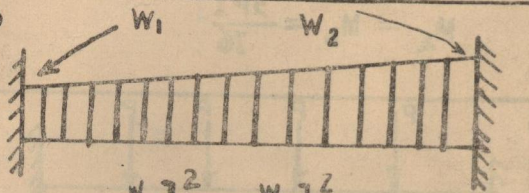
$$M_a = M_b = \frac{Wol^2}{32}$$

19



$$M_a = M_b = \frac{Wob}{12l}\left(6a^2+12ab+5b^2\right)$$

20

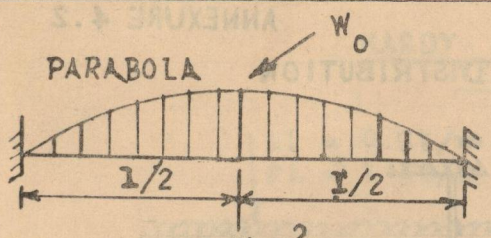


$$M_a = \frac{W_1l^2}{20}+\frac{W_2l^2}{30}$$

$$M_b = \frac{W_1l^2}{30}+\frac{W_2l^2}{20}$$

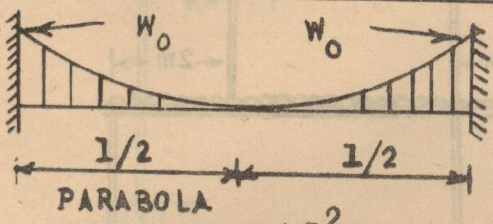
ANNEXURE 4.1 CONTD...

21



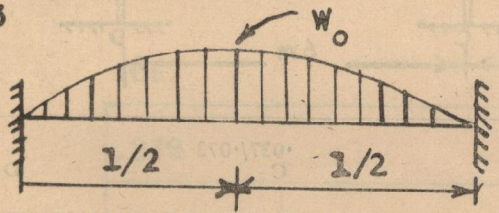
$$M_a = M_b = \frac{W_o l^2}{15}$$

22



$$M_a = M_b = \frac{W_o l^2}{60}$$

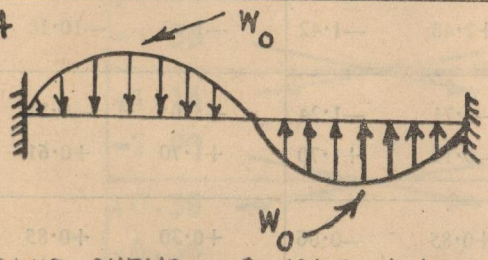
23



SINE CURVE - 1 HALF WAVE

$$M_a = M_b = \frac{2W_o l^2}{\pi^3}$$

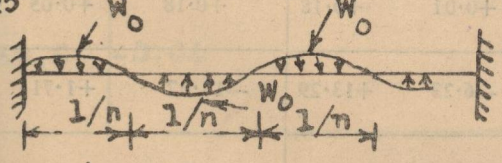
24



SINE CURVE - 2 HALF WAVES

$$M_a = M_b = \frac{3W_o l^2}{4\pi^3}$$

25

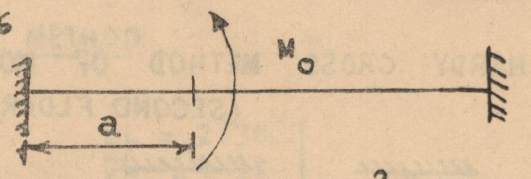


SINE CURVE - n HALF WAVES

$$M_a = M_b = \frac{2W_o l^2}{n^3\pi^3} \quad (n - \text{ODD})$$

$$M_a = M_b = \frac{6W_o l^2}{n^3\pi^3} \quad (n - \text{EVEN})$$

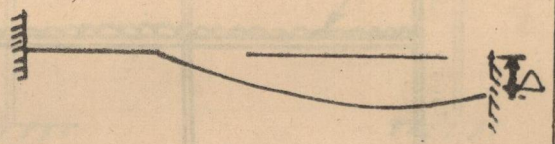
26



$$M_a = M_o\left(-1+4\frac{a}{l} - 3\frac{a^2}{l^2}\right)$$

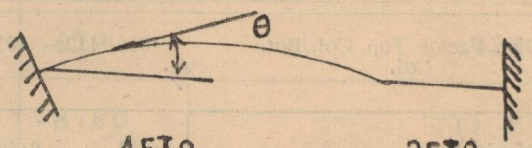
$$M_b = M_o \frac{a}{l}\left(2-3\frac{a}{l}\right)$$

27



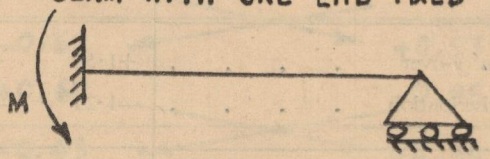
$$M_a = M_b = \frac{6EI\Delta}{l^2}$$

28



$$M_a = \frac{4EI\theta}{l} \qquad M_b = \frac{2EI\theta}{l}$$

29 BEAM WITH ONE END FIXED



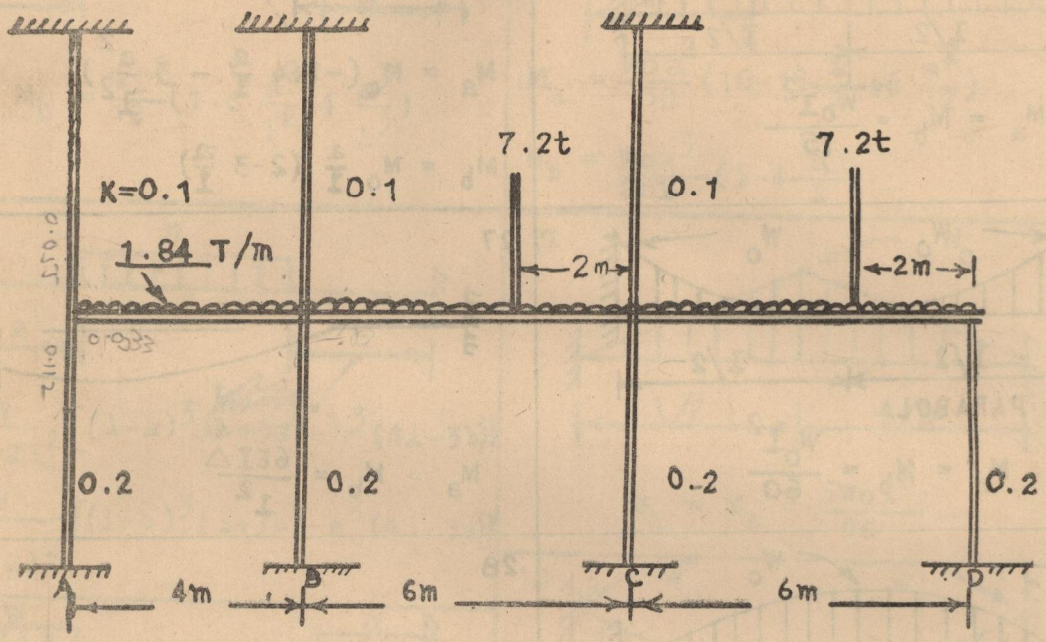
$$M = M_a - \frac{Mb}{2}$$

WHERE Ma AND Mb ARE FOUND FOR A BEAM WITH BOTH ENDS FIXED (CASES 1-28. ABOVE)

EXTRACTS FROM "MOMENT DISTRIBUTION" BY J.M. GERE.

ANNEXURE 4.2

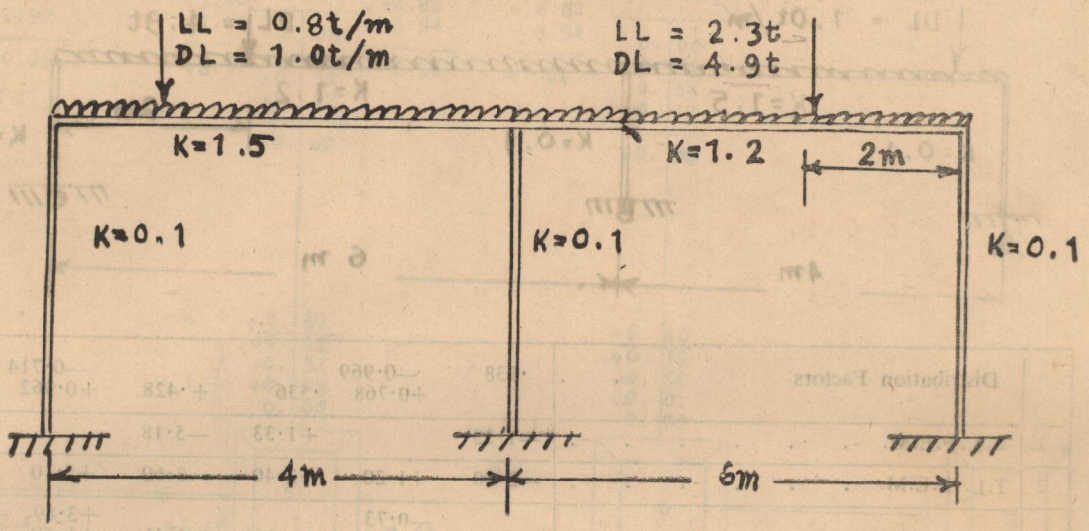
HARDY CROSS METHOD OF MOMENT DISTRIBUTION
(SECOND FLOOR)



Dist. Factor Top Col./Bot. Col.	·055/·112 A		·033/·067 B		·037/·073 C	D
Distribution factor	0·833	0·500	0·400	0·445	0·445	·857
F.E.M.	—2·40	+2·40	—8·60	+11·80	—8·60	+11·80
Distribution	+2·00	+3·10	+2·48	—1·42	—1·42	—10·10
Carry-over	+1·55	+1·00	—0·71	—1·24	—5·05	—0·71
Distribution	—1·29	—0·15	—0·12	+1·70	+1·70	+0·61
Carry-over	—0·08	—0·65	+0·85	—0·06	+0·30	+0·85
Distribution	+0·07	—0·10	—0·08	—0·11	—0·11	—0·73
Carry-over	—0·05	+0·04	—0·06	—0·04	—0·37	—0·06
Distribution	+0·04	+0·01	+0·01	+0·18	+0·18	+0·05
Total Design Moments	—0·16	+5·65	—6·23	+13·29	—13·37	+1·71
Top Column	+0·05	+·19			+0·03	..
Bottom Column	+0·11	+0·39			+0·05	—1·71

ANNEXURE 4.2

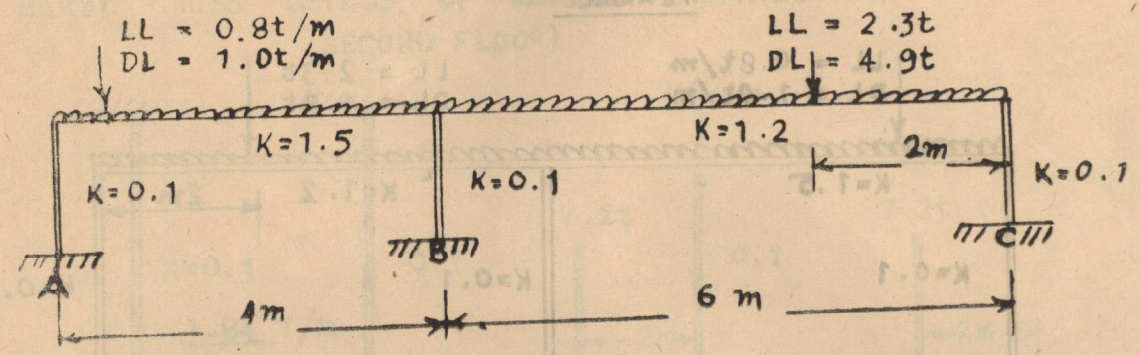
HARDY CROSS METHOD
TERRACE



	.062	.036		.077
	.938	.536	.428	.923
	-2.40	+2.40	-8.60	+11.80
	+2.250	+3.32	+2.66	-10.90
	+1.66	+1.13	-5.45	+1.33
	-1.56	+2.32	+1.85	-1.23
	+1.16	-0.78	-0.62	+0.93
	-1.09	+0.75	+0.60	-0.86
	+0.38	-0.55	-0.43	+0.30
	-0.36	+0.53	+0.42	-0.28
BEAM MOMENTS	+0.04	+9.12	-9.57	+1.09
COL. MOMENTS	-0.04		+0.45	-1.09

ANNEXURE 4.3

TWO CYCLE METHOD (TERRACE LEVEL)

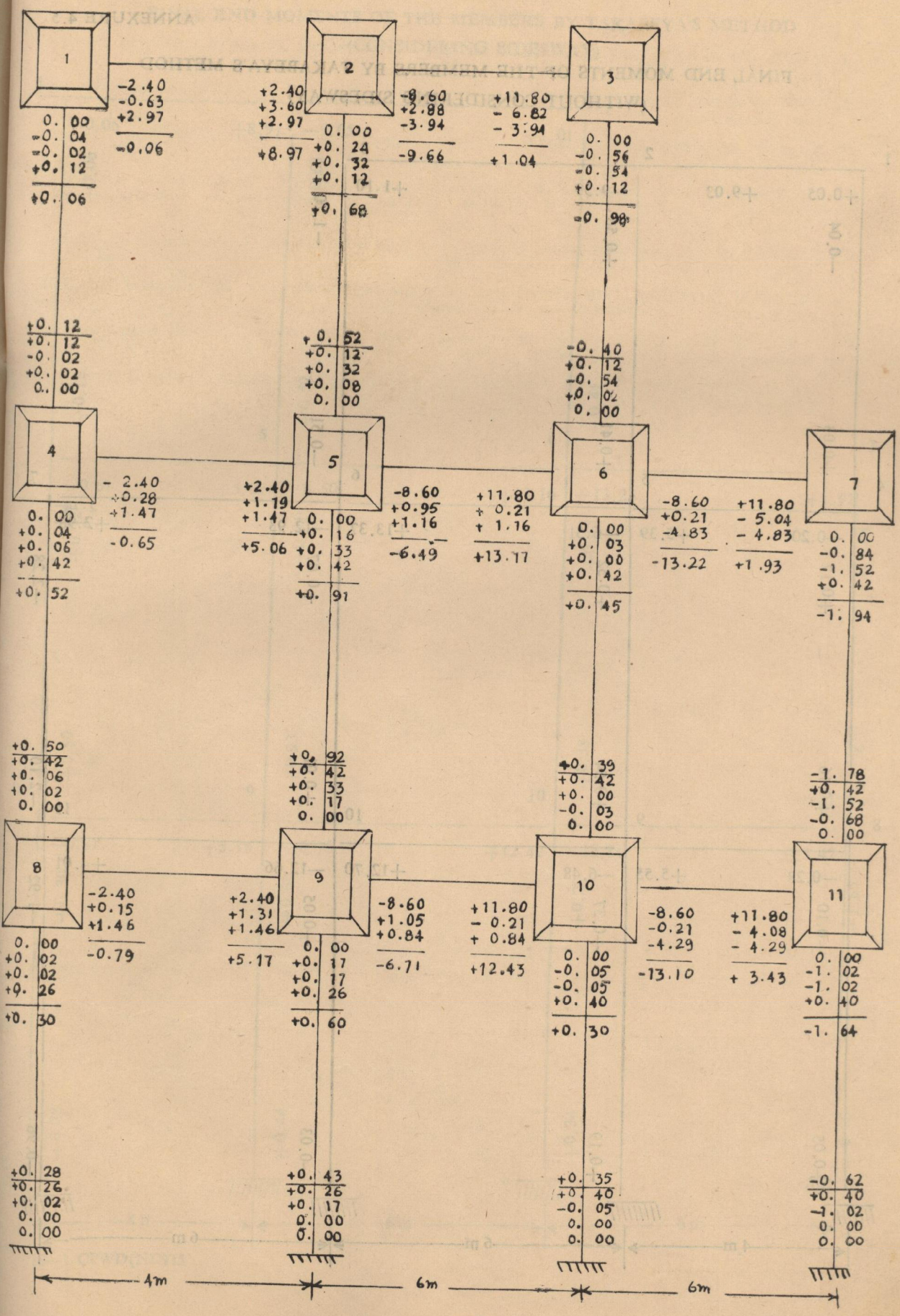


	Distribution Factors . .	·938	—0·969 +0·768	·536	+·428	—0·714 +0·962	·923
1	D.L.F.E.M.	—1·33		+1·33	—5·18		+7·36
2	T.L. F.E.M.	—2·40	+1·20	+2·40	—8·60	+5·10	+11·80
3	Distribute & Carry-over . . .	+0·75	—0·73 +0·87	+1·13	—5·45	+3·89 +1·50	+1·56
4	ADD (2) & (3)	—1·65		+3·53	—14·05		+13·36
5	Distribute	+1·55		+5·64	+4·50		—12·32
6	ADD (4) & (5)	—0·10	+1·34	+9·17	—9·55	+10·49	+1·04

MAXIMUM MOMENTS IN COLUMNS

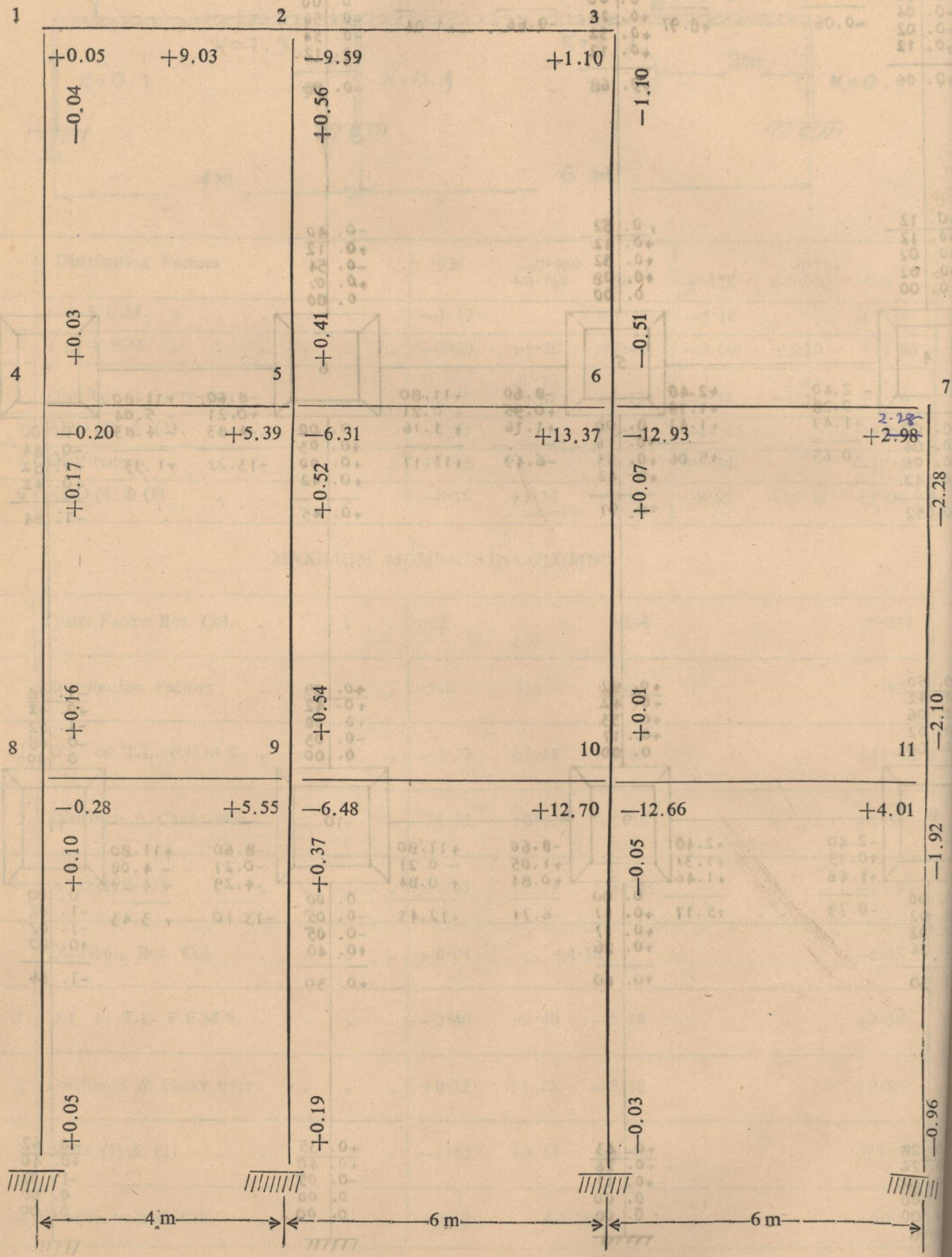
	Distr. Factor Bot. Col.	·062	·036		·0·077
	Distribution Factors	·938	·536	·428	·923
1	D.L. or T.L. F.E.M S.	—1·33	+1·33	—8·60	+11·80
2	Distribute & Carry over	+1·95	+0·62	—5·45	+1·56
3	ADD (1) & (2)	+0·62	+1·95	—14·05	+13·36
	Distribute Bot. Col.	—0·04	+4·36		—1·03
1	D.L. or T.L. F.E.M.S.	—2·40	+2·40	—5·18	+7·36
2	Distribute & Carry over . . .	+0·75	+1·13	—3·40	+0·59
3	ADD (1) & (2)	—1·65	+3·53	—8·58	+7·95
4	Distribute Bot. Col.	+0·10	+0·18		—0·61

ANALYSIS OF THE FRAME BY KANI'S METHOD (CONSIDERING SIDESWAY)

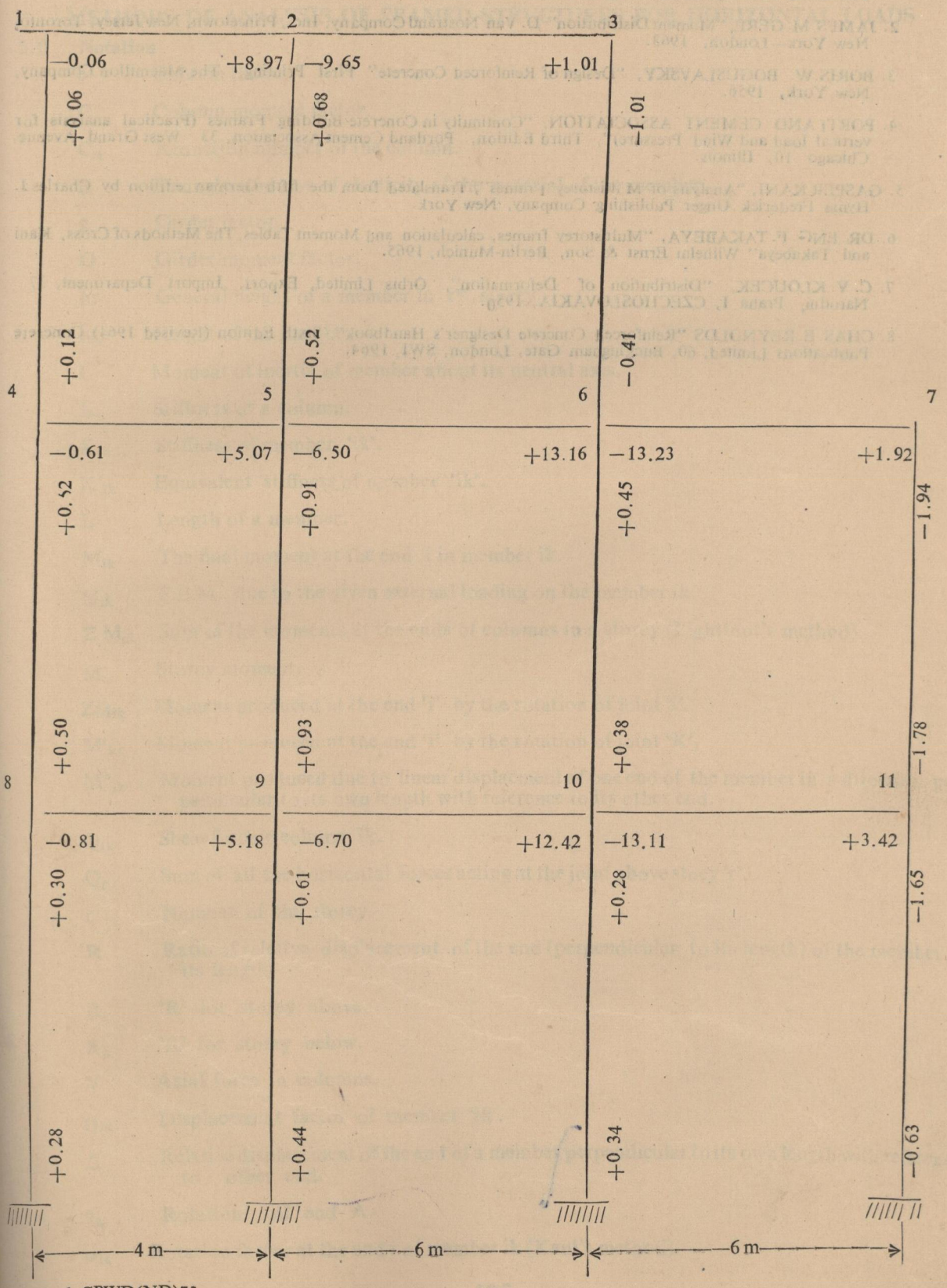


ANNEXURE 4.5

FINAL END MOMENTS OF THE MEMBERS BY TAKABEYA'S METHOD (WITHOUT CONSIDERING SIDESWAY)



FINAL END MOMENTS OF THE MEMBERS BY TAKABEYA'S METHOD (CONSIDERING SIDESWAY)



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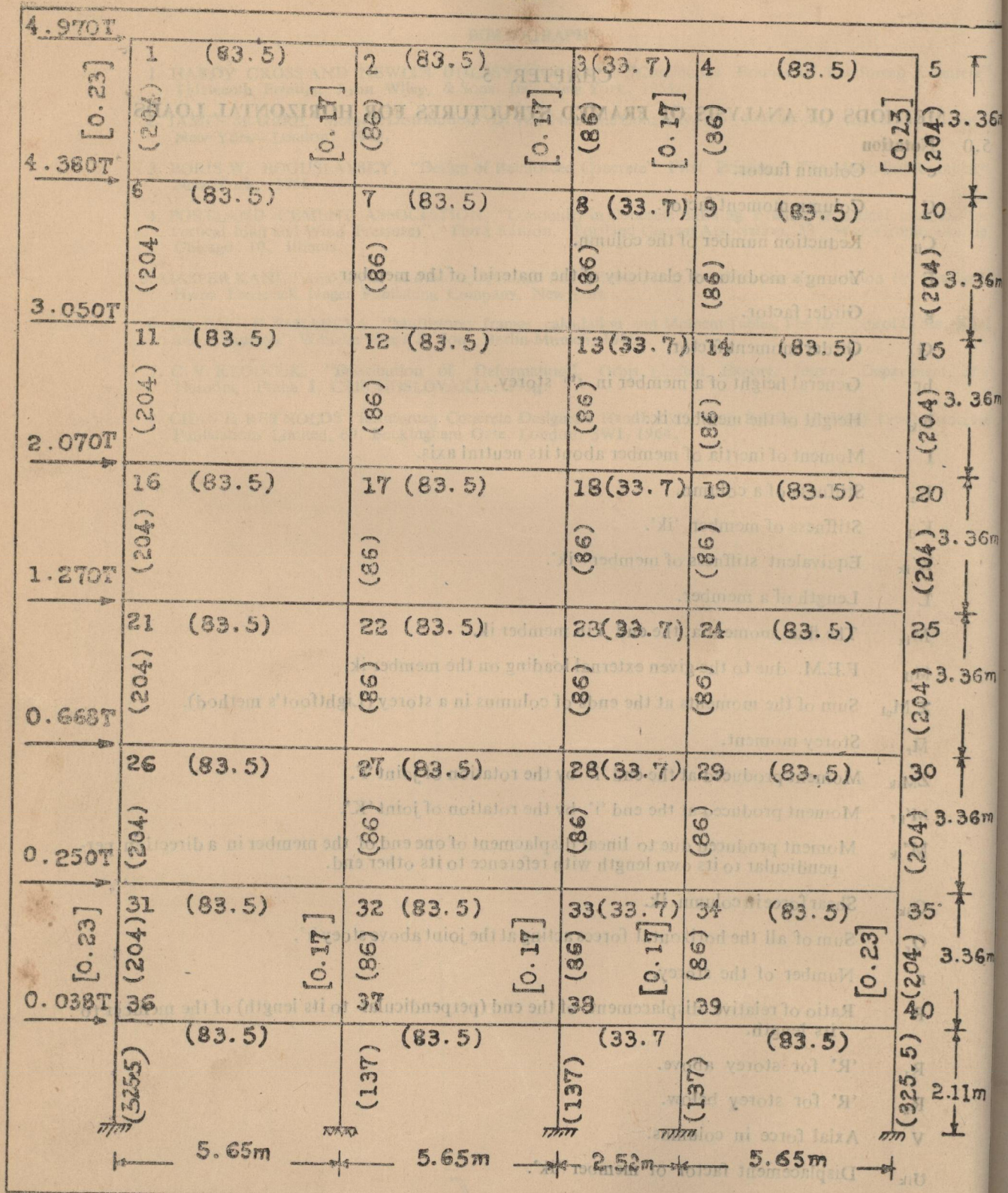
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CHAPTER 5

METHODS OF ANALYSIS OF FRAMED STRUCTURES FOR HORIZONTAL LOADS

5.0 Notation

c	Column factor.
C	Column moment factor.
C_{ik}	Reduction number of the column.
E	Young's modulus of elasticity of the material of the member.
g	Girder factor.
G	Girder moment factor.
hr	General height of a member in r^{th} storey.
h_{ik}	Height of the member ik.
I	Moment of inertia of member about its neutral axis.
K_c	Stiffness of a column.
K_{ik}	Stiffness of member 'ik'.
K'_{ik}	Equivalent stiffness of member 'ik'.
L	Length of a member.
M_{ik}	The final moment at the end i in member ik.
$\bar{M}_{ik}$	F.E.M. due to the given external loading on the member ik.
ΣM_{cl}	Sum of the moments at the ends of columns in a storey (Lightfoot's method).
$\bar{M}_r$	Storey moment.
$Z\bar{M}ik$	Moment produced at the end 'i' by the rotation of joint 'i'.
M'_{ki}	Moment produced at the end 'i' by the rotation of joint 'K'.
M''_{ik}	Moment produced due to linear displacment of one end of the member in a direction perpendicular to its own length with reference to its other end.
Q_{ik}	Shear force in column ik.
Q_r	Sum of all the horizontal forces acting at the joint above stoey 'r'.
r	Number of the storey.
R	Ratio of relative displacement of the end (perpendicular to its length) of the member to its length.
R_1	'R' for storey above.
R_2	'R' for storey below.
V	Axial force in columns.
υ_{ik}	Displacement factor of member 'ik'.
Δ	Relative displacement of the end of a member perpendicular to its own length with reference to other end.
θ_A	Rotation of end A.
ω_{ik}	Rotation factor at the end i of member ik (Kani's method).



Notes: (1) I(K) values are given against each member in small brackets.

(2) Cross-sectional areas of columns in square metres are given in square brackets.

(3) T stands for tonne.

FIG. 5·1

5.1 Methods of Analysis

A number of methods are available to analyse framed structures for horizontal loads. Some of these are :—

(a) Approximate methops :
  (i) Cantilever method
  (ii) Portal method

(b) Improved methods :
  (i) Bowman's methods
  (ii) Factor method
  (iii) Lightfoot's method

(c) Exact methods :
  (i) Slope Deflection or matrix methods.
  (ii) Hardy Cross method of moment distribution or Kani s method (provided sufficient number of iterations are carried out).
  (iii) Kloucek's distribution of deformation method (provided sufficient number of iteration are carried out).

Approximate methods may be used for preliminary design only. Any of the improved methods or Kani's method may be used for detailed design. In case of frames where principle of multiples applies (see para 5.7.1) it is advantageous to use Naylor's procedure given in Lightfoot's method. But in case of frames where the principle of multiples does not apply, Lightfoot's method involves more calculations than Factor and Bowman's methods and, therefore, requires more time for the analysis. Between Bowman's and Factor methods the latter follows a more systematic procedure and is easier to adopt.

Kani's method, though exact, has a limitation that the calculations become increasingly laborious with increase in number of joints beyond a certain limit. For practical purposes, this limit may be taken as 25 to 30 joints in a frame. Other exact methods are laborious. They are, therefore, usded when computer facilities are available.

5.2 Distribution of Horizontal Force Among Various Frames

Before the actual analysis of a frame is carried out by any of the above mentioned methods, the calculated horizontal force (acting on the whole structure) at every floor level is first destributed among the various frames in proportion to their relative stiffnesses. The relative stiffness of any frame may be taken as the sum of the relative stiffnesses of columns in that frame.

5.3 Cantilever Method

The assumptions made in this method have been enumerated in para 3.8.1 of Chapter 3. The steps to be followed for application of this method for analysis of frame shown in Fig 5.1 are given below :—

Step 1 : Unit Direct Stressess in Columns

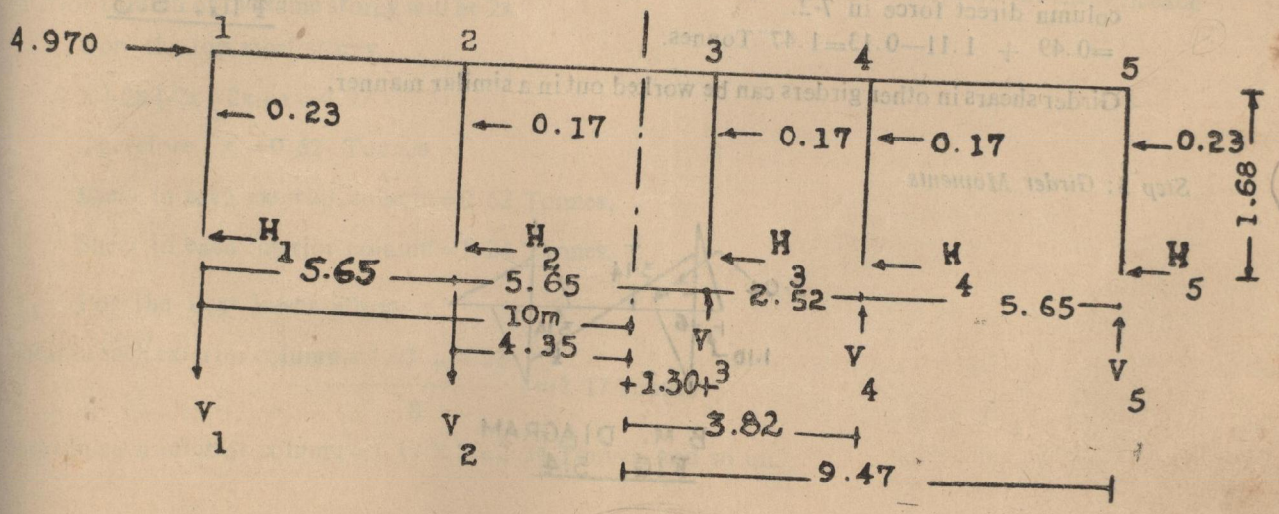


FIG. 5.2

Taking moments of the areas of the columns of top storey about C.G. of column 1-6 we get :

Distance of C. G. of the bent from C.G. of column 1-6

$$=\frac{0.17\times 5.65+0.17\times 11.30+0.17\times 13.82+0.23\times 19.47}{3\,(0.17)+2\,(0.23)}=10.0 \text{ metre.}$$

As per assumptions the unit direct stresses in the columns vary as the distances of the columns from the centre of gravity of the bent. Therefore

$$\frac{V_1}{0.23}=\frac{V_2}{0.17}=\frac{V_3}{0.17}=\frac{V_4}{0.17}=\frac{V_5}{0.23} \qquad \text{.....(5.1)}$$
$$10 \quad 4.35 \quad 1.30 \quad 3.82 \quad 9.47$$

By taking moments about the point of inflection in Col. 5-10,

$$4.97\times 1.68-V_1\times 19.47-V_2\times 13.82+V_3\times 8.17+V_4\times 5.65=0 \qquad \text{.....(5.2)}$$

Substituting the values of V_2, V_3, V_4 and V_5 in terms of V_1 from equation 5.1 in equation 5.2 we get the value of V_1, V_2, V_3, V_4, and V_5 can then be calculated from equation 5.1 $V_1=0.39$; $V_2=0.13$; $V_3=0.04$; $V_4=0.11$ and $V_5=0.37$

For other storeys, column direct forces are evaluated in a similar manner. These are given in Annexure 5.1.

Step 2 : Girder Shears

The girder shears are obtained from the column direct forces at the various joints :

For example,

Shear in girder 6-7= column direct force in 6-11—column direct force in 1-6=1.50—0.39

=1.11 Tonnes.

Shear in girder 7-8

=column direct force in 7-12+shear in girder 7-6-column direct force in 7-2.

=0.49 + 1.11—0.13=1.47 Tonnes.

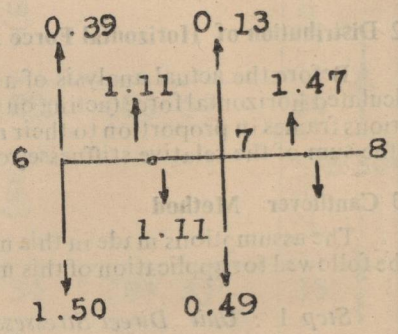
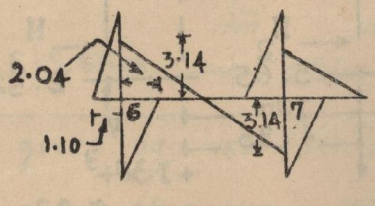


FIG. 5.3

Girder shears in other girders can be worked out in a similar manner.

Step 3: Girder Moments



B.M. DIAGRAM

FIG. 5.4

Since it is assumed that the moment at the centre of each girder is zero, the moment at each end of a girder equals the shear in that girder multiplied by half of the length of that girder.

For example: $M_{1\text{-}2} = 0.39 \times \frac{5.65}{2} = 1.10$ Tm.

$M_{6\text{-}7} = 1.11 \times \frac{5.65}{2} = 3.14$ Tm etc.

Step 4 : *Column Moments*

For a joint to be in equilibrium, sum of the girder moments should be equal to the sum of the column moments meeting at that joint. Column moments are calculated beginning at the top of each column stack and working progressively towards its base.

For example :

at joint (1) : the column moment equals the girder moment i.e. $M_{1\text{-}6} = 1.10$ Tm.

at joint (6) : $M_{6\text{-}1} + M_{6\text{-}11} = M_{6\text{-}7} = 3.14$ Tm.

Since as per assumption the points of inflection are located at mid heights of columns.

$M_{6\text{-}1} = M_{1\text{-}6} = 1.10$ Tm.

$M_{6\text{-}11} = 3.14 - 1.10 - 2.04$ Tm. and so on.

Step 5 : *Column Shears*

Since as per assumption the points of inflection are located at mid heights of columns.

$$\text{Shear in column 1-6} = \frac{\text{B M at end 1 or 6 of the column}}{\text{half the length of the column.}} = \frac{1.10}{1.68} = 0.65 \text{ Tonnes.}$$

$$\text{Shear in column 6-11} = \frac{2.04}{1.68} = 1.21 \text{ Tonnes etc.}$$

The frame shown in Fig. 5.1 has been analysed by this method and results are shown in Annexure 5.1.

5.4 Portal Method

The assumptions made in this method have been enumerated in para 3.8.2 of Chapter 3.

The steps to be followed for application of this method for analysis of the frame shown in fig. 5.1 are given below :—

Step 1 : *Column Shears*

Let x be the shear in each exterior column of a given storey, then by assumption the shear in each interior column of the same storey will be 2x

For the top most storey,

$x + 2x + 2x + 2x + x = 4.97$

Therefore, $x = 0.62$ Tonnes.

Shear in each exterior column = 0.62 Tonnes.

Shear in each interior column = 1.24 Tonnes.

For the next lower storey :

$$\text{Shear in each exterior column} = \frac{4.97 + 4.38}{8} = 1.17 \text{ Tonnes}$$

Shear in each interior column = $1.17 \times 2 = 2.34$ Tonnes, and so on.

Step 2: Column Moments

Since as per assumption the points of inflection are located at mid heights of columns, each e moment for a given column equals the shear on that column multiplied by half the length of that colum

For example:

$M_{1\text{-}6}=0.62\times 1.68=1.04$ Tm.

$M_{6\text{-}11}=1.17\times 1.68=1.97$ Tm.

Step 3 : Girder Moments

For any joint the sum of the column end moments equals the sum of the girder end moments.

For joint (1) $M_{1\text{-}6}=M_{1\text{-}2}=1.04$ Tm.

For joint (6) $M_{6\text{-}1}+M_{6\text{-}11}=M_{6\text{-}7}=1.04+1.97=3.01$ Tm. and so on.

Since as per assumption there is a point of inflection at the centre of each girder, therefore $M_{2\text{-}1}=1.04=M_{1\text{-}2}$ and $M_{6\text{-}7}=3.01=M_{7\text{-}6}$ and so on.

Girder end moment at other joints may be determined in a similar manner.

Step 4 : Girder Shears

As per assumption there is a point of inflection at the centre of each girder. Therefore, the shear in the girder is equal to the moment at the end of the girder (obtained from step 3) divided by half the length of the span.

$$\text{For example : Shear in girder 1-2} = 1.04 \times \frac{2}{5.65} = 0.37 \text{ T}$$

$$\text{Shear in girder 6-7} = 3.01 \times \frac{2}{5.65} = 1.07 \text{ T as and on.}$$

Step 5: Column Direct Loads

Column direct loads may be obtained by summing up from the top of the column, the shears applied to the column by the girders.

Thus direct load in Col.1-6 = 0.37 Tonnes.

Thus direct load in Col.6-11 = 1.07 + 0.37 = 1.44 Tonnes and so on.

The frame shown in Fig 5.1 has been analysed by this method and the results are shown in Annexure 5.2.

5.5 Bowman's Method

The following assumptions are made in this method :—

(a) Points of inflection in exterior girders are located at 0.55 of their length from their outer ends and in other girders at their mid points except (i) in the centre bay, where the total number of bays is odd (point of inflection will be at the mid point of the centre girder if the bent is symmetrical) and (ii) in the two bays nearest the centre, where the total number of bays is even. In these excepted cases the points of inflection in girders will be located as required by the conditions of symmetry and equilibrium.

(b) In bents of one or more storeys the points of inflection in bottom storey columns are at 0.60 height from the base; in bents of two or more storeys the points of inflection in top storey columns are at 0.65 height from the top; in bents of three or more storeys, the points of inflection in the columns of the storey next to the top are at 0.60 height from the upper end in bents of four or more storeys, the points of inflection in the columns of the second storey from the top are at 0.55 height from the upper end; in bents of five or more storeys, the points of inflection in the columns of storeys not provided for above are at mid height.

(c) An amount of shear equal to $\frac{(\text{Number of bays} - \frac{1}{2})}{\text{Number of columns}}$ times the total shear in the storey is divided equally* among the columns of the bottom storey and the remaining shear in the bottom storey is divided among the bays inversely as their widths** and the shear in a bay is divided equally between the two columns of the bay.

(d) An amount of shear equal to $\frac{(\text{Number of bays} - 2)}{\text{Number of columns}}$ times the total shear in the storey is divided equally* among the columns of the other storeys and the remaining shear in the storey is divided among the bays inversely as their widths** and the shear in a bay is divided equally between the two columns of the bay.

NOTE 1.—*Where the column moments of inertia of a storey are not equal, this part of the shear should be divided among the columns in proportion to their moments of inertia.

NOTE 2.—**Where the moments of inertia of the girders above any storey are not equal, this part of the shear should be divided among the bays directly as $\frac{I}{I}$ of the girders above the bays.

The distribution of shear in the columns of the frame shown in fig. 5.1 is illustrated below for only bottom most storey and second storey. Similar calculation as for the second storey may be extended to other storeys also.

Example : Bottom most storey

K value of the external columns = 204

K value of the interior columns = 86

K values of the beams with span 5.65 m = 83.5 and with span 252 = 33.7

Distribution of Shear Among Columns

		A 5.65m	B 5.65m	C 2.52m	D 5.65m	E
1	*Bottom storey*					
	Number of bays $-\frac{1}{2}$ / Number of columns $\times$ shear					
	$\frac{4-\frac{1}{2}}{5} \times 16{\cdot}70 = 11{\cdot}69$ Tonnes					
	This 11·69 Tonnes will be distributed in proportion to moment of inertia of columns (assumption C, Note 1)	3·58	1·51	1·51	1·51	3·58
	Remaining 5·01 Tonnes will be distributed in proportion to K-values of girders (Note 2)*	0·74	1·47	1·03	1·03	0·74
	Total shear resisted by each column	4·32	2·98	2·54	2·54	4·32
			=16·70 Tonnes			
2	*Second storey*					
	Number of bays—2 / Number of columns Total $\times$ shear					
	$4-2 \times 16{\cdot}66 = 6{\cdot}66$ Tonnes					
	This 6·66 Tonnes will be distributed in proportion to moments of inertia of columns (Assumption d, Note 1)	2·04	0·86	0·86	0·86	2·04
	Remaining 10·00 Tonnes will be distributed in proportion to K-values of girders (Note 2)	1·47	2·94	2·06	2·06	1·47
	Total shear resisted by each column	3·51	3·80	2·92	2·92	3·51
				=16·66 Tonnes		

*Column A $= 5{\cdot}01 \times \frac{83{\cdot}5}{284{\cdot}2} \times \frac{1}{2} = 0{\cdot}74$ Tonnes

Column B $= 0{\cdot}74 + 0{\cdot}74 = 1{\cdot}48$ Tonnes say 1·47 Tonnes.

Column C $= 0{\cdot}74 + 5{\cdot}00 \times \frac{33{\cdot}7}{284{\cdot}2} \times \frac{1}{2} = 1{\cdot}030$ Tonnes.

Column D $= 0{\cdot}29 + 0{\cdot}74 = 1{\cdot}03$ Tonnes.

Column E $= 0{\cdot}74$ Tonnes.

19—1 C. P. W. D. (N.D.)/75

The values of shear calculated for all storeys have been written down in the frame at the points of inflection of the columns as shown in Annexure 5.3. Knowing the points of inflection and shear in each column of a storey, moments at the ends of columns can be calculated. Girder moments are then calculated considering the equilibrium of each joint. The point of inflection in the girders are obtained from the assumption (a) and girder shears are then worked out.

The procedure to be adopted for working out moments in columns and beams meeting at the joint 1, 2, 3, 4 & 5 is illustrated below :—

Moment in column 1—6 = Shear force in column 1—6 × lever arm

= 1.05 × 2·18 = 2·29 tonne metre.

Therefore, moment in beam 1—2. = moment in column 1—6 = 2.29 tonne metre.

Moment in beam 2—1 = Shear force × lever arm

= 0.736 × 2.54 = 1.87 tonne metre.

Moment in column 2—7 = Shear force in column 2—7 × lever arm

= 1.13 × 2.180 = 2.46 tonne metre.

Moment in beam 2—3 = Moment in column 2—7 — Moment in beam 2—1

= 2.46 — 1.87 = 0.59 tonne metre.

Moment in column 3—8 = Shear force in column 3—6 × lever arm

= 0.87 × 2.18 = 1.90 Tonne metre.

Since 2—3 and 3—4 are two bays nearest the centre the position of the point of inflection in these beams will be decided by conditions of equilibrium.

Moment in column 3—8 = Moment in beam 3—2 + Moment in beam 3—4.

Further the sum of moments in beams 3—2 and 3—4 is to be divided between the two beams in proportion to their relative stiffnesses.

Therefore, Moment in beam $3-2 = \frac{1.90 \times 83.5}{83.5+33.7}$

= 1.35 Tonne metre.

Moment in beam $3-4 = 1.90 \times \frac{33.7}{83.5+33.7}$

Now moment diagram for the beam 2—3 can be drawn as shown below :—

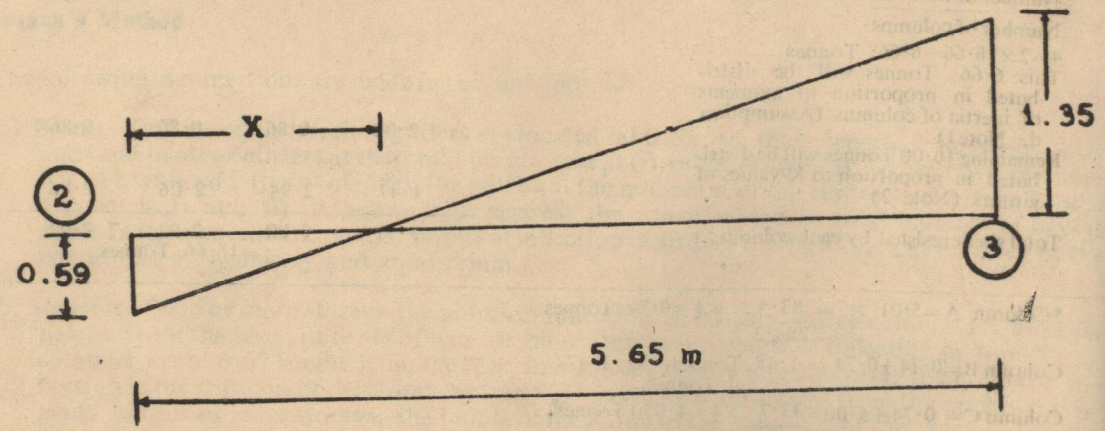


FIG. 5.5

From the above diagram we get

$$\frac{X}{0.59} = \frac{5.65 - X}{1.35}$$

i.e. $1.35X + 0.59X = 5.65 \times 0.59$ or $1.94X = 3.33$

Therefore $X = \frac{3.33}{1.94} = 1.72$ metre

Knowing the distance of point of inflection we can work out the shear in beam 2-3

$$\text{Shear in beam 2-3} = \frac{\text{moment in beam 2-3}}{\text{lever arm}}$$

$$= \frac{0.59}{1.72} = 0.34 \text{ Tonne}$$

Shear in beam 2-3 can also be worked out without locating the point of inflection by dividing the sum of moments at the ten ends of the beam by its span *e.g.* shear in beam 2-3.

$$= \frac{0.54 + 1.35}{5.65} = 0.34$$

We have to now start working out moments in column and beam at joint

5 and come backwards to joint 4

Moment in column 5–10 $= 1.05 \times 2.18$

$= 2.29$ Tonne metre.

Moment in beam 5–4 = Moment in column 5–10

$= 2.29$ Tonne metre.

Shere force in beam 5–4 $= \frac{\text{Moment}}{\text{Lever arm}} \quad \frac{2.29}{3.11}$

$= 0.736$ Tonne

Moment in beam 4–5 = Shear force × Lever arm

$= 0.736 \times 2.54$

$= 1.87$ Tonne metre.

Moment in column 4–9 $= 0.87 \times 2.18$

$= 1.90$ Tonne metre.

Moment in beam 4–3 = (Moment in column 4–9) —
(Moment in beam 4–5)

$= 1.90 - 1.87 = 0.03$ Tonne metre.

Kowing the moment at the two ends of the beam 3–4, point of inflection and shear force in beam 3–4 can be worked out as shown for beam 2–3.

The frame shown in Fig 5.1 has been analysed by this method and the results are shown in Annexure 5.3.

5.6 Factor Method

This method is based on slope deflection method. Stiffness values of various members are required before proceeding with the analysis.

Horizontal load analysis of a multistoreyed frame by this method is carried out in the following six steps:—

Step 1 : For each joint, compute the girder factor "g" by the relation $g = \frac{\Sigma kc}{\Sigma K}$ where Σ kc denotes the sum of the k values for the columns meeting at that joint and Σ k denotes the sum of the k-values for all the members at that joint. write each value of 'g' thus obtained at the near end of each girder meeting at the joint where it is compu'ed.

Step 2 : For each joint compute the column factor 'c' by the relation c = 1—g, where "g" is the girder factor for that joint as computed in step 1. Write each values of "c" thus obtained at the near end of each column meeting at the joint where it is computed. For the fixed column bases of the first storey, take c = 1.

Step 3 : From steps 1 and 2, there is a number at each end of each member of the bent. To each of these numbers, add half of the number at the other end of the member.

Step 4 : Multiply each sum obtained from step 3 by the K-value for the member in which the sum occurs. For columns, call this product the column-moment factor C; for girders, call this product the girder-moment factor G.

Step 5 : The column moment factors C from Step 4 are actually the approximate relative values for column end moments for the storey in which they occurs. The sum of the column end moment in a given storey, by statics, is equal to the total horizontal shear on that storey multiplied by the storey height. Hence, the column moments may be obtained by distributing the storey moment in proportion to the column moment factors.

Step 6 : The girder moment factors G from Step 4 are actually approximate relative values for girder end moments for each joint. The sum of the girder end moments at each joint is equal, by statics, to the sum of the column end moments at that joint, Hence, the girder moments may be obtained by distributing the sum of the column end moments in proportion to their girder moment factors.

Sample calculations :

The steps to be followed for application of this method for analysis of frame shown in Fig. 5.1 are given below :—

$$\text{Girder factor "g"} = \frac{\Sigma kc}{\Sigma K} = \frac{204.0+325.5}{204.0+325.5+83.5}$$

$$\frac{g}{\Sigma k} = 0.864$$

This value 0.864 is written at the near end of girder 36—37 meeting at the joint 36 (Fig. 5.6a).

At joint 33

$$\text{Girder factor "g"} = \frac{86+86}{83.5+86.0+33.7+86} = 0.595$$

This value 0.595 is written, at the near end of the girder 33-32 and 33-34 (Annexure 5.4).

Step 2 at joint 36

Column factor "c" = 1—g = 1—0.864 = 0.136

Write this value at the near ends of columns meeting at the joint 36 (Fig. 5.6a).

at joint 33.

Column factor "c" = 1—g = 1—0.595 = 0.405 which is written at the near end of columns meeting at the joint 33. Steps 3 and 4 are self-explanatory and the calculations for these steps are shown in Fig. 5.5 for a portion of the frame.

STEP 3.

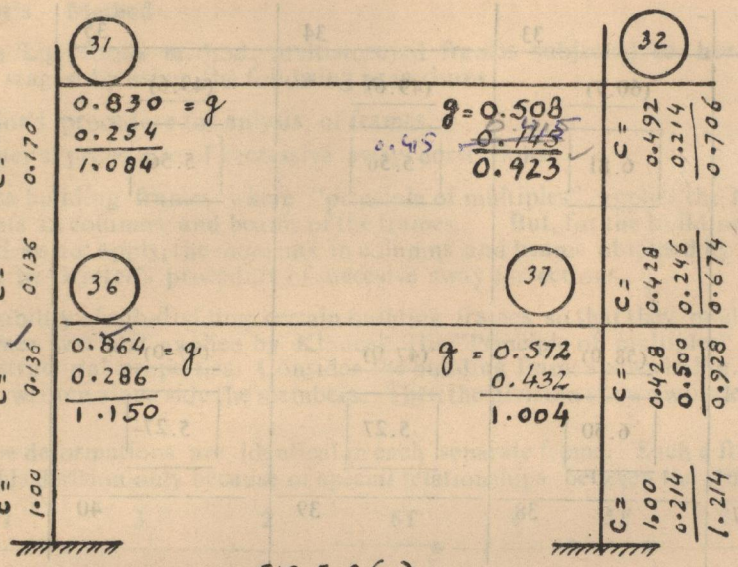


FIG. 5.6 (a)

STEP 4.

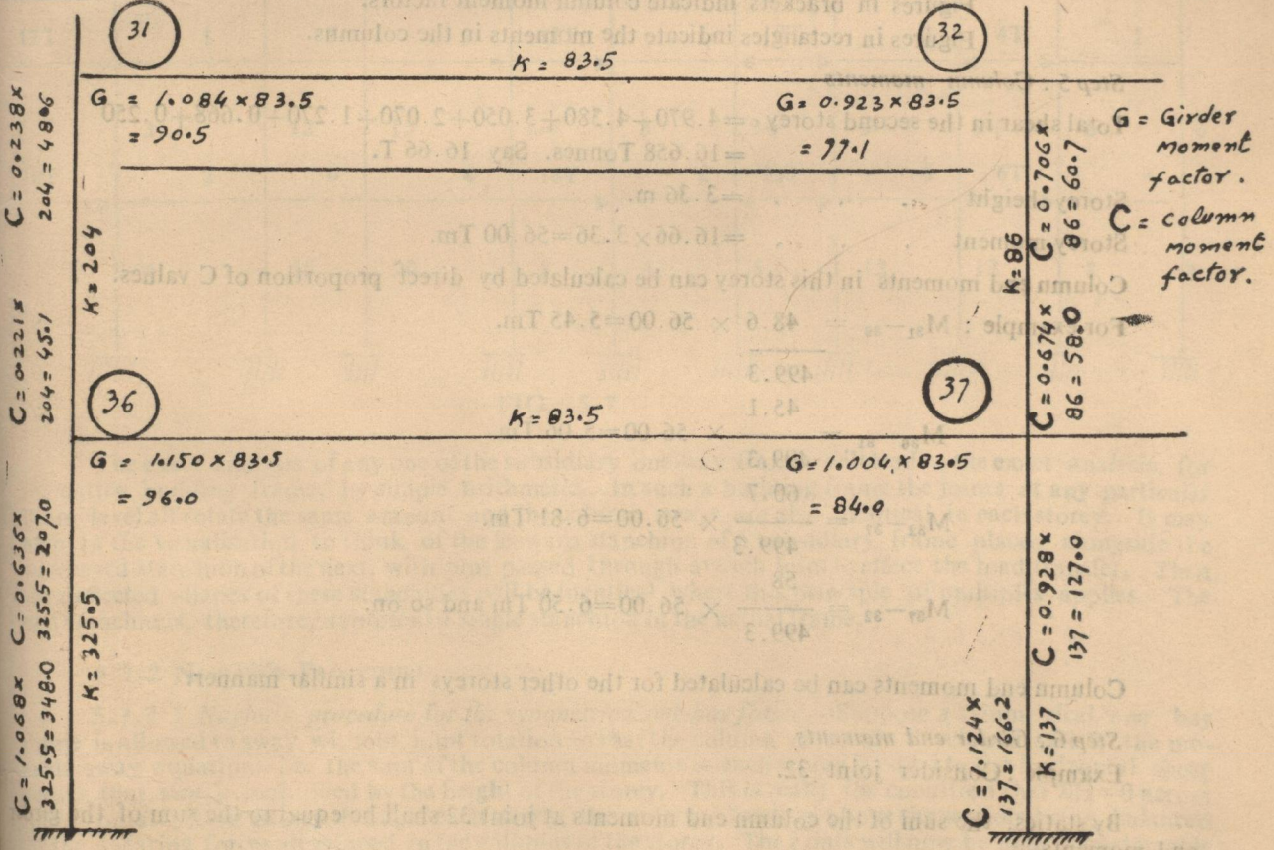


FIG. 5.6 (b)

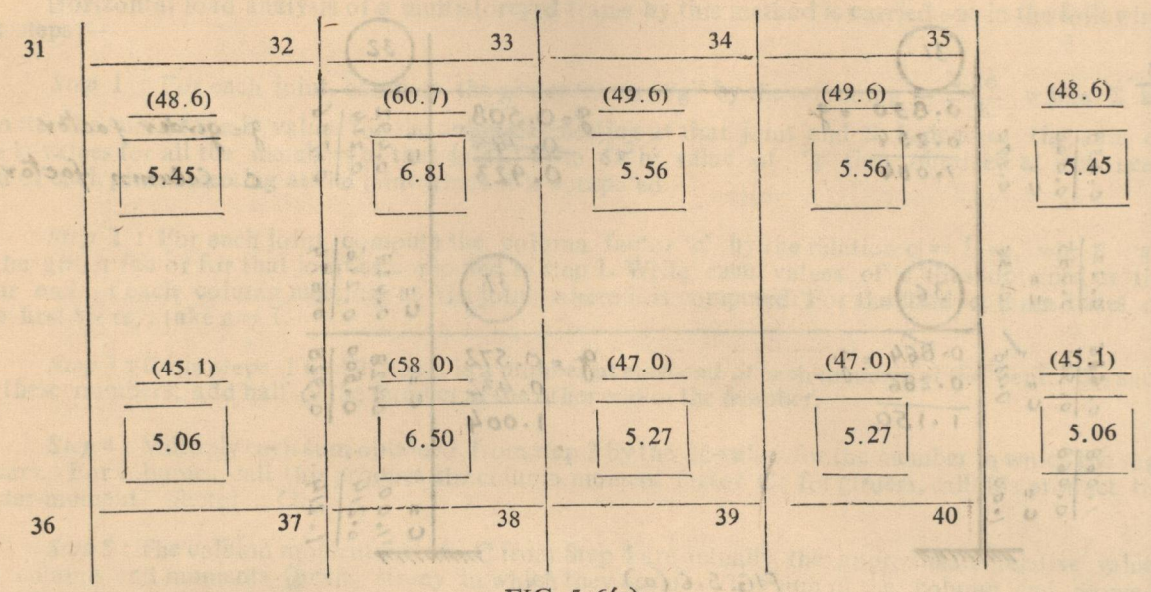


FIG. 5.6(c)

Figures in brackets indicate column moment factors.
Figures in rectangles indicate the moments in the columns.

Step 5 : *Column moments*

Total shear in the second storey $= 4.970+4.380+3.050+2.070+1.270+0.668+0.250$
$= 16.658$ Tonnes. Say 16.66 T.

Storey height . . . $= 3.36$ m.

Storey moment . . . $= 16.66 \times 3.36 = 56.00$ Tm.

Column end moments in this storey can be calculated by direct proportion of C values.

For example : $M_{31-36} = \dfrac{48.6}{499.3} \times 56.00 = 5.45$ Tm.

$$M_{36-31} = \frac{45.1}{499.3} \times 56.00 = 5.06 \text{ Tm.}$$

$$M_{32-37} = \frac{60.7}{499.3} \times 56.00 = 6.81 \text{ Tm.}$$

$$M_{37-32} = \frac{58}{499.3} \times 56.00 = 6.50 \text{ Tm and so on.}$$

Column end moments can be calculated for the other storeys in a similar manner.

Step 6 : *Girder end moments*

Example : Consider joint 32.

By statics, the sum of the column end moments at joint 32 shall be equal to the sum of the girder end moments.

$$M_{32-31} = (6.81+6.44) \times \frac{77.1}{77.1+67.3} = 7.07 \text{ Tm.}$$

$$M_{32-33} = (6.81+6.44) \times \frac{67.3}{77.1+67.3} = 6.18 \text{ Tm.}$$

In a similar manner Girder end moments can be calculated for all other girders of the frame. The results of analysis of frame shown in Fig. 5.1 are given in Annexure 5.4.

5.7 Lightfoot's Method

5.7.1 In Lightfoot's method, multistoreyed frames subjected to horizontal loading are analysised in two stages by using the following procedures.

(1) Naylor's procedure for anlysis of frames.

(2) Grinter's procedure of successive sway corrections.

For those building frames where "principle of multiples" applies the Naylor's procedure gives correct moments in columns and beams of the frames. But, for the building frames where "principle of multiples" does not apply, the moments in columns and beams obtained by Naylor's procedure have to be corrected by Grinter's procedure of succesive sway corrections.

The possibility of sub-dividing certain building frames so that they might be treated as a number of similar frames was first applied by Kloucek. His "Principle of multiples" is an application of super positioning to structural properties. Consider the building frame shown in Fig. 5.7, where the stiffnesses $k\left(\frac{I}{I}\right)$ are written along side the members. Then the division shown will clearly add up to the original frame, since the deformations are identical in each separate frame. Such a frame as that in Fig. 5.7 is amenable to this division only because of special relationships between the stiffnesses of the members.

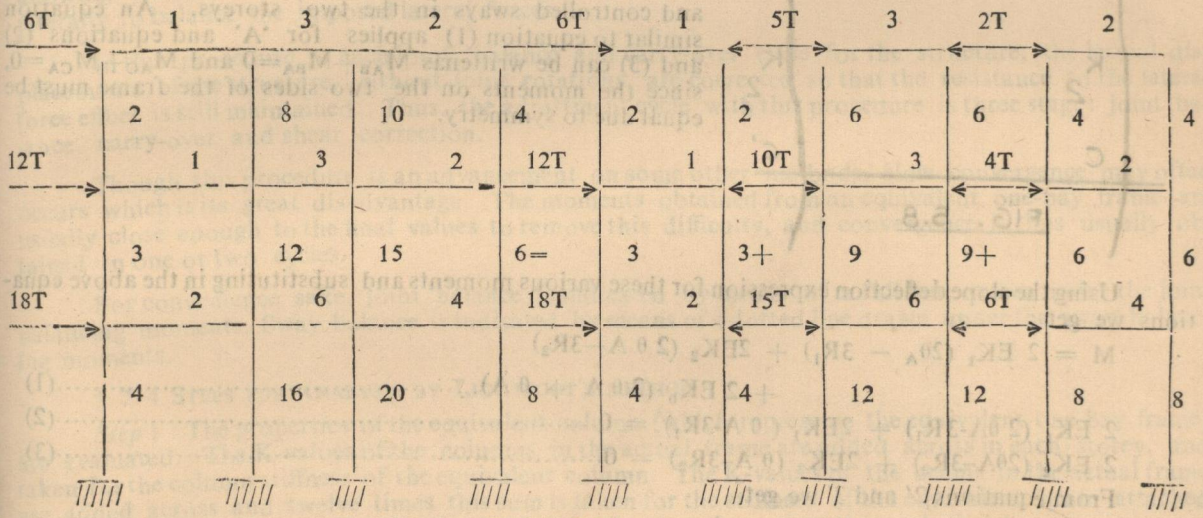


FIG. 5.7

The exact analysis of any one of the subsidiary one-way frames will lead to the exact analysis for the entire building frame, by simple arithmetic. In such a building frame the joints at any particular beam level all rotate the same amount and the column sways are also identical in each storey. It may help in the visualisation to think of the leeward stanchion of a subsidiary frame placed alongside the windward stanchion of the next, with pins passed through at each joint to effect the load transfer. Then the deflected shapes of these stanchions will be identical where this principle of multiples applies. The two stanchions, therefore, represent a single stanchion in the actual frame.

5.7.2 Naylor's Procedure

5.7.2.1 *Naylor's procedure for the symmetrical one-bay frame.*—Suppose a symmetrical one bay frame is allowed to sway without joint rotation so that the column moments developed satisfy the moment sway equations i.e. the sum of the column moments in each storey is equal to the horizontal shear across that storey multiplied by the height of the storey. This is really the condition that $\Sigma H=0$ across a horizontal section drawn through each storey, the external loading above the section being balanced by the shearing forces developed in the columns of the storey. The joints will now be under considerable restraint to prevent rotation. It is proposed to remove this restraint so that the joints can rotate to satisfy moment equilibrium and the structure can sway side-ways to remain in shear equilibrium. Since the structure considered is symmetrical, each column will be deformed in an identical manner, and points of inflection will, therefore, develop at the centre of each beam.

Now consider the effect of releasing joints A and A′ in the frame shown in Fig. 5.8. This release is made to allow the joints to rotate so that they each satisfy the condition $\Sigma M='O$. At the same time the joints A and A′, and the entire frame above and including the joints B and B′ are allowed to move sideways so that shear equilibrium is retained in the two storeys, one above and the other below the beam AA′. The joints at B, B′ and at C, C′ are, however, prevented from rotating.

Let the balancing moments at A and A′ be M. Then if the rotation of A and A′ is θA and if the $\triangle$ values in the storeys above and below AA′ are R_1 and R_2 respectively, then the equilibrium equations are :

The effect of releasing joints A and A′ in the frame.

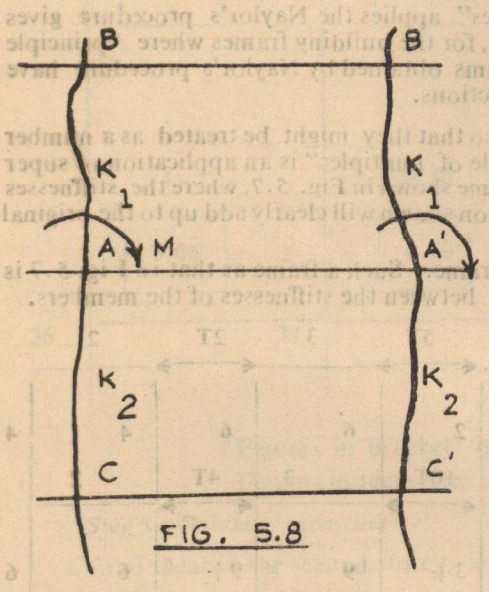


FIG. 5.8

$$M = M_{AB} + M_{AC} + M_{AA'} \text{ (at A)} \quad \ldots\ldots (1)'$$

$$M_{AB} + M_{BA} + M_{A'B'} + M_{B'A'} = 0 \quad \ldots\ldots (2)'$$

$$M_{AC} + M_{CA} + M_{A'C'} + M_{C'A'} = 0 \quad \ldots\ldots (3)'$$

These equations represent the release of the joints A and A′ and controlled sways in the two storeys. An equation similar to equation (1) applies for 'A' and equations (2) and (3) can be writtenas $M_{AB}+M_{BA}=0$ and $M_{AC}+M_{CA}=0$, since the moments on the two sides of the frame must be equal due to symmetry.

Using the slope deflection expression for these various moments and substituting in the above equations we get :

$$M = 2EK_1(2\theta_A - 3R_1) + 2EK_2(2\theta A - 3R_2) + 2EK_b(2\theta A + \theta A) \quad \ldots\ldots (1)$$

$$2EK_1(2\theta A - 3R_1) + 2EK_1(\theta A - 3R_1) = 0 \quad \ldots\ldots (2)$$

$$2EK_2(2\theta A - 3R_2) + 2EK_2(\theta A - 3R_2) = 0 \quad \ldots\ldots (3)$$

From equations 2′ and 3′ we get

$$R_1 = \frac{\theta A}{2} = R_2$$

Substituting for R_1 and R_2 in equation 1^t

$$M = 2EK_1\left(2\theta A - 3\frac{\theta A}{2}\right) + 2EK_2\left(2\theta A - 3\frac{\theta A}{2}\right) + 2EK_b(2A + A)$$

$$= 2EK_1\,\theta A\,\tfrac{1}{2} + 2EK_2.\,\theta A\tfrac{1}{2} + 2EK_b.\,\theta A.3$$

$$= 4EK_1\,\theta A \times \tfrac{1}{4} + 4EK_2 \times \theta A \times \tfrac{1}{4} + 4EK_b\,\theta A \times 3 \times \tfrac{1}{2}$$

It is thus seen that the balancing moment M is divided according to modified stiffness values which are $\frac{1}{4}$ times their original values for the columns and 3/2 times their original values for the beams. It is also observed that $M_{BA} = -M_{AB}$ from the requirement that the additional induced column moments provide no extra shear resistance. The carry-over factor is, therefore (—)1 in the columns. Naylor's method is based on the above conclusion.

In this method for the solution of symmetrical one-bay frames, a special joint balance and carry-over procedure allows for joint rotation and sway at the sametime. To comply with this special procedure, the column stiffnesses need to be multiplied by $\frac{1}{4}$ and the beam stiffnesses by 3/2. The carry-over factor in the columns must be taken as (—)1, but there is no carry-over in the beams.

Alternatively, in the calculations, an equivalent column can be considered as representing the one-bay frame. The column stiffnesses in the one-bay frame are then summed across in each storey to give the column stiffness in that storey of the equivalent column and the beam stiffnesses in the one-bay frame are taken at twelve times the corresponding stiffnesses of the beams. The full loading applies to this equivalent column and the resulting moment values after solution are to be divided among the columns and the beams of the actual one-bay frame. In the distribution table it is usual to omit the calculations for the beam moments since these can easily be determined from the final end moments in the columns.

5.7.2.2 *Naylor's procedure as an approximation for any building frame.*—An equivalent column can be obtained for any building frame by summing the column stiffnesses in each storey and taking 12 times the summed beam stiffnesses at any beam level. This gives the correct equivalent column for those building frames where the "principle of multiples" applies exactly. The end moments in this equivalent column correspond to the deformation actually occurring in the column. If a structural member corresponding to any member in the equivalent column, but with a different stiffness values, is subjected to identical deformations, then its end moments would be in proportion to the relative K-values. This observation leads to an approximate solution for any building frame subjected to side loading at the beam levels, for it is only necessary to take the end moments from the solution of the equivalent column and to divide them among the corresponding member of the actual building frame according to their respective K-values.

The final solution can then be obtained by an application of the procedure of successive sway corrections, which is briefly described below.

5.7.3 GRINTER'S PROCEDURE OF SUCCESSIVE SWAY CORRECTIONS:

This is a development of the original moment distribution in which sway balance is combined with the usual joint balance. The physical concept upon which this procedure depends is that of allowing the structure to take up an initial position without any joint rotations, but already deformed laterally to balance the imposed lateral forces.

In this procedure at each joint, balance and carry-over cycle for the structure, the lateral displacements of the structure, without joint rotations, are corrected so that the resistance to the lateral force effect is still maintained. Thus, the adjustment cycle with this procedure is three stage: joint balance, carry-over and shear correction.

Though this procedure is an advancement on some other methods, slow convergence may often occurs which is its great disadvantage. The moments obtained from an equivalent one-bay frame are usually close enough to the final values to remove this difficulty, and convergence is thus usually obtained in one or two cycles.

For convenience sake, joint balance is indicated by means of a full line drawn under the joint balancing moments. Sway balance is indicated by means of a dotted line drawn under the sway balancing moments.

5.7.4 STEPS FOR ANALYSIS BY LIGHTFOOT'S METHOD :

Step 1 : The properties of the equivalent column (which represents the equivalent one-bay frame) are evaluated. The K-values of the columns in the actual frame are added across in each storey, and taken for the column stiffness of the equivalent column. The K-values of the beams in the actual frame are added across and twelve times this sum is taken for the stiffness of the equivalent beam attached to the equivalent column. The distribution factors for the columns are then calculated.

Step 2 : The F.E.M. in each storey of the equivalent column are found. They are equal at top and bottom in each storey and together equal to product of the shear across the storey and the storey height. These moments are then adjusted according to the "no-shear" conditon. This is usually done in a tabular form using Naylor's procedure.

Step 3 : The column moments obtained by solution of the equivalent column are apportioned according to the respective K-values in the actual frame. The beam moments are similarly apportioned, but the values so calculated from the equivalent column have to be shared equally between the two ends of each beam. The two rules required for this purpose are

(i) Any terminal column moment equals the value found from the equivalent column multiplied by the respective column stiffness and divided by the sum of the column stiffnesses.

(ii) Any terminal beam moment equals the value found from the equivalent column (as shown in table 5.1) multipled by the respective beam stiffness and divided by twice the sum of the beam stiffnesses.

Step 4 : Appotioned moments are then corrected by the procedure of sucessive sway corrections.

5.7.5 EXAMPLE :

The application of Lightfoot's method for analysis of the frame shown in Fig. 5.1 is indicated below :—

Step 1:

K-value for the equivalent column in the top most storey is equal to $204+86+86+86+204=666$. K-value of the equivalent beam attached to the equivalent column at the top-most level is equal to $12\times(83.5+83.5+33.7+83.5)=3410.4$. Similarly, K-values of equivalent columns and beams in other storeys can be worked out.

The distribution factors for the columns (only) can now be calculated. For example distribution factors for columns at point 6 are (refer Fig. 5.9).

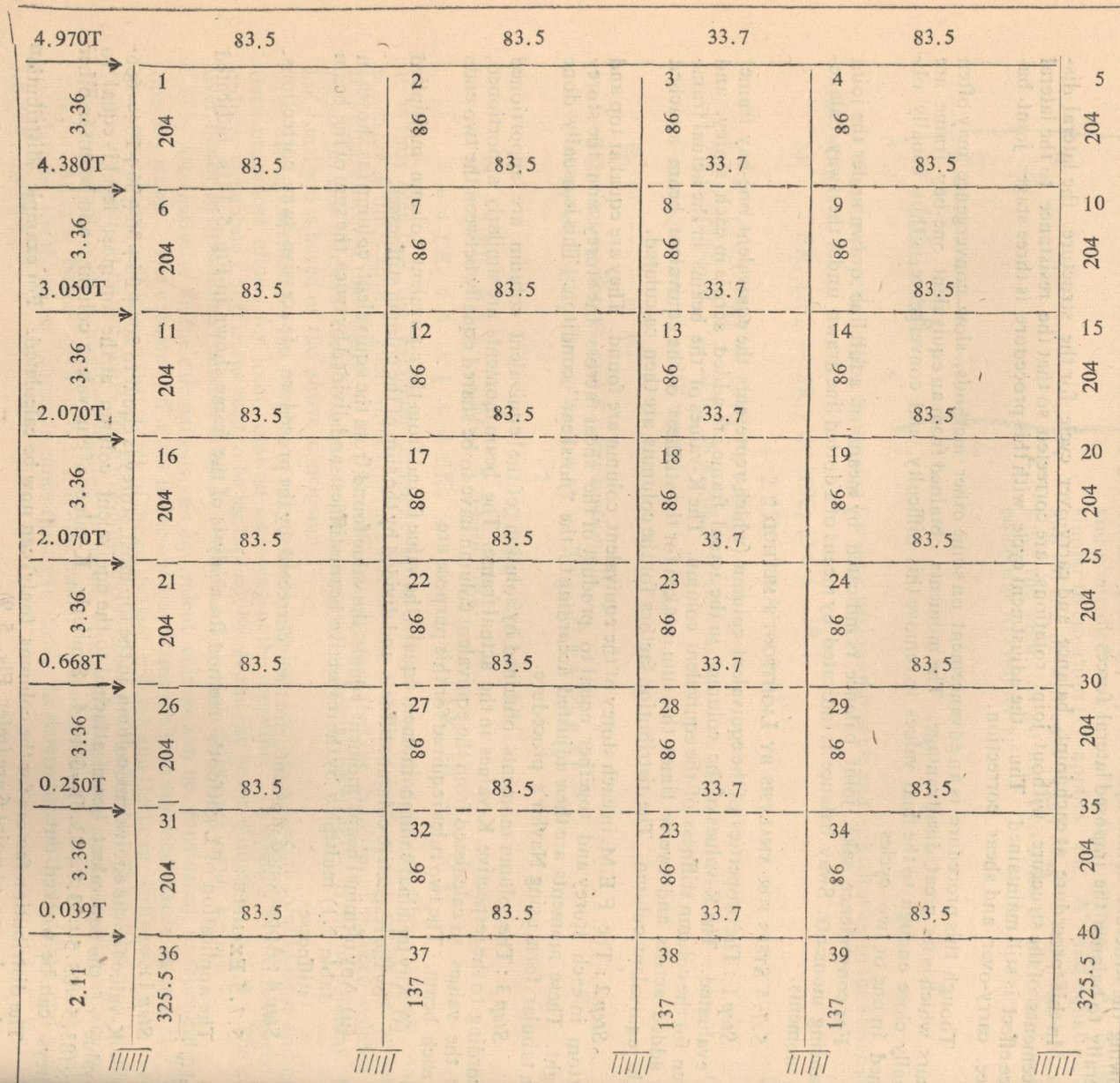
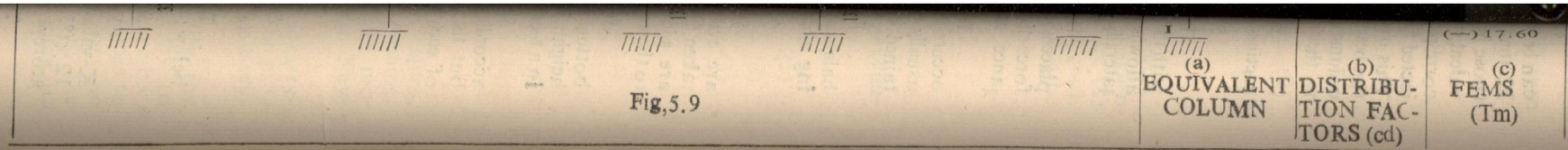


Fig 5.2

Sway Distribution of moments in the equivalent column



Fig,5.9

Sway Distribution of moments in the equivalent column

(Moments are in T. M.)

Joints	I	H		G		F		E		D		C		B		A
Members . . .	IH	HI	HG	GH	GF	FG	FE	EF	ED	DE	DC	CD	CB	BC	BA	AB
Distribution Factors . . .	..	0.2070	0.1296	0.1406	0.1406	0.1406	0.1406	0.1406	0.1406	0.1406	0.1406	0.1406	0.1406	0.1406	0.1406	0.1632
Initial Sway Moments .	(—)17.60	(—)17.60	—28.00	—28.00	—27.60	—27.60	—26.40	—26.40	—24.30	—24.30	—20.80	—20.80	—15.70	—15.70	—8.35	—8.35
Balance . . .	..	+9.45	+5.91	+7.82	+7.82	+7.59	+7.59	+7.13	+7.13	+6.35	+6.35	+5.13	+5.13	+3.38	+3.38	—1.36
Carry Over .	—9.45	..	—7.82	—5.91	—7.59	—7.82	—7.13	—7.59	—6.35	—7.13	—5.13	—6.35	—3.38	—5.13	—1.36	—3.38
Balance . . .	..	+1.62	+1.01	+1.90	+1.90	+2.10	+2.10	+1.96	+1.96	+1.72	+1.72	+1.37	+1.37	+0.91	+0.91	+0.55
Carry Over . . .	—1.62	..	—1.90	—1.01	—2.10	—1.90	—1.96	—2.10	—1.72	—1.96	—1.37	—1.72	—0.91	—1.37	—0.55	0.91
Baiance . . .		+9.39	+0.25	+9.44	+0.44	+0.54	+0.54	+0.54	+0.54	+0.47	+0.47	+0.37	+0.37	+0.27	+0.27	+0.15
Carry Over . . .	—0.39	..	—0.44	—0.25	—0.54	—0.44	—0.54	—0.54	—0.47	—0.54	—0.37	—0.47	—0.27	—0.37	—0.15	0.27
Ba lance . .	..	+0.09	+0.06	+0.11	+0.11	+0.14	+0.54	+0.14	+0.14	+0.13	+0.13	+0.10	+0.10	+0.07	+0.07	+0.04
Carry Over . .	—0.09	..	—0.11	—0.06	—0.14	—0.11	—0.14	—0.14	—0.13	—0.14	—0.10	—0.13	—0.07	—0.10	—0.04	—0.07
Balance . . .	..	+0.02	+0.01	+0.03	+0.03	+0.04	+0.14	+0.04	+0.04	+0.03	+0.03	+0.03	+0.03	+0.02	+0.02	+0.01
Carry Over . .	—0.02	..	—0.03	—0.01	—0.04	—0.03	—0.04	—0.04	—0.03	—0.04	—0.03	—0.03	—0.02	—0.03	—0.01	—0.02
Balance . . .	..	+0.01	+0.00	+0.01	+0.01	+0.01	+0.01	+0.01	+0.1	+0.01	+0.01	+0.01	+0.01	+0.01	+0.01	+0.00
Final Column Moments .	—29.17	—6.02	—31.06	—24.93	—27.70	—27.48	—25.79	—26.99	~~+3.182~~ 23.18	—25.40	~~+19.09~~ —19.09	—22.49	—13.34	—18.04	—5.80	—10.89
		+37.08		~~+52.63~~ ~~+53.27~~		+53.27		+50.17		+44.49		+35.83		+23.48		+10.89

TABLE 5.1

$$\text{for 6-7} \quad \frac{666}{666 + 3410.4 + 666} = 0.1406$$

$$\text{for 6-5} \quad \frac{666}{666 + 3410.4 + 666} = 0.1406$$

Step 2 :

F.E.M. in top-most storey at each end

$$= -4.970 \times \frac{3.36}{2} = -8.25 \text{ tm.}$$

F.E.M. in next storey below top one at each end

$$= -(4.970 + 4.380) \times = \frac{3.36}{2} -15.70 \text{ tm.}$$

After the F.E.Ms. are calculated, the distribution of moments for the equivalent column is carried out and the same is shown in Table 5.1.

Step 3 :

At Level B of the equivalent column

$$M_{6—1} \text{ of actual frame} = -5.80 \times \frac{204}{666} = -1.78 = M$$

$$M_{6—11} \text{ of actual} = -18.04 \times \frac{204}{666} = -5.58 = M$$

$$M_{7—2} = M = M = -5.80 \times \frac{86}{666} = -0.75$$

$$M_{7—12} = M = M = -18.04 \times \frac{86}{666} = 2.33$$

and so on.

$$M_{6—7} = \tfrac{1}{2} (23.84 \times \frac{83.5}{284.2}) = \times 3.50 = M$$

$$M_{7—8} = M_{8—7} = +3.50$$

$$M_{8—9} = M_{9—8} = \tfrac{1}{2}(+23.84 \times \frac{33.7}{284.2}) = +1.41$$

$$M_{9—10} = M_{10—9} = +3.50$$

Similarly the moments in all other member of the actual frames are worked out and the values are shown in Fig 5.10.

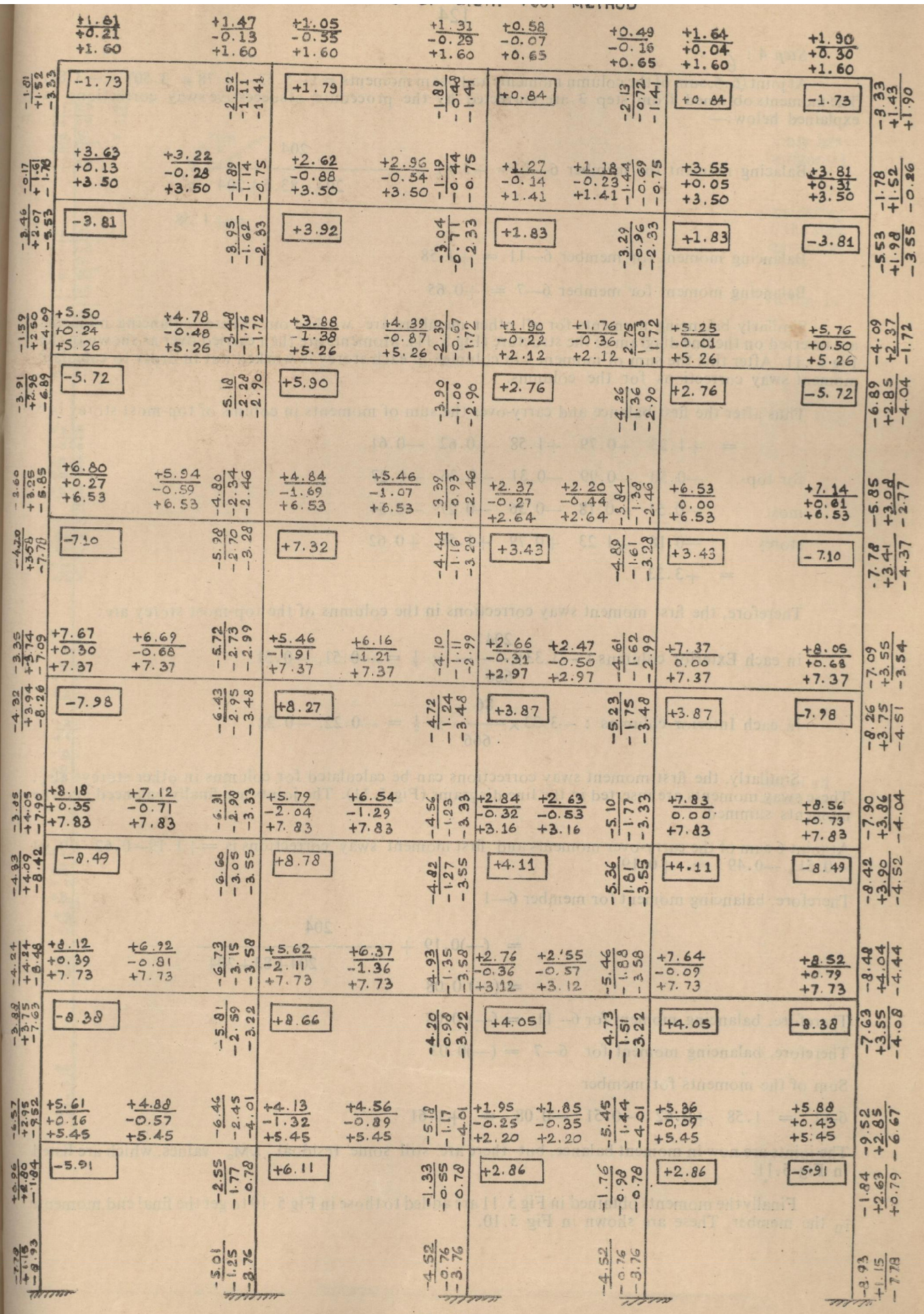


FIG. 5.10

FIGURES IN RECTANGLES ☐ INDICATE UNBALANCED MOMENTS AT JOINTS

Step 4 :

At joint 6, sum of the column moments and beam moments = (—5.53 — 1.78 + 3.50) = —3.81. The moments obtained from step 3 are corrected by the procedure of successive sway corrections as explained below:—

$$\text{Balacing moment for member } 6\text{—}1 = +3.81 \times \frac{204}{204+83.5+204}$$

$$= +1.58$$

Balancing moment for member 6—11 = +1.58

Balancing moment for member 6—7 = +0.65

Similarly balancing moments for all other members are worked out. These balancing moments are inserted on the line diagram of the structure (Fig 5.11). Moments are then carried over as shown in the Fig 5 .11. After this the sum of moment s in columns of each storey is worked out in order to calculate moment sway corrections for the columns.

Thus after the first balance and carry-over the sum of moments in colums of top-most storey is :

=	+1.23	+0.79	+1.58	+0.62	—0.61
for top-	—0.50	—0.99	—0.31	—0.36	—0.27
most	—0.54	—0.18	—0.36	—0.27	—0.54
storey	—0.18	+1.23	+0.79	+1.58	+0.62
=	+3.33				

Therefore, the first moment sway corrections in the columns of the top-most storey are:

$$\text{in each Exterior columns : } -3.33 \times \frac{204}{666} \times \tfrac{1}{2} = -0.51, -0.51$$

$$\text{in each Interior columns : } -3.33 \times \frac{86}{666} \times \tfrac{1}{2} = -0.22, -0.22$$

Similarly, the first moment sway corrections can be calculated for columns in other storeys alsc. These sway moments are inserted in the line diagrams (Fig 5.11). The joints are finally balanced and the moments summed.

At joint 6 sum of the carry-over moments and first moment sway corrections is =+1.19—0.62 +0.62 —0.51 —0.49 = +0.19

Therefore, balancing moment for member 6—1

$$= (-)0.19 + \frac{204}{204+83+204}$$

$$= (-)\ 0.08$$

Therefore, balancing moment for 6—11 = (—) 0.08

Therefore, balancing mement for 6—7 = (—)0.03

Sum of the moments for member

6—1 = 1.58 +0.62 —0.51 —0.08 = +1.61

The joints are now in moment balance, but there are still some residual ΣM_c values, which are listed in Fig 5.11.

Finally the moments obtained in Fig 5.11 are added to those in Fig 5.10 to get the final end moments in the member. These are shown in Fig 5.10.

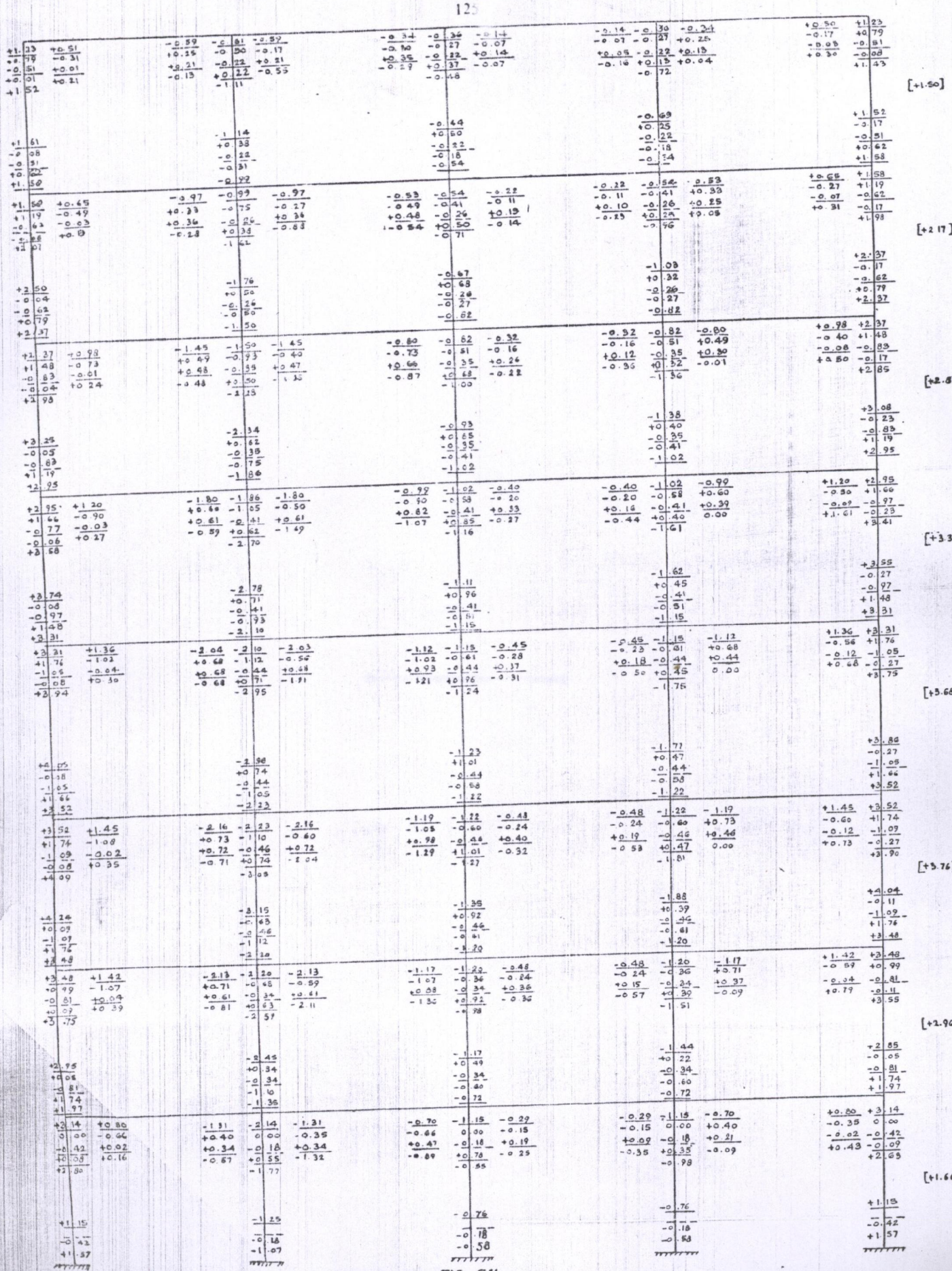


FIG. 5.11

5.8. Kani's Method

5.8.1 Analysis of a multistoreyed frame by Kani's method subjected to vertical loads only has already been explained in the previous chapter. Now another multi-storeyed frame subjected to horizontal loads at the joints as shown in fig 5.1 is considered for analysis.

By taking a horizontal cut through all the columns of some storey 'r', we obtain by considering the condition of equilibrium the sum of the shear forces of all the columns of a storey 'r' equal to the sum of all the horizontal forces H which are acting at joints above the storey 'r' Therefore, storey shear forces for all the storeys can be easily calculated by using this condition of equilibrium.

Again designating the shear force in column i—k of the storey 'r' by Q_{ik} and substituting in the expression for the

shear force $Q_{ik} = \dfrac{M_{ik}+M_{ki}}{h_{ik}}$ the values of M_{ik} and M_{ki} from the

equations

$$M_{ik} = \bar{M}_{ik} + 2M'_{ki} + M'_{ki} + M''_{ki} \text{land}$$

$$M_{ki} = \bar{M}_{ki} + 2M_{ki} + 2M_{ik} + M_{ki}$$

We obtain in case the height 'h' of all columns of the storey is equal

$$\Sigma Q_{ik} = -\frac{1}{h_r}\Sigma_r\,(2M'_{ik} + M'_{ki} + M''_{ik} + 2M'_{ki} + M''_{ik} + M'_{ki})$$

($\bar{M}_{ik}$ = O in case of columns as there are no fixed end moments)

For straight bars with constant cross section we have

$$M''_{ik} = M''_{ki} \text{ on substitution}$$

$$\Sigma Q_{ik} = -\frac{1}{h}\sum_{(r)}\left[3(M'_{ik} + M'_{ik}) + 2M''_{ik}\right]$$

where Q_{ik} is the sum of the shear forces of all colums of the storey 'r', which is equal to the sum of all horizontal forces acting at joints above the storey 'r'. Designating the sum of the restraint forces acting above the storey 'r' as Q_r, we have

$$Q_r = \Sigma Q_{ik}$$

$$Q_r = -\frac{1}{h_r}\,\Sigma_r\left[3(M'_{ik} + M'_{ki}) + 2M''_{ik}\right]$$

where Q_r = Storey shear forces of the storey r,

h_r = Storey height of the storey r,

M'_{ik} = Rotation contribution of the end of column ik

M'_{ki} = Rotation contribution of the end k of column ik and

M''_{ki} = Linear displacement contribution.

Therefore, the sum of the linear displacement contributions of all columns of the storey 'r',

$$\sum_{(r)} M''_{ik} = -3/2\left[\frac{Q_r h_r}{3} + \sum_{(r)}(M'_{ik} + M'_{ki})\right]$$

The quantity $\frac{Q_r h_r}{3}$ that is, a third of the product of the storey shear force and the storey height, shall be designated as the storey moment $\bar{M}_r$

$$\bar{M}_r = \frac{Q_r h_r}{3}$$

The displacement factor s are calculated in the same manner as for vertical loading. The only difference is that in performing the basic operation for determining the linear displacement contributions, we have to include the storey moments $\bar{M}_r$ in the sum of the rotation contributions of all bar ends of the storey considered.

In general, the analysis of a multistoreyed frame with horizontal loading differs from the analysis in case of vertical loading only by the fact that in performing the basic operation for the determination of the displacement contribution, the sum of the rotation contributions of all bar ends of the storey considered must contain also the storey moment $\bar{M}_r$.

Designating the linear displacement factor of the bar ik by Vik the general formula for the determination of the linear-displacement contribution of bar i-k of the storey 'r' is :—

$$M''_{ik} = \upsilon_{ik}\left[\bar{M}_r + \Sigma(M'_{ik} + M'_{ki})\right]$$

The general formula for the determination of the rotation contribution of bar ik at i is :—

$$M'_{ik} = \omega_{ik}\left[\bar{M}_i + \Sigma(M'_{ki} + M''_{ik})\right]$$

where ω_{ik} = rotation factor of the end of i bar ik.

5.8.2. In order to analyse a multistoreyed frame for horizontal loads the following procedure may be adtopted.

1. *Fixed-end state* : All F.E.Ms., restraint moments, and also the storey shear forces and storey moments $\frac{\bar{M}_r = Q_r h_r}{3}$ are calculated. In case the loads are acting at joints only, then only, storey shear forces and storey moments need be calculated. It is convenient to indicate the storey moments in the calculating scheme at mid heights of the corresponding storeys.

2. The rotation and displacement factors are calculated in a similar manner as in the case of vertical loading—(See pages 112 & 117 of Chapter 4).

3. The calculation of the displacement and rotation contributions begins with the basic opertation for the calculation of the linear-displacement contributions. After this basic operation is carried out for all storyes, the application of th e basic opertion follows for the rotaten contribution at all joints. These two basic operations are carried out one after the other until the results reach the desired accuracy.

4. The determination of the end moments from the final contributions is performed in the same manner as in the case of vertical load analysis by Kani's method.

If besides the horizontal loading vertical loading exists simultaneously, no difficulties are encountered. After writing down the storey moments and the restraint moments, the rotation and displacement contributions are determined by means of two basic operations.

5.8.3. EXAMPLE

The frame shown in Fig. 5.1 is analysed by Kani's method in Annexure 5.5.

The calculation of the rotation factors will be shown for joint 7 of the frame. The sum of the K-values of the bars of this joint is :

$$83.5 + 86.0 + 83.5 + 86.0 = 339.0$$

The distribution of $(-\frac{1}{2})$ in proportion to the k-values then yields

$$\omega_{7\text{-}6} = -\tfrac{1}{2} \times \frac{83.5}{339.0} = -0.123 \qquad \omega_{7\,8} = -\tfrac{1}{2} \times \frac{83.5}{339.0} = -0.123$$

$$\omega_{7\text{-}2} = -\tfrac{1}{2} \times \frac{86.0}{339.0} = -0.127 \qquad \omega_{7\,12} = -\tfrac{1}{2} \times \frac{86.0}{339.0} = -0.127$$

There are written at joint 7 as shown in Annexure 5.5. As a verification, the rotation factors of the joint may be added, and again $-\frac{1}{2}$ is obtained.

The displacement factors are written down at the centre of eac h column, to the left of column itself. For example, the distribution of $-\frac{3}{2}$ among the columns of top-most storey yields for each exterior column—0.459 and for each interior column—0.194. Total again works out to $-\frac{3}{2}$

Similarly linear displacement factors for other storeys can be worked out.

In the calculating scheme (Annexure 5.5) all the rotation and the displacement factors are written down.

Now follows the calculation of the storey moments. The storey shear forces are follows:

Top-most storey : 4.970 T

Next below storey : 4.970 + 4.380 = 9.35 T. and so on.

The storey moments

$$M = 4.970 \times \frac{3.36}{3} = 5.57 \text{ Tm.}$$

$$M = 9.350 \times \frac{3.36}{3} = 10.48 \text{ Tm and so on.}$$

After these storey moments are written down in the calculating scheme, always to the left of the storey, it is convenient to start the calculation of the linear-displacement contributions because they yield larger results than the rotation contributions. Since there are as yet no approximate values for the rotation contributions, the sums consist so far only of the corresponding storey-moments. Multiplying by the corresponding displacement factors we obtain the linear-displacement contributions for the columns of the upper most storey as $(-0.459) \times 5.57 = -2.56$ for each exterior column and $(-0.194) \times 5.57 = 1.08$ for each interior column. These numbers are entered in the calculation scheme. In a similar manner, the linear-displacement contributions for all other storeys can be worked out.

Now follows the determination of rotation contributions.

Joint 1 : Rotation contribution 1—2 = —2.56 = (—0.145) = + 0.37

Rotation contribution 1—6 = —2.56 × (—0.355) = + 0.909 Say + 0.91

Similarly the rotation contributions for other joints can be worked out. The linear-displacement contributions and rotation contributions obtained from the first cylce of calculations are shown in Fig. 5.12.

While determining the second set of linear-displacement contributions rotation contributions obtained from the first cycle are also taken into account. Thus linear-displacement contributions for the upper most storey for the second approximation are

For each exterior column = (+5.57+0.91+1.34+0.12+0.31+0.20+0.39+0.21+0.41+0.84+ 1.28) × (−0.459) = 11.58 × (−.459) = −5.31 &

fo each interior column + 11.58 × (−0.194) = −2.25

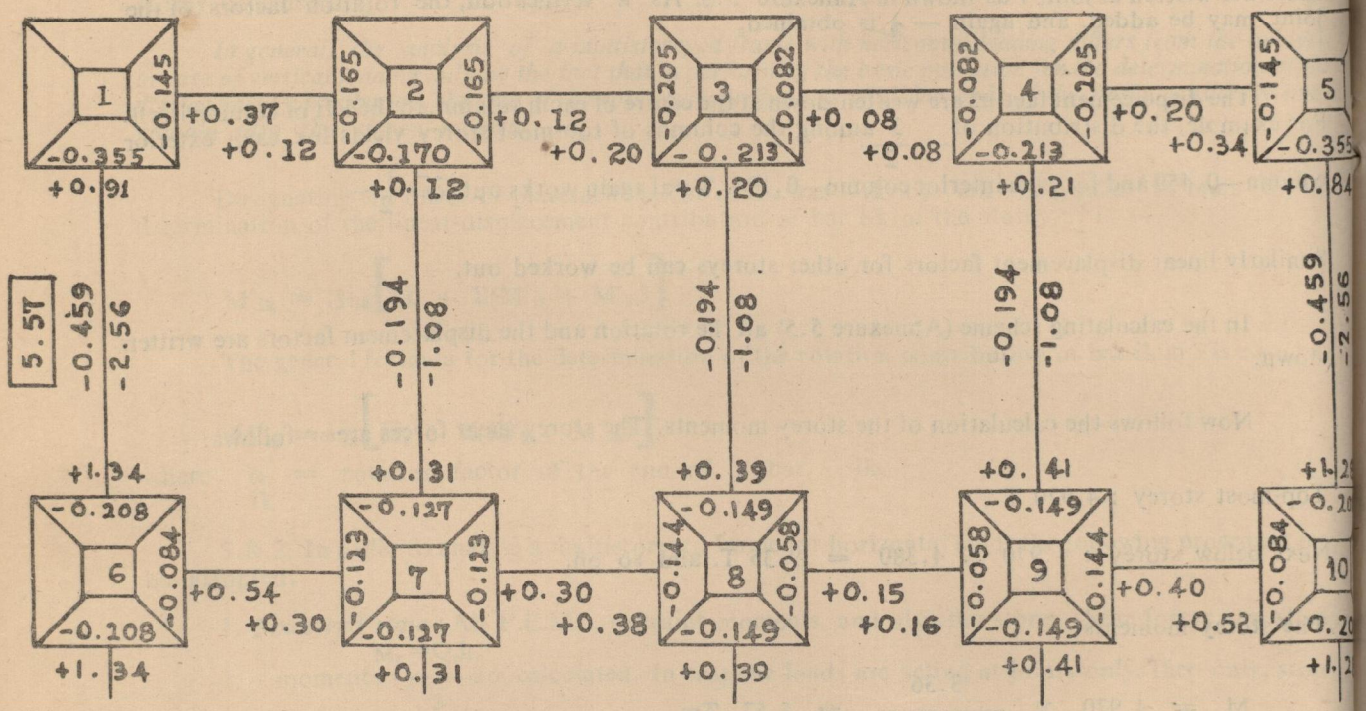


FIG. 5.12

These values are now entered in the calculation scheme.

These two basic operations namely linear-displacement contributions and rotation contributions are carried out one after the other until the results each the desired accuracy.

The end moments can be worked out separately for greater clarity. All the final rotation and displacement contributions are written down in the scheme. Now the final values for end moments are calculated by the formula:

$$M_{ik} = \bar{M}_{ik} + 2M'_{ik} + M'_{ki} + M''_{ik}$$

For example and moment in member 1—2 is (Fig 5.13)

$$= 0.00 + 2 \times 0.84 + 0.32 + 0.00 = +2.00 \text{ Tm}$$

iHowever, for convenience for each bar the sum of the rotation contributions at each end and the displacement contribution is evaluated and written down at each bar end below the rotation contribution already ndicated. The summation yields the final end moments.

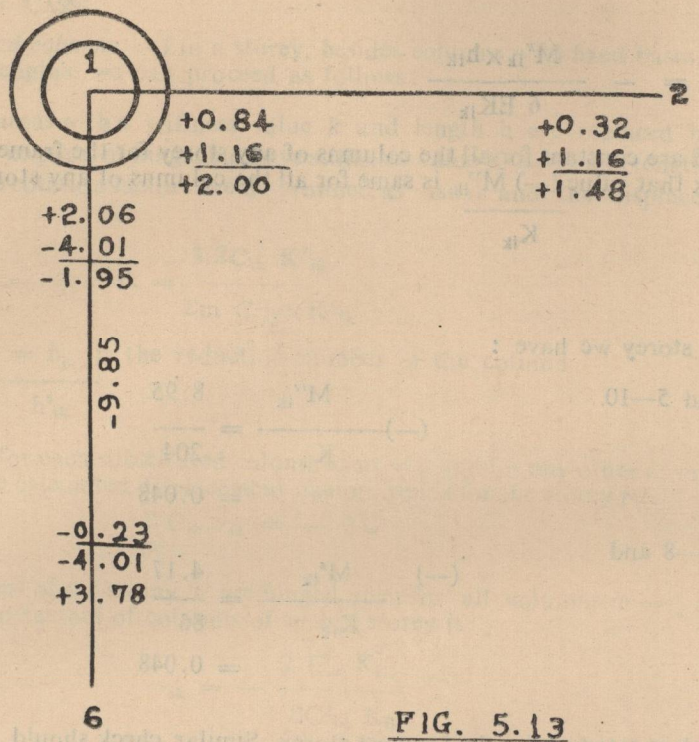
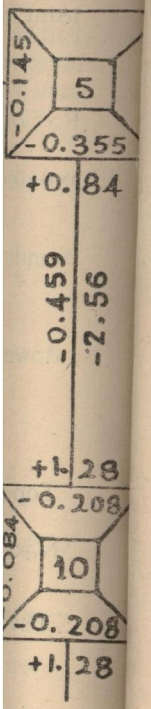


FIG. 5.13



As a rule this summation is carried out in the calculating scheme, in which the calculation of rotation and displacement contributions is performed, but only after all the superfluous numbers, that is, all numbers except the last value of contributions are crossed out.

5.8.4. CHECKS :

The correctness of calculated moments in columns and beams is verified by applying the following three checks:—

(i) Check whether the sum of the final end moments at each joint is zero.

(ii) Check whether in each storey, for a horizontal section through all columns of a storey, the sum of the external horizontal loading, which acts above the section is equal to the sum of all shear forces of the columns is the storey.

(iii) Check whether relative transverse displacement of the column ends is the same for all columns of the same storey.

Check (i) .—At joint 2 the sum of the final end moments of members meeting at the joint is = 1.48 +1.16—2.61=0.03. As this is negligible, it may be presumed that the calculations for the joint 2 are correct. Similar check should be carried out for all other joints.

Check (ii) .—Taking a horizontal section in the topmost storey we get the sum of all the shear forces of the columns in the topmost storey.

$$= -\frac{(-1.95-0.23-2.61-2.04-1.93-1.31-2.20-1.55-2.03-0.30)}{3.36}$$

$$= \frac{16.15}{3.36} = 4.81.$$ As this is approximately equal to the external horizontal load of 4.97 Tonnes (Difference is about 3.2%) acting above the horizontal section in the topmost storey it may be presumed that the check is satisfied. Similar check should be carried out for all other storeys.

Check (iii) :

$$= - \frac{M''_{ik} \times h_{ik}}{6\ EK_{ik}}$$

As h_{ik} and 6E are constant for all the columns of any storey for the frame under consideration it is sufficient to chek that value $\frac{(-)\ M''_{ik}}{K_{ik}}$ is same for all the columns of any storey.

Considering topmost storey we have :

For column 1—6 and 5—10

$$(-)\frac{M''_{ik}}{K_{ik}} = \frac{8.95}{204}$$

$$= 0.048$$

For column 2—7, 3—8 and 4—9

$$(-)\ \frac{M''_{ik}}{K_{ik}} = \frac{4.17}{86}$$

$$= 0.048$$

Hence the check is satisfied for the topmost storey. Similar check should be carried out for all other storeys.

5.8.5. SUMMARY (The Storey has columns of equal lengths)

1. *Storey moments.*—Determine the storey moments by the fomula $\frac{Q_r . h_r}{3}$ and write down at the mid heights of the corresponding storeys on the left side.

2. *Rotation factors.*—These are obtained by distributing the value (—) ½ at each joint among the connecting bar ends in the ratio of the K values. As a check the sum of the rotation factors at any joint must yield (—)½.

3. *Linear displacement factors.*—These are obtained by distributing the value (—)3/2 at each storey among the columns in the ratio of their K values. The displacement factors are written down, always on the left of the middle of columns. The sum of all the displacement factors of the columns of a storey must yield (—) 3/2.

4. *Linear -displacement contributions.*—The calculation of these is performed by repeated application of the basic opation for , the determination of the displacement contributions, as per formula:

$$M''_{ik} = \omega_{ik} [\ \overline{M}_r + \Sigma_r\ (M'_{ik} + M'_{ki})\ ]$$

proceeding from storey to storey in an arbitrary sequence.

5. *Rotation contributions.*—The calculation of these is performed by repeated application of the basic operation for the determination of the rotation contributions as per formula :

$$M'_{ik} = \omega_{ik}\ [\overline{M}_i + \Sigma\ (M'_{ki} + M''_{ik})\ ],$$

proceeding from joint to joint in an arbitrary sequence.

6. *Final end moments.*—Final end moments are calculated by the formula:

$$M_{ik} = \overline{M}_{ik} + 2M'_{ik} + M'_{ki} + M''_{ik}$$

7. *Checks.*— Checks on the accuracy of the operations should be carried out at the end as described earlier.

8.6 SPECIAL CASES

8.6.1. *Hinged columns.*—If in a storey, besides columns with fixed bases, there are also hinged of various lengths, we can proceed as follows:

The hinged columns with a stiffness value k and length h are replaced by substitute columns clamped at the place of hinge, but have stiffness value $K' = \frac{3}{4}k$ and length $h' = \frac{3}{2}h$. with these ed values, the rotation factors are determined as usual and the displacement actors by the n :

$$\gamma ik = \frac{-3/2 C_{ik} \cdot K'_{ik}}{\Sigma m \cdot C^2_{ik} \times K'_{ik}}$$

where $C_{ik} = \frac{h_r}{h'_{ik}}$ is the reduction number of the column

is to be taken for each substituted column as $m = \frac{3}{4}$ and for any other as $m = 1$. The control of putation of the calculated displacement factors yields for the storey r :

$$\sum_{(r)} C_{ik} \cdot \gamma_{ik} = -3/2$$

If all the columns of the storey 'r' are hinged, then for all columns $m = \frac{3}{4}$ and the expression for ar-displacement factors of columns of such a storey is

$$\gamma_{ik} = -\frac{2 \cdot C_{ik} \cdot K'_{ik}}{\sum_{(r)} C^2_{ik} \cdot K_{ik}}$$

e control thereof : $\sum_{(r)} C_{ik} \cdot \gamma_k = -2$

8.6.2. *Columns of unequal heights in the same storey*

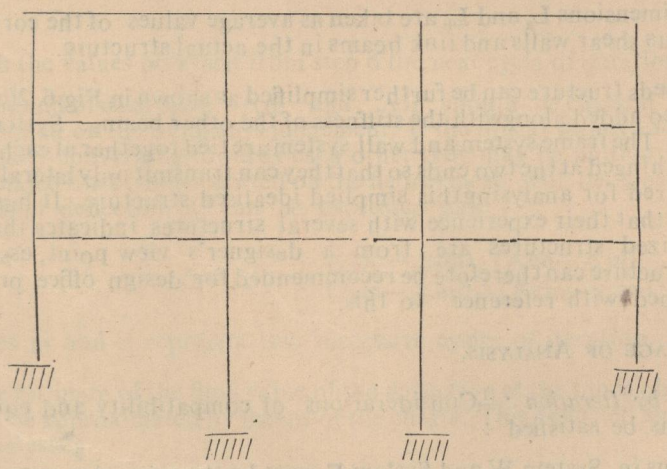


Fig. 5.14

An arbitrary column height is chosen as the stroey height (h_r). it is convenient to assume a heigtht represents the height of most of the coumns in the storey as the storey height.

The calculation of the rotation factor (ω) and their verification is carried out as in the case of ns of equal heights. But for calculating displacement factors reduction factor (C_{ik}) is cal- d for each column i-k of the storey.

$$C_{ik} = \frac{h_r}{h_{ik}}$$

The displacement factors U_{ik} for the columns of the storey are then calculated by :

$$\gamma_{ik} = \frac{-3/2\ C_{ik} \cdot K_{ik}}{\sum_{(r)} C^2_{ik} \cdot K_{ik}}$$

The correctness of these calculated displacement factors is verified by :

$$\sum_{(r)} (C_{ik} \cdot \gamma_{ik} = -3/2$$

The calculation of the rotation contributions (M'_{ik}) remains the same as for storeys with columns of equal heights. However, the displacement contribution (M''_{ik}) is calculated by the following equation.

$$M''_{ik} = \gamma_{ik} [\bar{M}_r + \Sigma C_{ik} (M'_{ik} + M'_{ki})]$$

Final end moments are calculated as in the case of columns of equal heights.

5.9. Method developed by Khan and Sbarounis

5.9.1. In this method, the analysis of the structure is performed in two stages, In the frst stage, the deflected shape and the amount of lateral load resisted by the shear walls and the frames at each storey are determined. For this purpose, the structure is considered to be a combination of two distinct systems as follows :—

(i) *System "W" (Wall system)*—This system consists of an idealised shear wall whose moment of inertia at any storey is equal to the sum of the moments of inertia of all the shear walls in the structure at that level. Coupled shear walls may be represented as a single wall with an equivalent stiffness.

(ii) *System "F" (Frame system)*—This is an idealization of all the frame elements (columns and beams/slabs) in the structure. The stiffnesses of the columns, beams and "link" beams of the idealised structure are obtained by summing of the stiffnesses of the corresponding elements of the actual structure. Since there are two columns in the idealised structure, the stiffness of each is taken as half of the sum obtained above. The idealised structure is shown in Fig. 6.2(b).

In this the dimensions L_a and L_b are taken as average values of the corresponding dimensions of the various shear walls and link beams in the actual structure.

5.9.2The idealizeds tructure can be further simplified as shown in Fig.6.2(c).In this, the stiffnesses of the link beams are also added alongwith the stiffness of the other beams. System F now consists of an idealised one-bay frame. The frame system and wall system are tied together at each floor level by members which are assumed to be hinged at the two ends so that they can transmit only lateral forces. The dimensions L_a and L_b are not required for analysingthis simplied idealised structure. It has been observed by the authors of this method that their experience with several structures indicates that the values computed by either of the idealized structures are, from a designer's view point, essentially the same. The simplifiedid ealized structure can therefore be recommended for design office practice and the method will be further explained with reference to this.

5.9.3 First Stage of Analysis.

5.9.3.1 *Solution by Iteration* :—Considerations of compatibility and equilibrium require that the following conditions be satisfied :

(a) The deflections in System W and System F must be the same at corresponding levels.

(b) The sum of the horizontal shears developed in System F and System W at any storey must be equal to the total external shear of that level.

5.9.3.1.1 The iterative procedure for analysis is as follows :

Step 1: The total external loads are applied to System W and the deflections at each floor level are worked out treating the idealized Wall as a free cantilever. The computations may be carried out by any convenient method e.g. the moment area method. Deflections due to shear and base-rotation can also be taken into account where considered necessary. For the usual proportions of shear walls and with firm foundations these effects can be ignored. The aim of this step is to obtain a first approximation for the deflected shape. In order to obtain quick convergence the deflected shape could be assumed or approximated from Figs. charts. The values of the deflections may be expressed in terms of the modulus of elasticity E. It is not necessary to get the absolute values of the deflections.

Step 2 : The System F is forced to undergo the deflections at each floor as obtained in step 1. Moments induced by this force-fitting will be worked out by moment distribution. In this step, the fixed end moments are worked out for beginning the moment distribution. Since lateral movements only are involved, the fixed end moments for the beams will be zero. The fixed end moment for columns of the i^{th} storey is given by

$$FMci = \frac{\sigma\, EIci\ (\triangle i - \triangle i-1)}{hi^2}$$

$$= \frac{\sigma Ici}{hci}\left[\frac{E(\triangle i - \triangle i-1)}{hi}\right]$$

Step 3 : Moment distribution is carried out in System F. Since a fixed sideway has been imposed on the frame, there is no need to do side way correction and the solution will converge rapidly.

Step 4 : After force-fitting System F to System W as above, the shears in each storey of system F are computed.

Step 5 : At each storey, the total external shear minus the shear in system F with proper algebraic sign is applied to the system W and the deflections of System W are again worked out at each floor level. This is the end of one cycle of iteration. For stable condition, the assumed initial deflections at any floor "i" at the beginning of the n^{th} cycle $\triangle ii(n)$ must be the same as the end deflections $\triangle ei(n)$ at the completion of the n^{th} cycle of iteration.

Step 6 : The initial value for the next cycle is not taken directly from the end value of the previous cycle as it may at times load to divergence of the solution. A convergence correction is applied as follows. If the initial value at the floor "i" for the n^{th} cycle is $\triangle ii(n)$ and the end value is $\triangle ei(n)$ the initial trial values for the $(n+1)^{th}$ cycle should be

$$\triangle ii\ (n+1) = \triangle ii\ (n)\ \frac{\triangle ei(n) - \triangle ii(n)}{1+\left[\frac{\triangle fi - \triangle ei(n)}{\triangle ii(n)}\right]}$$

in which $\triangle fi$ is the free deflection of the wall at floor "i" assuming the entire lateral force to be acting on the wall.

Step 7: With the values obtained from step 6 the next cycle of iteration is done.

Step 8 : At the end of each cycle $\triangle ei$ and $\triangle ii$ should be checked until convergence is within a specific tolerance based on the designer's judgement. If the initial values and final values of the top deflections are gradually converging towards each other they may be plotted as shown in Fig. 6.3 . The point where the extended lines (shown dotted on the figure) meet will lie close to the correct solution. This estimate of final deflection $\triangle'$ can be expressed as

$$\triangle = \triangle in + \frac{(\triangle in - \triangle im)\ (\triangle en - \triangle in)}{(\triangle in - \triangle im) + (\triangle em - \triangle en)}$$

in which the suffixes m and n represent two successive cycles of iteration.

Once a close estimate of the final value of the deflection at the top is obtained, the deflected shape of the structure can be approximated by means of the charts. This deflected shape can then be used in the second stage of analysis.

Step 9 : The slopes are calculated at every floor level for system W. It may be noted that the values may be expressed in terms of E0s in the case of deflections.

3.2. *Use of Charts :* The charts given in Figs 6.4 to 6.9 are directly applicable to structures having uniform sections of members throughout and subjected to a parabolic loading with zero intensity at base and maximum at the top. These charts based on the simplified idealised structure have been taken from the paper by Chandra and Jain. Charts for other types of loading such as a point load at the top, uniformly distributed load, triangular load, and for base rotation have been given in the paper by Khan and Sbarounis. They have also given charts for certain types of variations in the sections of the columns, shear walls and beams. When the type of loading and the sectional properties of the members are almost the same as assumed in the charts, the values obtained from the charts may be used directly without any iteration. However, for structures in which the shear wall stiffness is uniform throughout, the charts under-estimate the frame shear in the topmost storey.

In deriving the curves, System F and W were made competible at ten points. When the number of storeys in a structure is other than 10, the wall-column stiffness ratio used to enter the charts should be computed by

$$\frac{S_s}{S_c} = \left[\frac{\Sigma(EI)s}{\Sigma(LI)c}\right]\left(\frac{10}{N}\right)^2$$

in which N is the number of storeys in the structure. For obtaining the ratio S_c/S_b, the quantities S_c and S_b should be taken simply as the sum of the stiffnesses of the columns and beams respectivelv, in the direction under consideration.

5.9.4 Second stage of Analysis

5.9.4.1 The final deflected shape of the structre as obtained from the first stage of analysis is used in the second stage for determining the moments and shears in every member of the structure. Each frame is force fitted to match the final deflected shape of the structure and the final moments are obtained by moment distribution. For independent frames, the procedure is the same as mentioned in steps 2 and 3 of the first stage of analysis. The number of bays in the frame and the stiffness properties to be consider will be those of the particular frame being considered.

5.9.4.2 In the case of frames which are connected to the shear walls the effect of the deformation of the link beams should also be considered in the moment distribtion. At the point of connection with the shear wall, each link beam undergoes a rotation θi which is equal to the slope of the shear wall at that level, and a vertical displacement $\triangle ui$ which is equal to θi multiplied by the distance Ls from the neutral axis of the wall to the edge where the beam is connected to it. (See Fig. 6.10.). The fixed end moments at the two ends of the link beam of the i^{th} floor are given by the following equations :

At the end connected to the wall :

$$FMbin = \left(\frac{4EIbi}{Lb}\right)\theta i + \left(\frac{6EIbi}{Lb^2}\right) \triangle vi$$

$$= \frac{2Ibi}{Lb}\left[2 + \frac{3Ls}{Lb}\right]E\theta i$$

At the frame end of the link beem :

$$FMbif = \left(\frac{2EIbi}{Lb}\right)\theta i + \left(\frac{6EIbi}{Lb^2}\right)\triangle bi$$

$$= \frac{2Ibi}{Lb}\left[1 + \frac{3Ls}{Lb}\right]E\theta i$$

The fixed end moments for the columns are obtained in the same way as in step 2 of the first stage of analysis. The number of cloumns will be as in the actual frame and may be different from that shown in Fig. 6.10. The final moments are then obtained by moment distribution.

5.9.4.3 If the final deflected shape was arrived at by the process of iteration the forces acting on system W in the last cycle may be distributed to the various shear walls in proportion to their moments of in ertia. This is possible only if relative stiffnesses of different shear walls ramain the same at all the floors. In the cases when the charts are directly applicable the forces may also be obtained from the charts and distributed to the different walls as above. The bending moments can then be worked out. In those cases when the above procedure is not applicable, the bending moment of any shear wall must be calculated by using the deflected shape of the structure. The bending moment at floor i is given by

$$Mi = \left[\frac{EIsi}{hi^2}\right](\triangle i+1-2\triangle i+\triangle i-1)$$

in which Is is the moment of inertia of the wall at floor i, $\triangle i$ the deflection at floor i, $\triangle i+1$ the deflection at floor i+1, and, $\triangle i-1$ is the deflection at floor i−1. If the storey height above or below floor i, is other than hi it is necessary to obtain the deflection at a distance equal to hi After getting the moments the shears are easily computed.

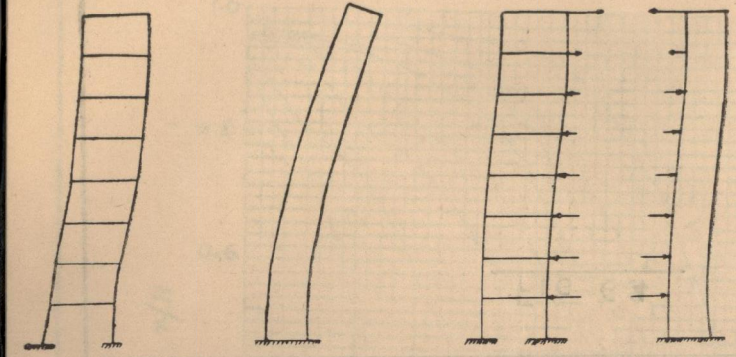
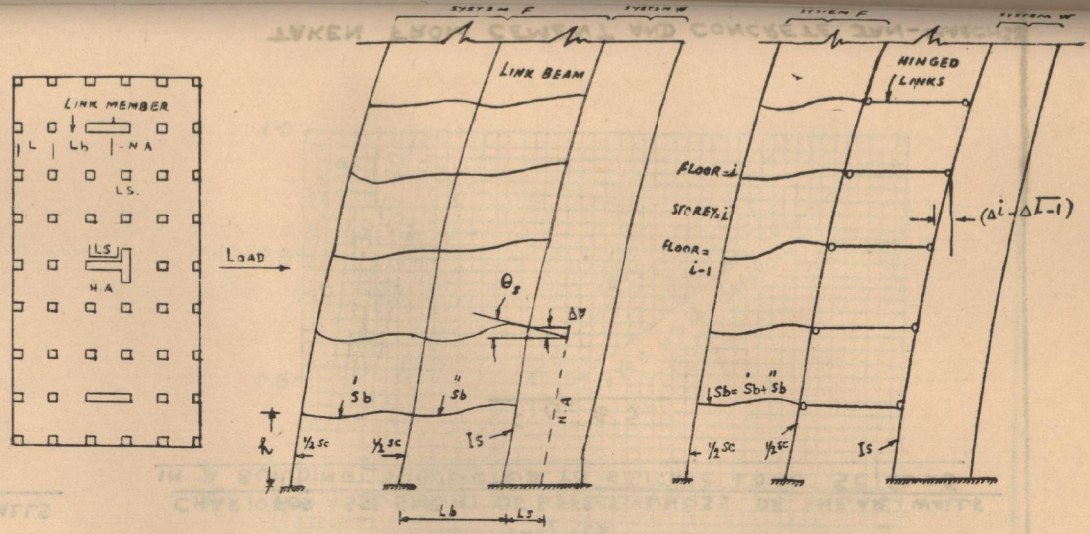


FIG 6.1



FLOOR PLAN (a)

IDEALIZED STRUCTURE (b)

SIMPLIFIED IDEALIZED STRUCTURE (c)

FIG. 6.2

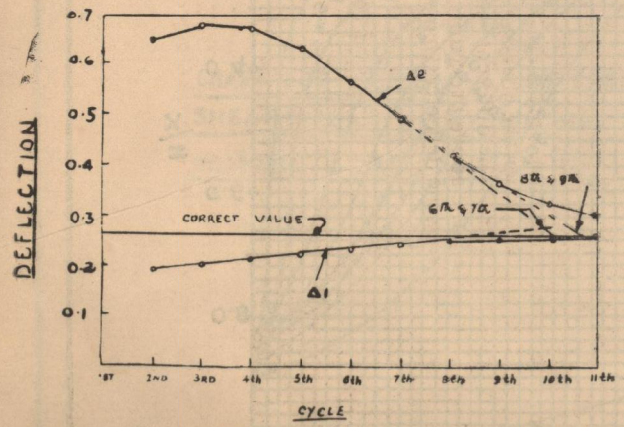
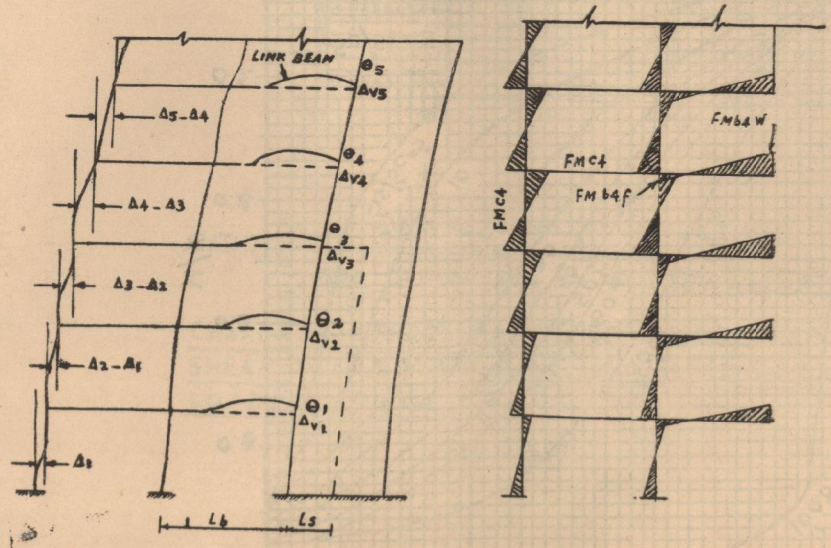


FIG. 6.3



"FORCE-FITTING" OF FRAMES CONNECTED TO SHEAR WALLS (a)

FIG. 6.10

FIXED E.ID MOMENTS FROM FORCE-FITTING (b)

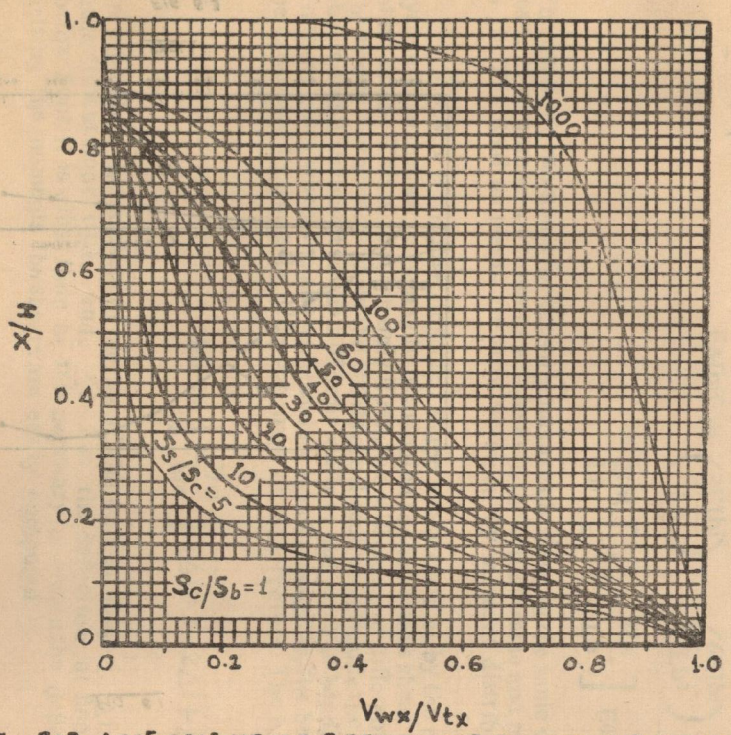


CHART FOR ASSESMENT OF EFECTIVENESS OF SHEAR WALLS IN A BUILDING SUBJECTED TO SEISMIC LOAD $Sc/Sb=1$

FIG. 6.4

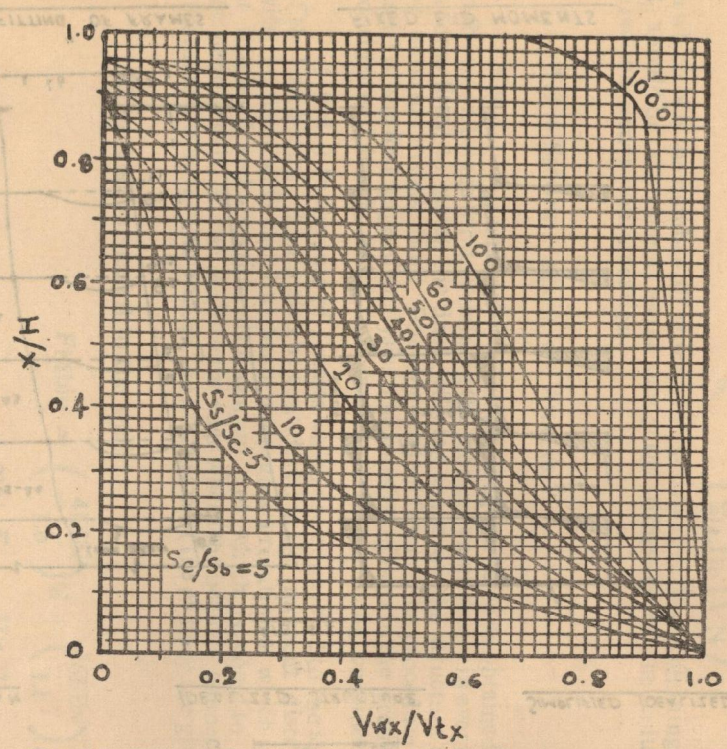


CHART FOR ASSESMENT OF EFFECTIVENESS OF SHEAR WALLS IN A BUILDING SUBJECTED TO SEISMIC LOAD $Sc/Sb=5$

FIG. 6.5

TAKEN FROM CEMENT AND CONCRETE JAN-MARCH'71

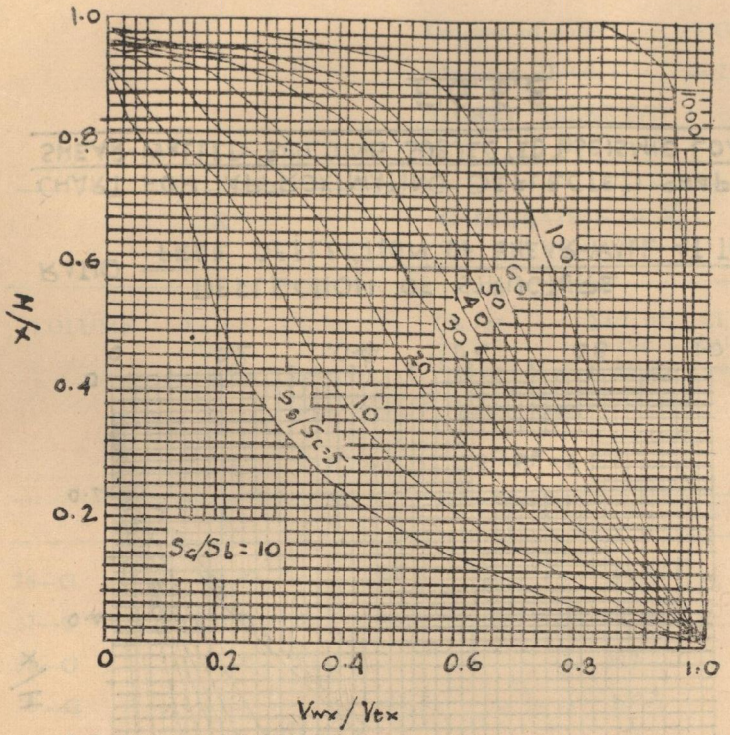
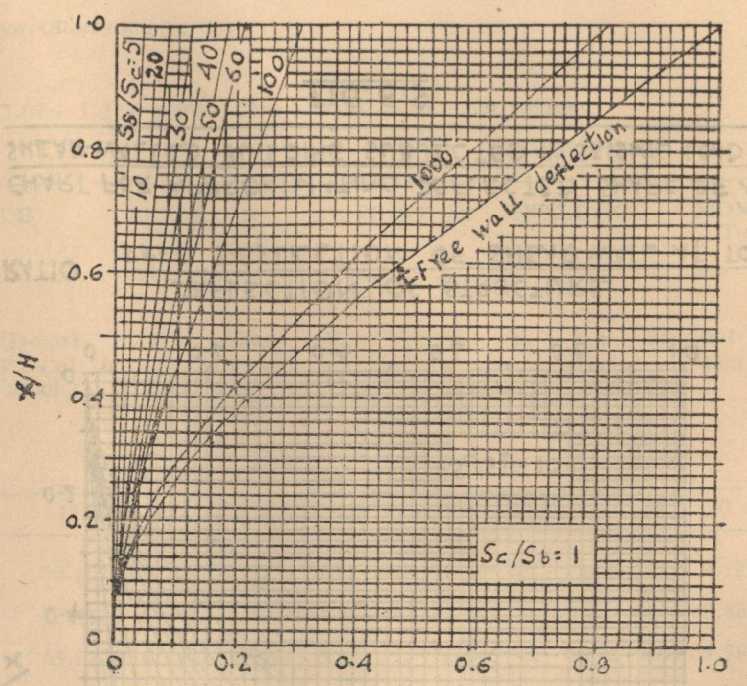


CHART FOR ASSESSMENT OF EFFECTIVENESS OF SHEARWALLS IN A BUILDING SUBJECTED TO SEISMIC LOAD, $S_c/S_b = 10$.

FIG. 6.6

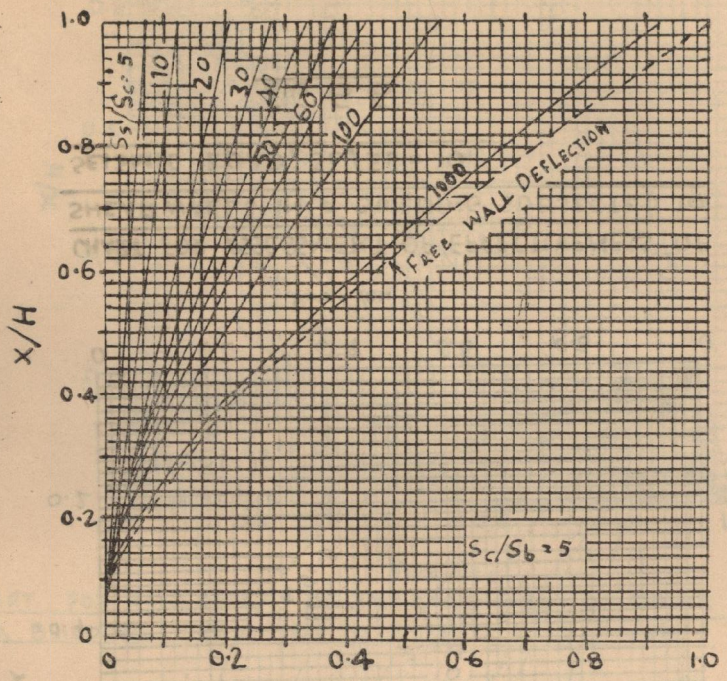


RATIO $\frac{\text{DEFLECTION OF STRUCTURE}}{\text{FREE DEFLECTION OF SHEARWALL AT TOP}}$

CHART FOR APPROXIMATING DEFLECTED SHAPE OF A SHEAR WALLED BUILDING SUBJECTED TO SEISMIC LOAD, $S_c/S_b = 1$

FIG. 6.7

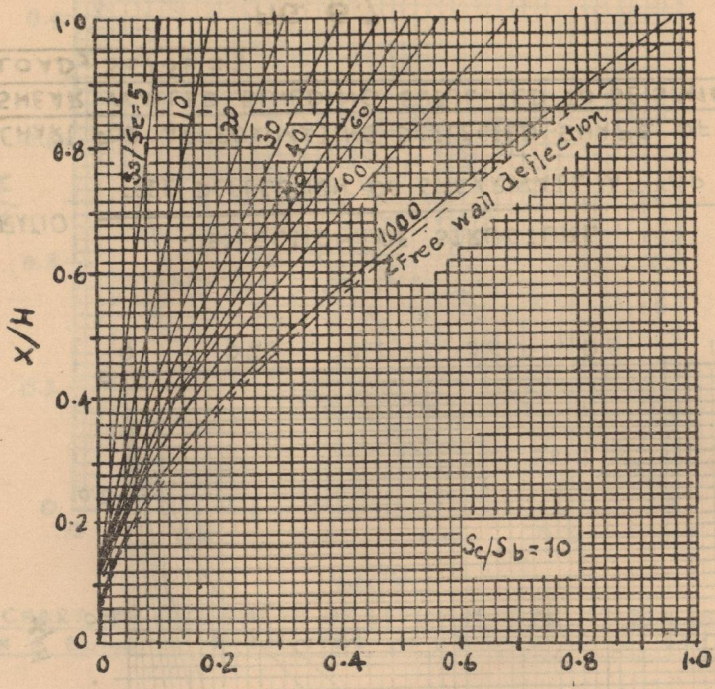
(TAKEN FROM CEMENT & CON. JAN–MAR. 70)



RATIO $\frac{\text{DEFLECTION OF STRUCTURE}}{\text{FREE DEFLECTION OF SHEAR-WALL AT TOP}}$

CHART FOR APPROXIMATING DEFLECTED SHAPE OF A SHEAR-WALLED BUILDING SUBJECTED TO SEISMIC LOAD, $S_c/S_b = 5$

FIG. 6.8



RATIO $\frac{\text{DEFLECTION OF STRUCTURE}}{\text{FREE DEFLECTION OF SHEAR-WALL AT TOP}}$

CHART FOR APPROXIMATING DEFLECTED SHAPE OF A SHEAR-WALLED BUILDING SUBJECTED TO SEISMIC LOAD $S_c/S_b = 10$

FIG. 6.9

Location of storey — Ground floor to first floor. (Storey number 1).

STOREY HEIGHT = 2.11 metres.

STOREY SHEAR (Vt) : (4.97 + 4.38 + 3.05 + 2.07 + 1.27 + 0.668 + .250 + .038) = 16.6960 Tonnes

DIRECTION :

	K	CALCULATION OF $\bar{K}$							D-VALUE		SHEAR IN COLUMNS		POINT OF INFLECTION						MOMENT T-m.	
COLUMN	I/L of each column	I/L for top beams		I/L for botton beams		ΣI/L of top beams	ΣI/L of bottom beams	$\frac{\bar{K}}{2+\bar{k}}$ [Col.(7) + Col. (8)] / Col. (2)	$a=\frac{0.5+\bar{k}}{2+\bar{k}}$	D=axK $\bar{K}$=Col. 10 ×Col.2	$\frac{D}{\Sigma D}$	Shear $\frac{Vt \times D}{\Sigma D}$	Y_0	Y_1	Y_2	Y_3	Y= Y_0+ Y_1+ Y_2+ Y_3	Yh	Bot-tom	Top
		Left	Right	Left	Right															
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
36—G . .	325.5	...	83.5	..	..	83.5	...	0.2565	0.3352	109.1076	0.2589	4.3225	0.91	..	0.090	..	1.000	2.11	9.12	..
37—G . .	137.0	83.5	83.5	..	..	167.0	..	1.2189	0.5340	73.1580	0.1736	2.8984	0.61	..	0.025	..	0.635	1.34	3.88	2.23
38—G . .	137.0	83.5	33.7	..	..	117.2	..	0.8554	0.4746	65.0202	0.1542	2.5745	0.65	..	0.045	..	0.695	1.47	3.78	1.65
39—G . .	137.0	33.7	83.5	..	..	117.2	..	0.8554	0.4746	65.0202	0.1542	2.5745	0.65	..	0.045	..	0.695	1.47	3.78	1.65
40—G . .	325.5	83.5	..	..	..	83.5	..	0.2565	0.3352	109.1076	0.2589	4.3225	0.91	..	0.090	..	1.000	2.11	9.12	..
										ΣD 421.4136										

NOTE:—Analysis for Moments in all the columns in any particular storey of a building is done in a single chart considering the 'D' values of all the columns instead of repeating the analysis for each frame.

Location of storey : First floor to second floor (Storey number 2)

STOREY HEIGHT : : 3.36 metres

STOREY SHEAR : (Vt) = (16.696 —.038) = 16.6580 Tonnes

COLUMN	K			CALCULATION OF $\bar{K}$						D—VALUE	SHEAR IN COLUMNS	POINT OF INFLECTION						MOMENT Tm	
	I/L of each column	I/L for top beams		I/L for bottom beams		ΣI/L of top beams	ΣI/L of bottom beams	$\frac{\bar{K}}{2\times Col(2)}$ [Col.(7) Col.8]	$a=\frac{\bar{K}}{2+\bar{K}}$	D-axk =Col (10) ×Col(2)	D	$\frac{\text{Shear}=V_t\times D}{D}$	Y_0	Y_1	Y_2	Y_3	$Y=Y_0+Y_1+Y_2+Y_3$	Yh	Bottom Top
		Left	Right	Left	Right														
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20 21
31—36 . .	204.0	..	83.5	..	83.5	83.5	83.5	0.4093	0.1698	34.6392	0.1910	3.1816	0.53	..	0	0	0.53	1.780	5.67 5.03
32—37 . .	86.0	83.5	83.5	83.5	83.5	167.0	167.0	1.9418	0.4926	42.3636	0.2336	3.8913	0.50	..	0	0	0.50	1.680	6.54 6.54
33—38 . .	86.0	83.5	33.7	83.5	33.7	117.2	117.2	1.3627	0.4052	34.8472	0.1921	3.2000	0.50	..	0	0	0.50	1.680	5.38 5.38
34—39 . .	86.0	33.7	83.5	33.7	83.5	117.2	117.2	1.3627	0.4052	34.8472	0.1921	3.2000	0.50	..	0	0	0.50	1.680	5.38 5·38
35—40 . .	204.0	83.5	..	83.5	..	83.5	83.5	0.4093	0.1698	34.6392	0.1910	3.1816	0.53	..	0	0	0.53	1.848	5.67 5.03
										ΣD =181.3364									

Location of Storey — Seventh floor to terrace (storey number 8).

Location of Storey — Seventh floor to terrace (storey number 8).

STOREY HEIGHT = 3.36 metres.

STOREY SHEAR (Vt) = 4.97 Tonnes.

COLUMN	K	CALCULATION OF K						D-VALUE			SHEAR IN COLUMN	POINT OF INFLECTION						MOMENT Tm	
	I/L of each Column	I/I for top beams		I/L for bottom beams		Σ I/L of top beams	Σ I/L of bottom beams	$\frac{\overline{K}}{2\times Col(2)}$ [Col. (7)=+ Col(8)	$a=\frac{\overline{K}}{2+\overline{K}}$	D=axK =col(10) ×Col(2)	$\frac{D}{\Sigma D}$	Shear =VtxD $\frac{}{\Sigma D}$	Y 0	Y 1	Y 2	Y 3	Y= Y_0+ Y_1+ Y_2+ Y_3	Yh	Bot-tom	Top
		Left	Right	Left	Right															
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
1—6	204.0	..	83.5	..	83.5	83.5	83.5	0.4093	0.1698	34.6392	0.1910	0.9492	0.16	..	..	0	0.16	0.54	0.51	2.67
2—7	86.0	83.5	83.5	83.5	83.5	167.0	167.0	1.9418	0.4926	42.3636	0.2336	1.1609	0.42	..	..	0	0.42	0.41	1.64	2.26
3—8	86.0	83.5	33.7	83.5	33.7	117.2	117.2	1.3627	0.4052	34.8472	0.1921	0.9547	0.38	..	..	0	0.38	1.28	1.22	1.99
4—9	86.0	33.7	83.5	33.7	83.5	117.2	117.2	1.3627	0.4052	34.8472	0.1921	0.9547	0.38	..	..	0	0.38	1.28	1.22	1.99
5—10	204.0	83.5	..	83.5	..	83.5	83.5	0.4093	0.1698	34.6392	0.1910	0.9492	0.16	..	..	0	0.16	0.54	0.51	2.68

ΣD =181.3364

5.10 Method Developed By Muto

5.10.1 The basic principle of this method is to distribute the lateral shear at any storey to the resisting elements of the storey in proporation to the D-values (distribution coefficients for storey shears) of these elements. The D-Value principle was first presented by Tacha Naito in 1922 and further developed and systematised by Kiyoshi Muto.

The D-value of a resisting element (column or wall) at any storey is defined by the amount of shear reactive to the element when the relative horizontal displacement at the storey under consideration (i.e. the difference between the lateral displacements at the top and bottom of the storey) has a unit value.

5.10.2 D-VALUES OF COLUMNS

The value of the shear distribution coefficients can be found by calculating the deflections at every storey for an assumed loading. However this is a laborius procedure. For most practical cases, the formula derived by Muto can be used for calculating column D-values of frames. These are as follows :

(a) Uniform storey height :

The D-value of a column is given by

$$D = a\, K_c \qquad \ldots\ldots(1)$$

where, a = a constant depending on the end conditions of the column and on $\bar{K}$
K_c = stiffness ratio K of the column being considered.

For ease of calculation, a convenient value of K_o is assumed as standard stiffness and relative stiffness ratios K are used in the calculations. Relative stiffness ratio $K = \frac{K}{K_o}$

The value of the constant 'a' for different cases are given below.

Case 1. General : $$a = \frac{\bar{k}}{2+\bar{k}} \qquad \ldots\ldots(2)$$

Case 2. One end fixed $$a = \frac{0.5+\bar{k}}{2+\bar{k}} \qquad \ldots\ldots(3)$$

Case 3. One end pinned $$a = \frac{0.5\cdot\bar{k}}{i+\bar{k}} \qquad \ldots\ldots(4)$$

where $\bar{K}$ = mean value of k of beams relative to column stiffness (see Fig. 6.10). In case 1, where the sum of the beam stiffness at one end, i.e. K_3+K_4 is much larger than that of the other end, the value of 'a' should be taken as not larger than that to be obtained by case 2 as if that end were fixed.

(b) Non —uniform height :

Case 4. A column with height 'h', differing from the standard height, h

$$D' = a'\, k'c$$

where $$a' = a\left[\frac{h}{h'}\right]^2$$

Case 5. A column composed of two short columns of heights h_1 and h_2, which totalled equals the standard height, h. (This is the case of having an intermediate beam between two floors e.g. at mid landing of a staircase).

$$D' = \frac{1}{\frac{1}{D_1}\left(\frac{h_1}{h}\right)^2 + \frac{1}{D_2}\left(\frac{h_2}{h}\right)^2}$$

if $h_1 = h_2$, $D' = \dfrac{4}{\frac{1}{D_1} + \frac{1}{D_2}}$

if $D_1 = D_2$; $D' = D_1 + D_2$

The D-values as indicated in this section are not absolute values. They are in terms of a common unit $\frac{12\ EK_o}{h_n^2}$ for the n-th storey. If the D-values are obtained from deflection calculations, the deflections may be expressed in terms of $\frac{h_n^2}{12EK_o}$

5.10.3 D-VALUE OF SHEAR WALLS

5.10.3.1 For calculating the D-value of shear walls, the deflections of the shear wall at each floor level are worked out as a free vertical cantilever acted on by an assumed distribution of lateral loads. In addition to the bending deformation, the deflection due to shear is also added when its effect is significant. The effect of foundation rotation can also be added when necessary. The total deflection is obtained by adding the three. If the relative lateral displacement between two floors is represented by s, then

$$\delta = \delta_B + \delta_S + \delta_R$$

where the subscripts B,S, and R stand for bending shear and foundation rotation respectively (see Fig. 6.11

If the storey shear as per assumed loads is V, the D-value is obtained by the formula

$$D = \frac{V}{\delta}$$

5.10.3.2 *Bending deformation.*—The shape of the bending moment diagram is triangular or trapazoidal in any storev. Assuming a rectangular diagram of equvialent area, the interfloor deflection at the n-th storey, by the moment area method is given by

$$\delta_{Bn} = \sum_{i=1}^{n-1} \frac{mihi}{EI_i}\ h_n + \tfrac{1}{2}\frac{M_n\ h_n^2}{EI_n}$$

where =mean bending moment at storey *i*

For convenience in calculations a common unit of $\frac{hn^2}{12E\ k_o}$ is adopted. The stiffness $\frac{I}{l}$ of the wall at i-th floor is denoted by Kwi and the relative stiffness ratio $\frac{kwi}{k_o}$ is denoted by Kwi. The bending deformation can then be expressed as

$$\delta_{Bn} = 4\triangle_{Bn}\ \frac{3}{h_n}\ \text{in terms of}\ \frac{hn^2}{12E\ k_o}$$

$$\text{where}\quad \triangle_{Bn} = \sum_{i=1}^{n-1} \frac{Mi}{kwi} + \tfrac{1}{2}\ \frac{M_n}{kwn}$$

For ease in calculating the values of δ_B, the tabular form shown in Fig. 6.12 is adopted. The table is filled up, following the direction indicated by the arrows.

5.10.3.3 *Shearing deformation*:—The inter-floor deflection due to shear at the n-th storey is given by

$$\delta_{sn} = \frac{X.\ Vn\ hn}{G.\ Awn}$$

where, x=coefficient of the shearing angle

(select values between 1.0 and 1.2)

V_n =storey shear at n-th storey

Awn = cross sectional area of the wall at n-th stroey

G = Modulus of rigidity.

Using the common unit of $\frac{hn^2}{12EK_o}$ and G = E/2.3, the nominal value of δsn can be expressed as

$$\delta_{sn} = \triangle s_n.\ \frac{27.6\ K_o}{hn}$$

$$\text{where}\quad \triangle s_n = \frac{X.\ Vn}{Awn}.$$

5.10.3 4. *Foundation rotation* :—If the foundation rotates through and angle θ the wall as a whole rotates through the same angle. The inter-floor deflection of the n-th store is given by

$$\delta_{RN} = \theta \; h_n.$$

Using the common unit, as before, the nominal value of S_{RN} may be expressed as

$$\delta_{Rn} = \frac{12EK_o \theta}{h_n}$$

5.10.4. PROCEDURE FOR ANALYSIS

The procedure for analysis without considering the boundary effects of the beams, connecting the frames and shear walls will be described here. This corresponds to the simplified idealised structure of Khan's method. Muto has also given the method for considering the boundary effects but it will not be discussed here.

Step 1 : The D-values of colums are calculated as indicated in para .2. The D-vaules of all the columns in a storey are summed up.

Step 2 : The distribution of storey shears between the frames and shear walls is assumed by judgement.

Step 3 : The D-values of shear walls (lumped together) are calculated as indicated in para .3. Generally it may be adequate if the bending deflections only are considered.

Step 4 : The storey shears are distributed to the columns and shear walls in proportion to their D-values at each storey.

Step 5 : The distribution of storey shears obtained in step 4 is compared to assumed distribution. If the two are close engough, there is no need to do further cycles. Otherwise steps 3 and 4 are repeated till convergence is reached. If the D-values of columns are worked out from deflection calculations, these are also revised after each cycle.

Step 6 : The storey shears in the shear wall group is distributed to the different walls in proportion to their individual D-values, and the individual walls are then designed.

Step 7 : The storey shears in the column group is distributed to the various columns in proportion to their D-values.

Step 8 : The points of inflextion in the columns are obtained from Figs 6.13 to 6.15. y_o is the height of the standard point of inflexion, y_1 is a correction factor due to the difference between stiffnesses of the upper beam and lower beam. y_2 and y_3 are correction factors due to the difference in the heights of the storeys. The position of the point of inflexion is obtained by

$$y = y_o + y_1 + y_2 + y_3$$

Step 9 : The column moments are obtained from the column shear and the point of inflexion.

Step 10 : The beam moments are obtained from the column moments by considering the equilibrium of the joints. At internal joints of the frame, the value of bending moment in each beam is assumed to be proportional to its stiffness.

5.10.5. APPLICABILITY OF MUTO'S METHOD

Muto's method can be conveniently adopted for the analysis of interacting frames and shear walls or for simple framed structures. In the latter case only steps 1,7,8,9 and 10 mentioned earlier are required. The use of the formulae for calculating D-values of columns is generally good enough for practical use. However, the error increases when the relative stiffness of the beams is small in comparision with the columns, i.e. when k—is less than 0.5. If k—is less than 0.2, use of these formulae may not be accurate enough for reliable use. However, the results obtained will be better approximation than that by other approximate methods like cantilever method, portal method etc.

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2. Norris, C,, Honsen R J, Holley, M. J. Biggs, J.M. Nauryet S., and Minami, J. K. Structural Design for Dynamic Loads. McGraw-Hill Book Co., New York, 1959.

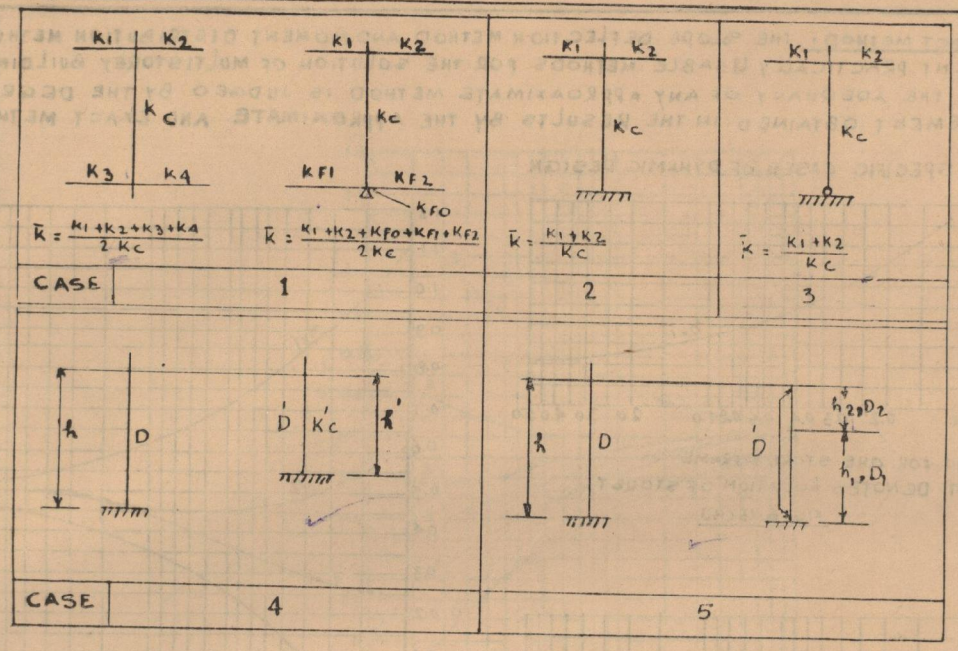


FIG. 6.10

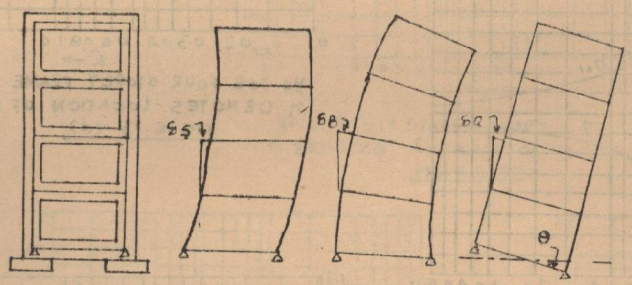
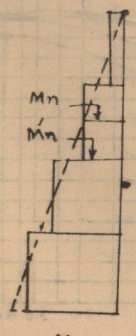


FIG. 6.11

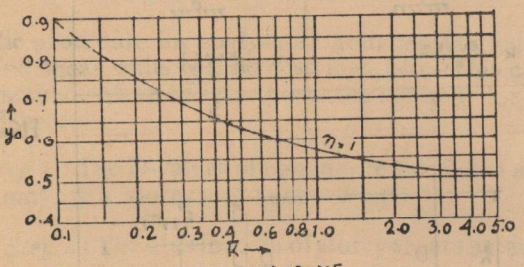


0	1	2	3	4	5	6	7	8
								δBn
n	Vn+1 hn	M'n	2 Mn	Kw	2Mn/Kw	4 ΔB	3/5 LW	(6)x(7)
4	A_1	A_2 (=A_1)	A_3 (=A_2)		A_4	A_5 ($B_4+B_3+A_4$)		
3	B_1	B_2 (A_2+B_1)	B_3 (A_2+B_2)		B_4	B_5 ($C_4+C_3+B_4$)		
2	C_1	C_2 (B_2+C_1)	C_3 (B_2+C_2)		C_4	C_5 ($D_4+D_3+C_4$)		
1	D_1	D_2 (C_2+D_1)	D_3 (C_2+D_2)		D_4	D_5 (=D_4)		

FIG. 6.12

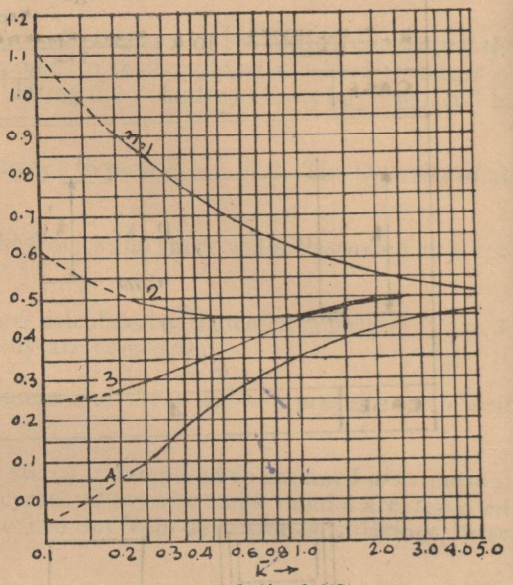
d. EXACT METHOD. THE SLOPE DEFLECTION METHOD AND MOMENT DISTRIBUTION METHOD REPRESENT PRACTICALLY USABLE METHODS FOR THE SOLUTION OF MULTISTOREY BUILDING FRAMES. THE ADEQUACY OF ANY APPROAXIMATE METHOD IS JUDGED BY THE DEGREE OF AGREEMENT OBTAINED IN THE RESULTS BY THE APPROXIMATE AND EXACT METHODS.

SPECIFIC CASES OF DYNAMIC DESIGN



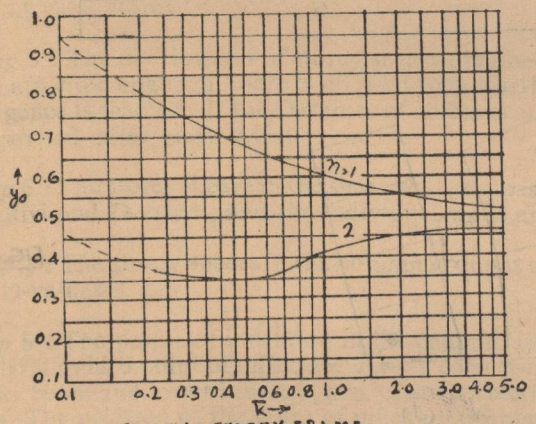
yo FOR ONE STOREY FRAME
n DENOTES LOCATION OF STOREY.

FIG. 6.13 (a)



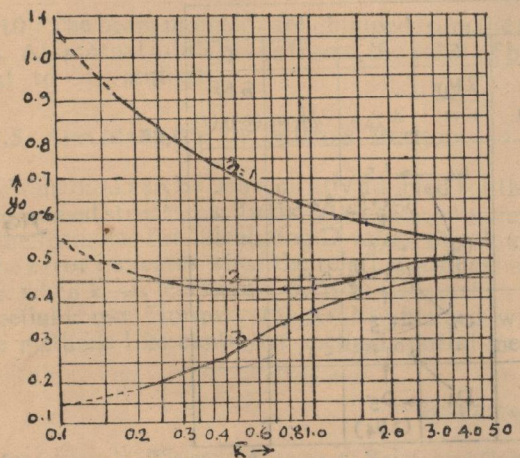
yo FOR FOUR STOREY FRAME
n DENOTES LOCATION OF STOREY

FIG. 6.13 (d)



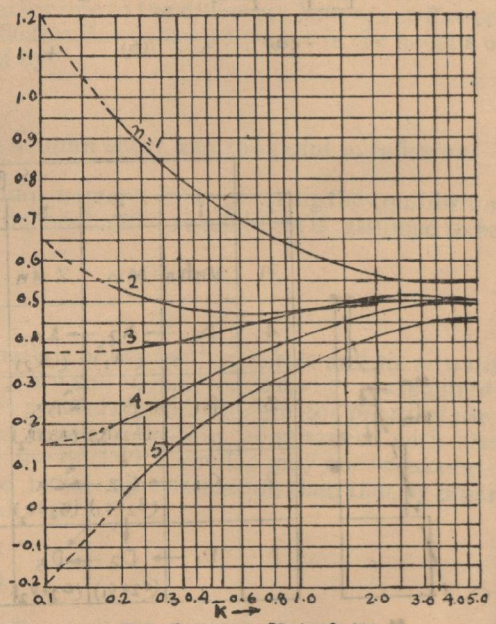
yo FOR TWO STOREY FRAME
n DENOTES LOCATION OF STOREY.

FIG. 6.13 (b)



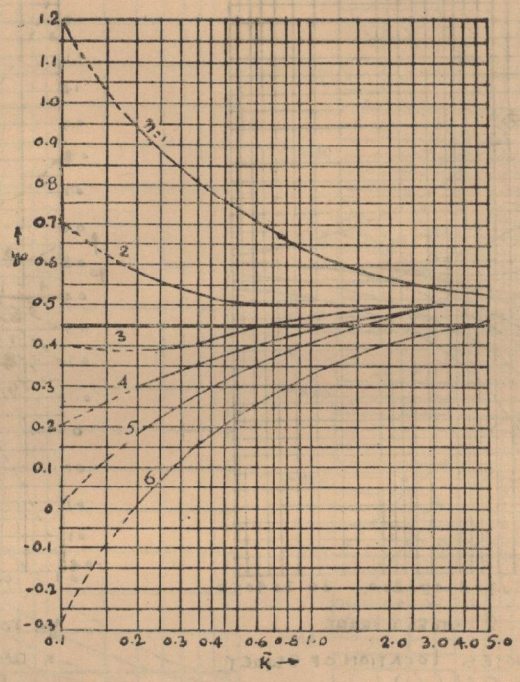
yo FOR THREE STOREY FRAME
n DENOTES LOCATION OF STOREY

FIG. 6.13 (c)

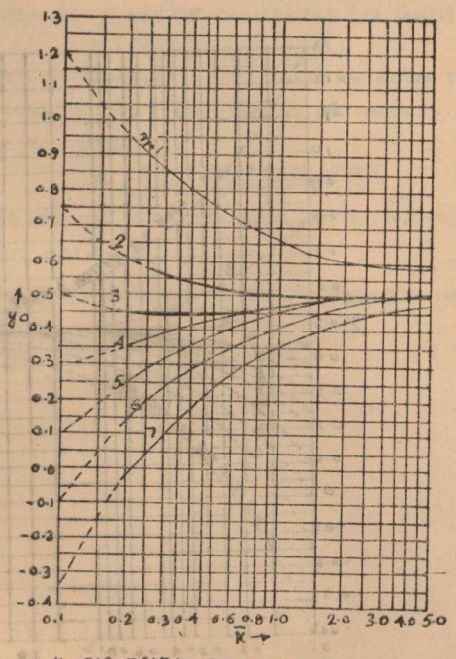


yo FOR FIVE STOREY FRAME
n DENOTES LOCATION OF STOREY

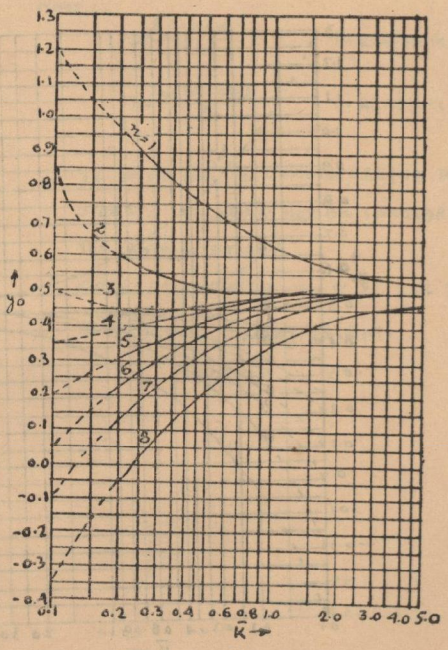
FIG. 6.13 (e)



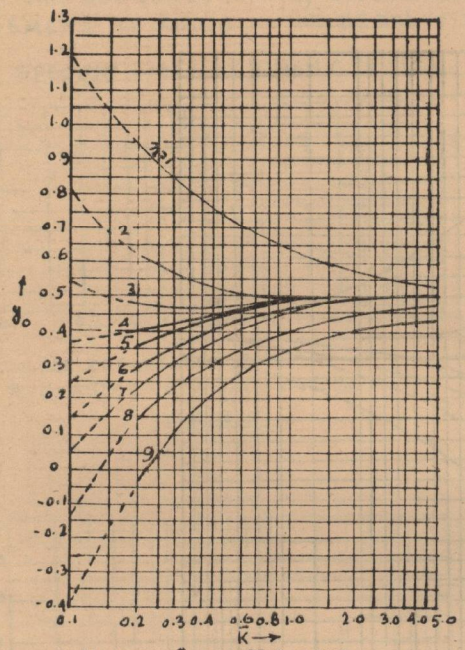
y_o FOR SIX STOREY FRAME
n DENOTES LOCATION OF STOREY
FIG. 6.13 (f)



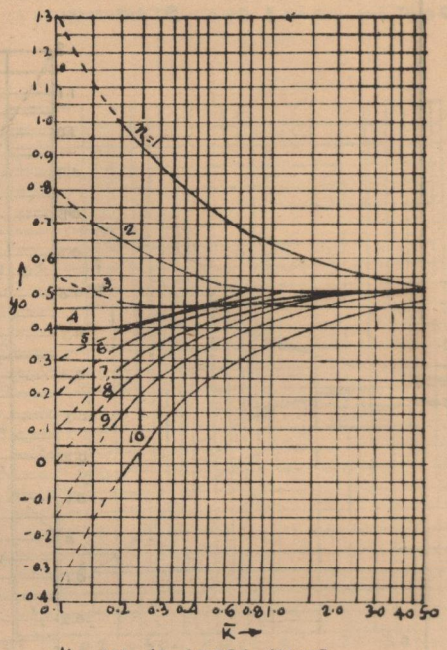
y_o FOR SEVEN STOREY FRAME.
n DENOTES LOCATION OF STOREY
FIG. 6.13 (g)



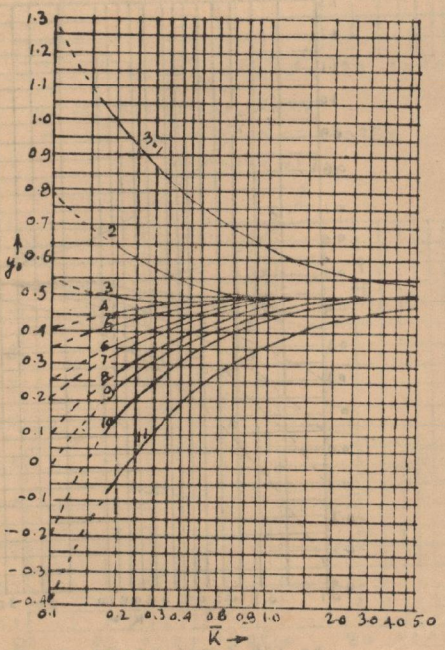
y_o FOR EIGHT STOREY FRAME
n DENOTES LOCATION OF STOREY
FIG. 6.13 (h)



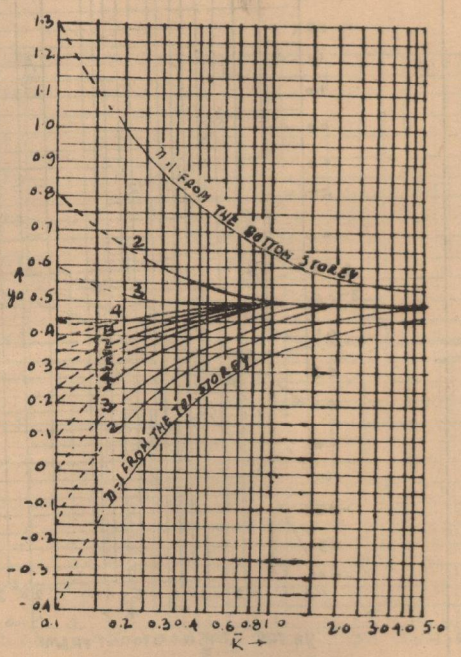
y_o FOR 9 STOREY FRAME
n DENOTES LOCATION OF STOREY
FIG. 6.13 (i)



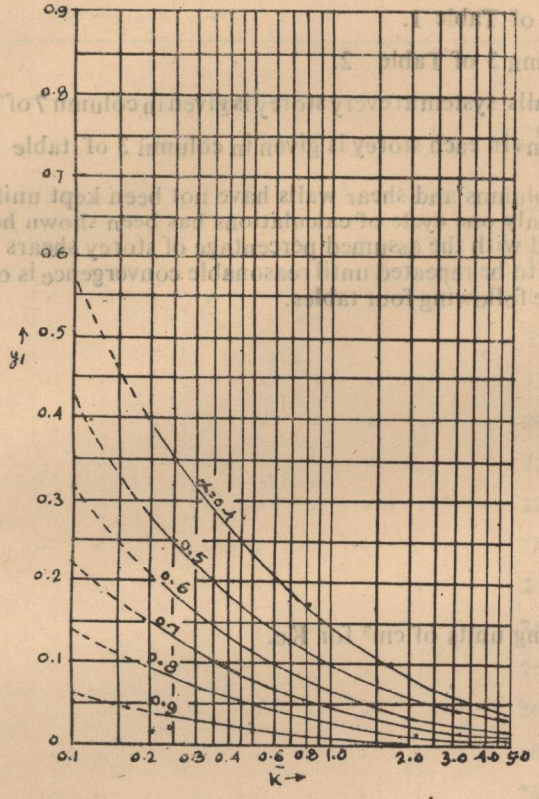
y_o FOR 10 STOREY FRAME
n DENOTES LOCATION OF STOREY
FIG. 6.13 (j)



y_o FOR 11 STOREY FRAME
n DENOTES LOCATION OF STOREY
FIG. 6.13 (K)



y_o FOR 12 STOREY OR MORE FRAME
n DENOTES LOCATION OF STOREY
FIG. 6.13 (l)



NOTE:- NEED NOT BE CONSIDERED FOR LOWEST STOREY.

$$\alpha = \frac{\Sigma K \text{ OF GIRDERS ABOVE}}{\Sigma K \text{ OF GIRDERS BELOW}}$$

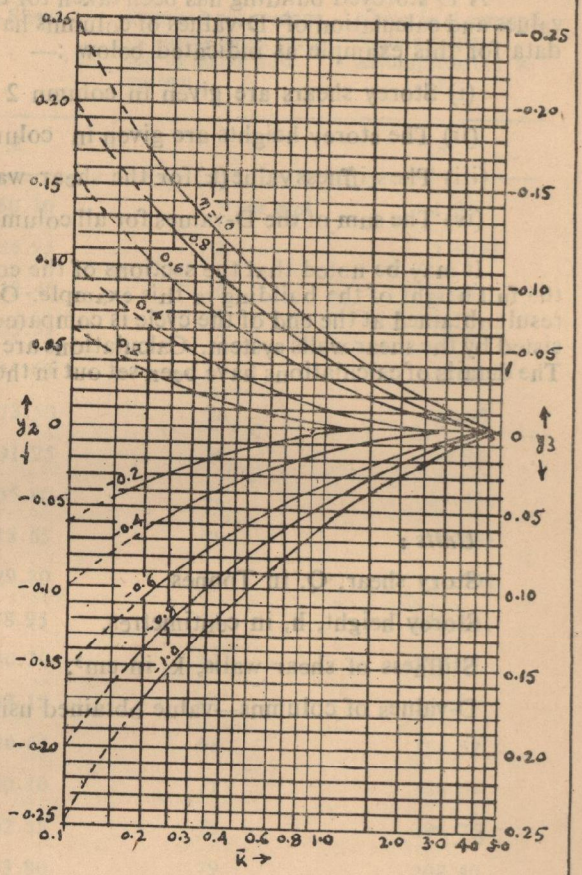
WHEN THE STIFFNESS OF THE GIRDERS BELOW THE COLUMN IS LESS THAN THAT OF THE GIRDERS AT THE TOP TAKE

$$\alpha = \frac{\Sigma K \text{ OF GIRDERS AT BOTTOM}}{\Sigma K \text{ OF GIRDERS AT TOP}}$$

AND USE THE NEGATIVE SIGN OF THE VALUE THUS OBTAINED

CORRECTION VALUE y_1 DUE TO VARIATION IN GIRDER STIFFNESS.

FIG. 6.14



NOTE:-

$$\eta = \frac{h_{(ABOVE)}}{h} - 1 \quad \text{IN READING } y_2$$

$$\eta = \frac{h_{(BELOW)}}{h} - 1 \quad \text{IN READING } y_3$$

y_2 NEED NOT BE CONSIDERED FOR THE TOP STOREY. y_3 NEED NOT BE CONSIDERED FOR THE BOTTOM STOREY.

CORRECTION VALUE y_2 & y_3 DUE TO VARIATION IN STOREY HEIGHT.

FIG. 6.15

Example of analysis of interacting frames and shear walls by Muto's method.

A 19 storeyed building has been taken for this example. The calculations of laterial forces, stiffness values and calculation of D-values of columns have not been shown here. These have been taken as given data for this example as indicated below :—

(i) Storey shears are given in column 2 of Table 1.

(ii) The storey heights are given in column 3 of Table 2.

(iii) The stiffness value (k) for the shear walls system at every storey is given in column 7 of Table 2.

(iv) The sum of the D-values for all columns in each storey is given in column 3 of table 4.

It may be noted that the sections of the columns and shear walls have not been kept uniform for the full height of the building in this example. Only one cycle of calculations has been shown here. The result obtained at the end of the cycle is compared with the assumed percentage of storey shears to be resisted by the shear walls system. Calculations are to be repeated until reasonable convergence is obtained. The details of calculations have been set out in the following four tables.

Units :

Story shear, Q, in Tonnes.

Storey height, h, in centimetres.

Stiffness of shear walls, k, in cm^3.

D-values of columns—value obtained using units of cm^3 for Kc.

TABLE 1

Storey	Storey Shear	Assumed per centage of storey shear resisted by shear wall system.	Shear in shear wall system as per assumption (2) × (3)
1	2	3	4
19	60.30	3 per cent	1.81
18	86.95	3	2.6085
17	112.45	6	6.747
16	135.35	12	16.242
15	155.65	21	32.6865
14	173.50	28	48.58
13	191.25	36	68.85
12	205.80	42	86.436
11	218.65	48	104.952
10	229.80	53	121.794
9	238.95	57	136.201
8	246.30	62	152.71
7	252.15	64	161.38
6	256.65	68	174.52
5	260.10	71	184.67
4	262.35	75	196.76
3	263.80	79	208.40
2	264.65	84	222.31
1	265.00	97	257.50

TABLE 2

Calculations For $4\triangle B_n$

Floor	Vn	hn	Vn hn	M^1n	$2M_i$	K	$2M_{ik}$	4 Bn
1	2	3	4	5	6	7	8	9
19	1.81	2.9×10^2	5.244×10^3	5.244×10^3	5.244×10^3	771.80×10^4	$.067\times10^3$	1534.2781×10^3
18	2.608	2.9	7.56	12.804	18.048	—do—	.2338	1533.9773
17	6.747	2.9	19.57	32.374	45.178	—do—	.5853	1533.1582
16	16.242	2.9	47.10	79.474	111.848	—do—	1.4491	1531.1238
15	32.68	2.9	94.77	174.184	253.658	—do—	3.2865	1526.3882
14	48.58	2.9	140.88	315.064	489.248	—do—	6.3390	1516.7627
13	68.85	2.9	199.66	514.724	829.788	1149.60×10^4	7.218	1503.2057
12	86.44	2.9	250.67	765.394	1280.118	—do—	11.1353	1484.8524
11	104.95	2.9	304.35	1069.744	1835.138	—do—	15.9632	1457.7539
10	121.79	2.9	353.19	1422.934	2492.678	—do—	21.6830	1420.1077
9	136.20	2.9	394.98	1817.914	3240.848	—do—	28.191	1370.2484
8	152.71	2.9	442.86	2260.774	4078.688	—do—	35.479	1306.5784
7	161.38	2.9	468.00	2728.774	4989.548	—do—	43.4024	1227.6970
6	174.52	2.9	506.11	3234.884	5963.658	—do—	51.8759	1132.4187
5	184.67	2.9	535.54	3770.424	7005.308	1182.20×10^4	59.2565	1021.2863
4	196.76	2.9	570.60	4341.024	8111.448	—do—	68.6131	893.4167
3	218.40	2.9	604.36	4945.384	9286.408	—do—	78.5 19	746.2517
2	222.31	2.9	644.70	5590.084	10535.468	—do—	89.1174	578.5834
1	257.05	5.5	1413.77	7003.854	12593.938	514.60×10^4	244.7325	244.7325

TABLE 3

Calculations for δ_{Bn} & D_w

Storey	Vn	hn	$\frac{3}{hn}$	$SBn=4\Delta Bn \times \frac{3}{hn}$	$Dw\frac{Vn}{\delta Bn}$
1	2	3	4	5	6
19	1.81	2.9×10^{3}	1.0344×10^{3}	1587.057×10^{5}	114
18	2.608	—do—	—do—	1586.740	164
17	6.75	—do—	—do—	1585.90	425
16	16.24	—do—	—do—	1583.79	1025
15	32.68	—do—	—do—	1578.90	2069
14	48.58	—do—	—do—	1568.94	3100
13	68.85	—do—	—do—	1554.92	4427
12	86.44	—do—	—do—	1535.93	5627
11	104.95	—do—	—do—	1507.90	6960
10	121.79	—do—	—do—	1468.96	8290
9	136.20	—do—	—do—	1417.38	9609
8	152.71	—do—	—do—	1351.52	11300
7	161.38	—do—	—do—	1269.93	12710
6	174.52	—do—	—do—	1171.37	14300
5	184.67	—do—	—do—	1056.42	17480
4	196.76	—do—	—do—	924.15	21290
3	208.40	—do—	—do—	771.92	27000
2	222.51	—do—	—do—	598.49	37180
1	257.05	5.5×10^{3}	$.5454\times10^{-2}$	133.48	1,92,600

TABLE 4

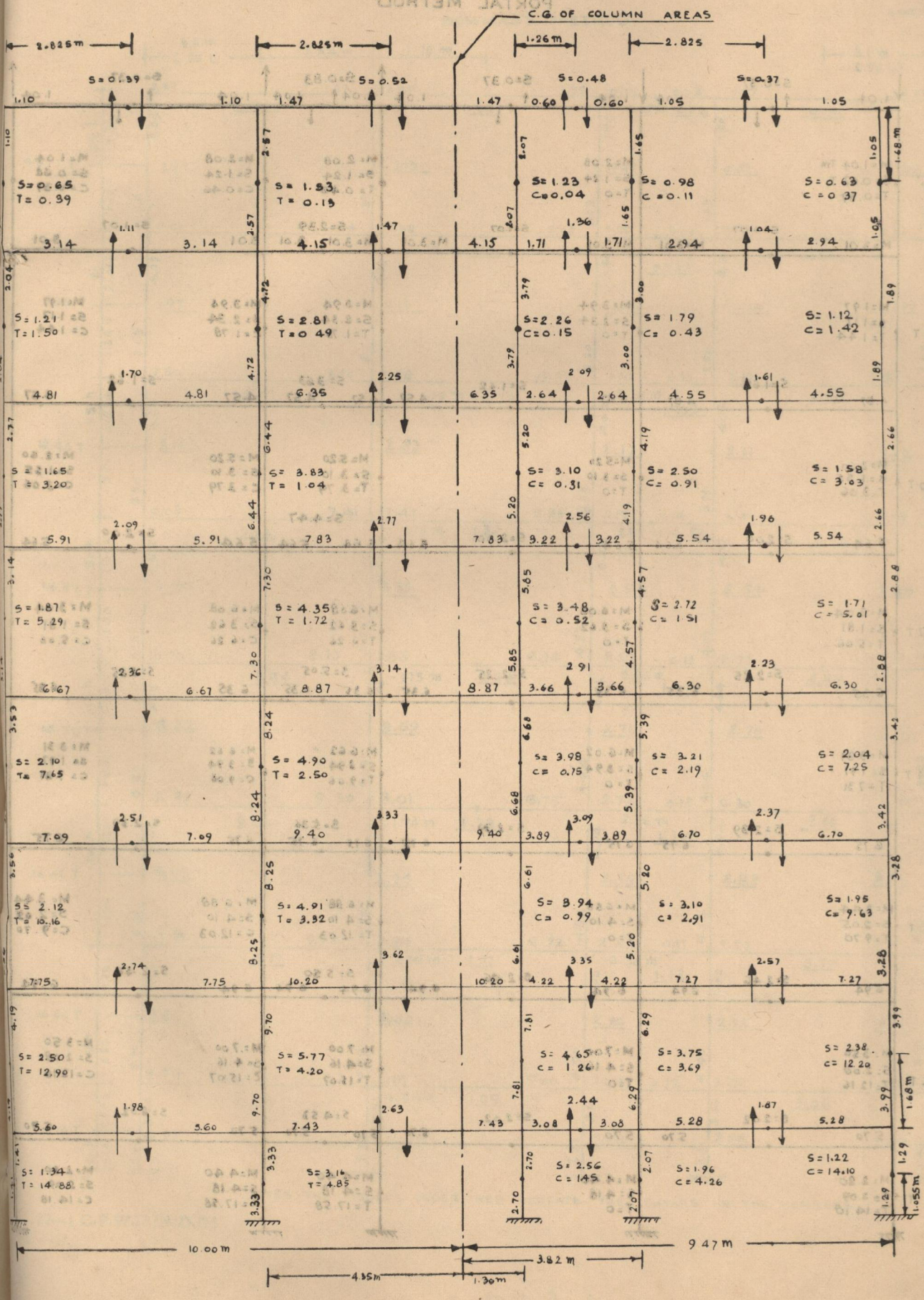
Calculations for percentage shear shared by Shear wall system

Storey	'D'value of shear wall system(Dw)	'D'value of frame ΣDc	Total 'D'value ΣD	Percentage shear shared by shear wall system	Assumed percentage of storeyshear resisted by shear wall system*
1	2	3	4	5	6
19	114	5783.8	5897.8	1.9	3
18	164	5783.8	5947.8	2.7	3
17	425	5783.8	6208.8	6.8	6
16	1025	5783.8	6808.8	15	12
15	2069	5783.8	7852.8	26.3	21
14	3100	5783.8	8883.8	34.8	28
13	4427	5783.8	10210.8	43.3	36
12	5627	5783.8	11410.8	49.3	42
11	6960	5783.8	12743.8	54.6	48
10	8290	5783.8	14073.8	58.9	53
9	9609	5783.8	15392.8	62.4	57
8	11300	5783.8	17083.8	66.1	62
7	12710	5783.8	18493.8	68.7	64
6	14900	5818.0	20718.0	72.1	68
5	17480	5917.2	23397.2	74.7	71
4	21290	5917.2	27207.2	78.2	75
3	27000	5917.2	32917.2	82	79
2	37180	5917.2	43097.2	86.2	84
1	1,92,600	55081	247681	77.8	97

*Column 3 of Table .1 repeated here for comparison

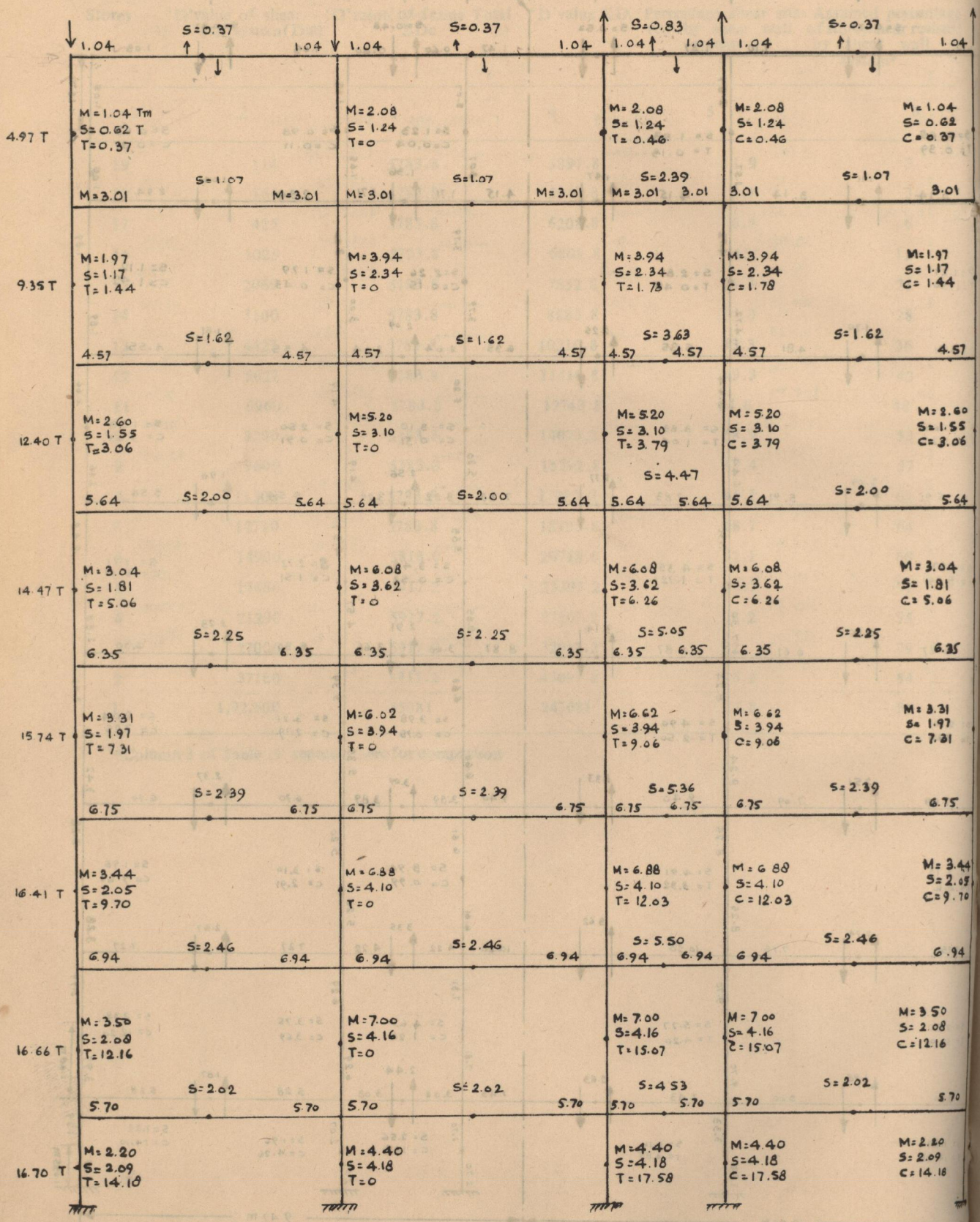
(ANNEXURE 5.1)

CANTILEVER METHOD



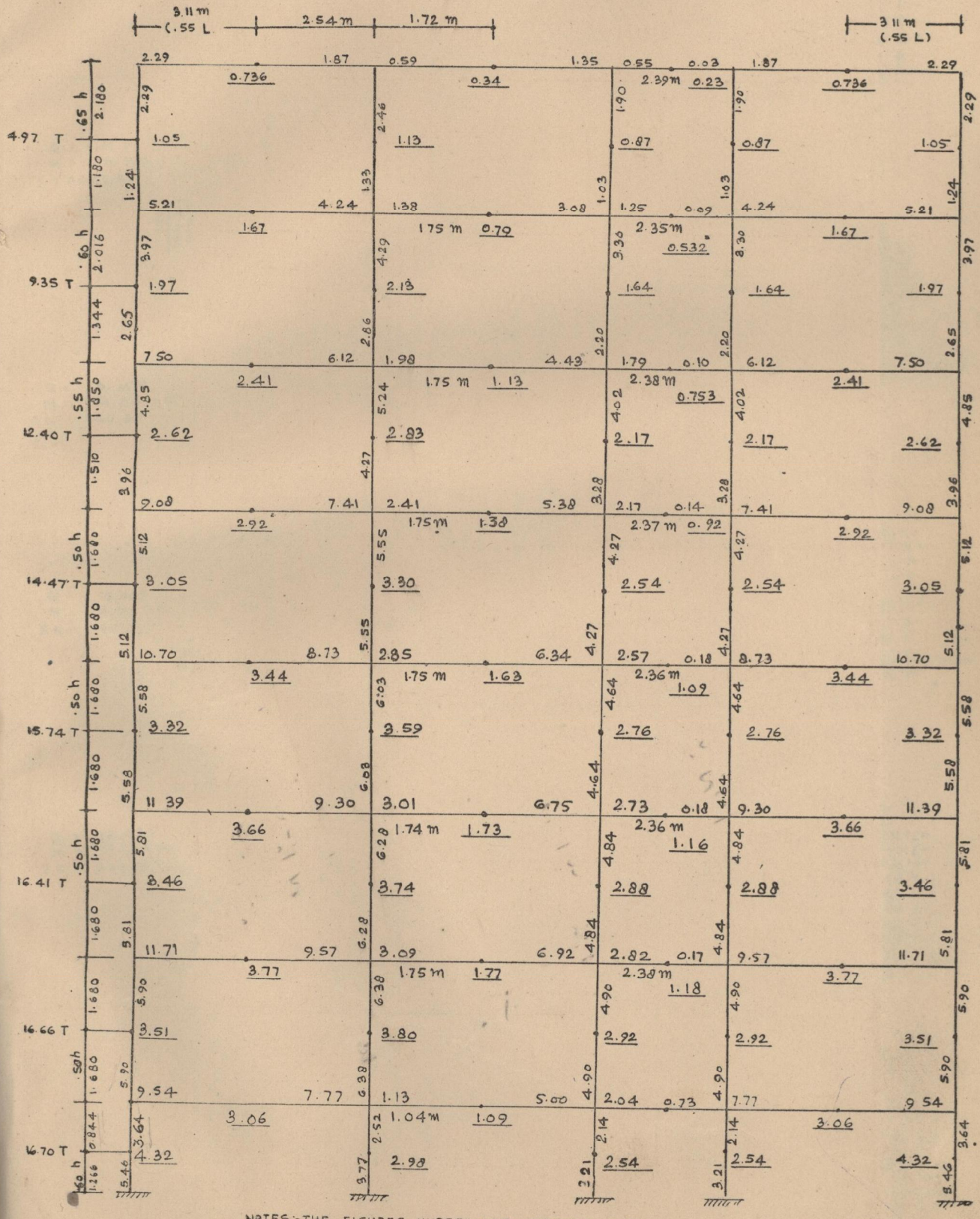
ANNEXURE 5.2

PORTAL METHOD



ANNEXURE 5.3

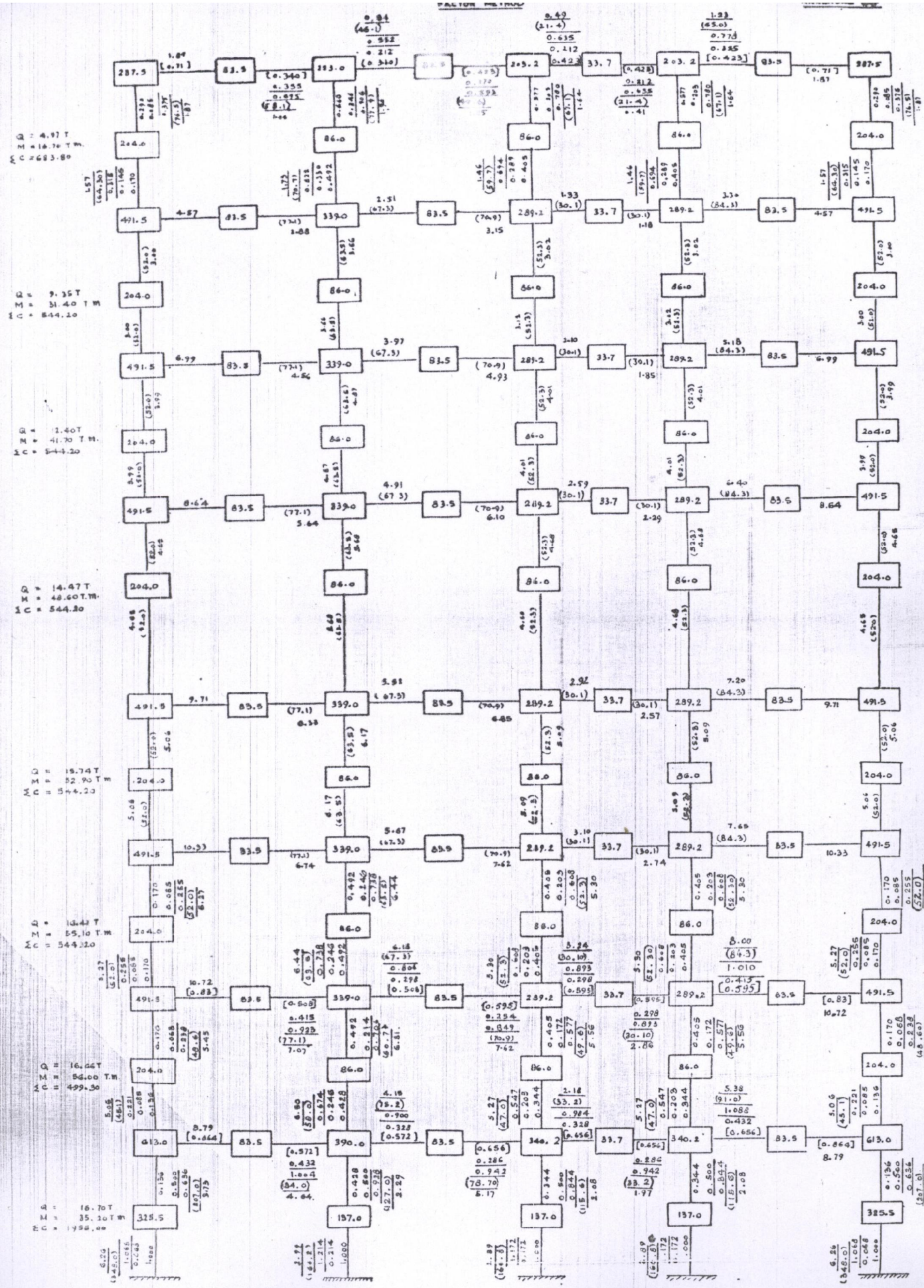
BOWMAN'S METHOD



NOTES:- THE FIGURES UNDERLINED INDICATE THE SHEARS IN THE MEMBERS

5.65 : 2.52

8.17



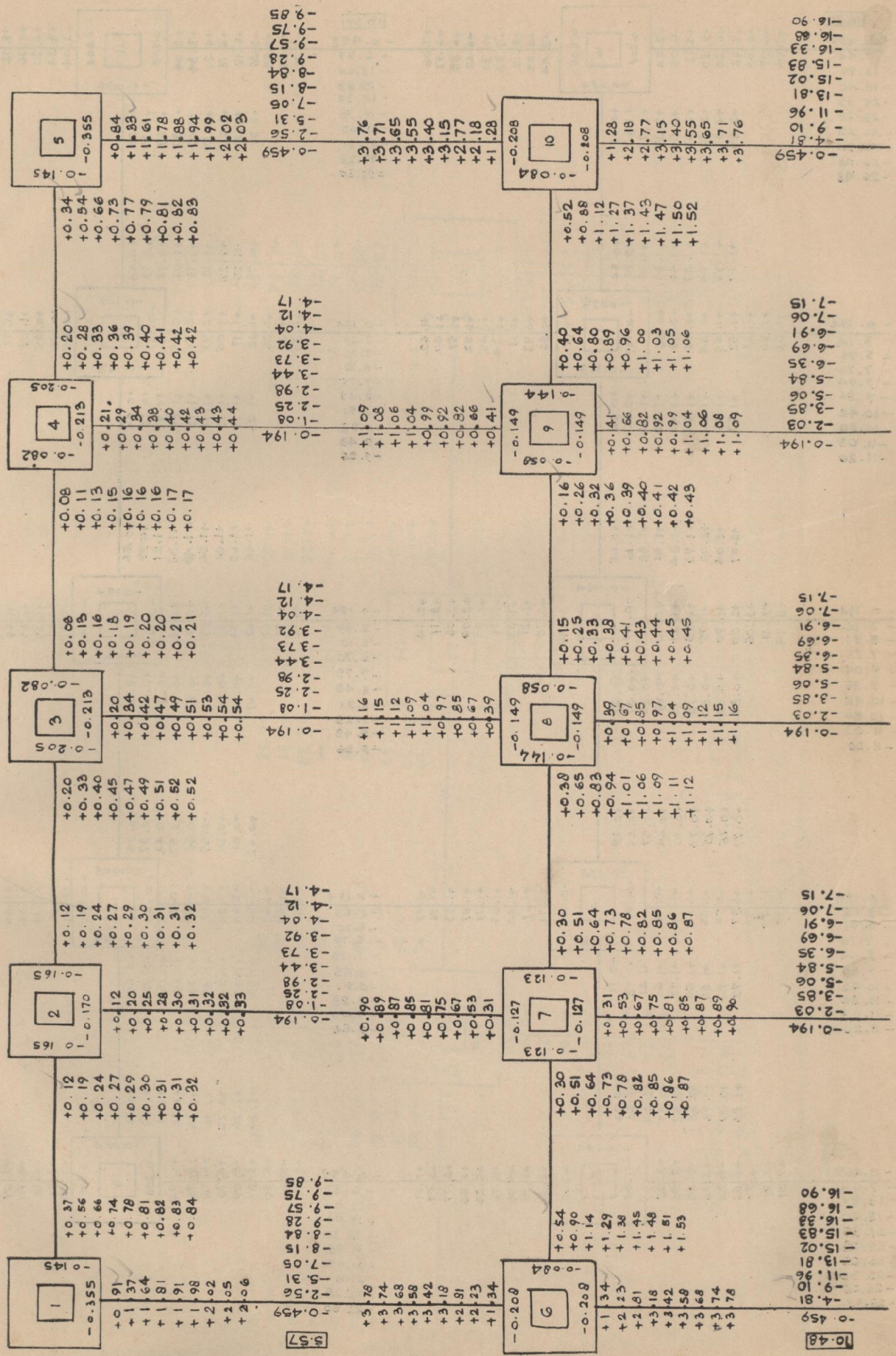
NOTE :- 1) FIGURES SHOWN INSIDE SQUARE BRACKETS INDICATE THE GIRDER FACTORS.

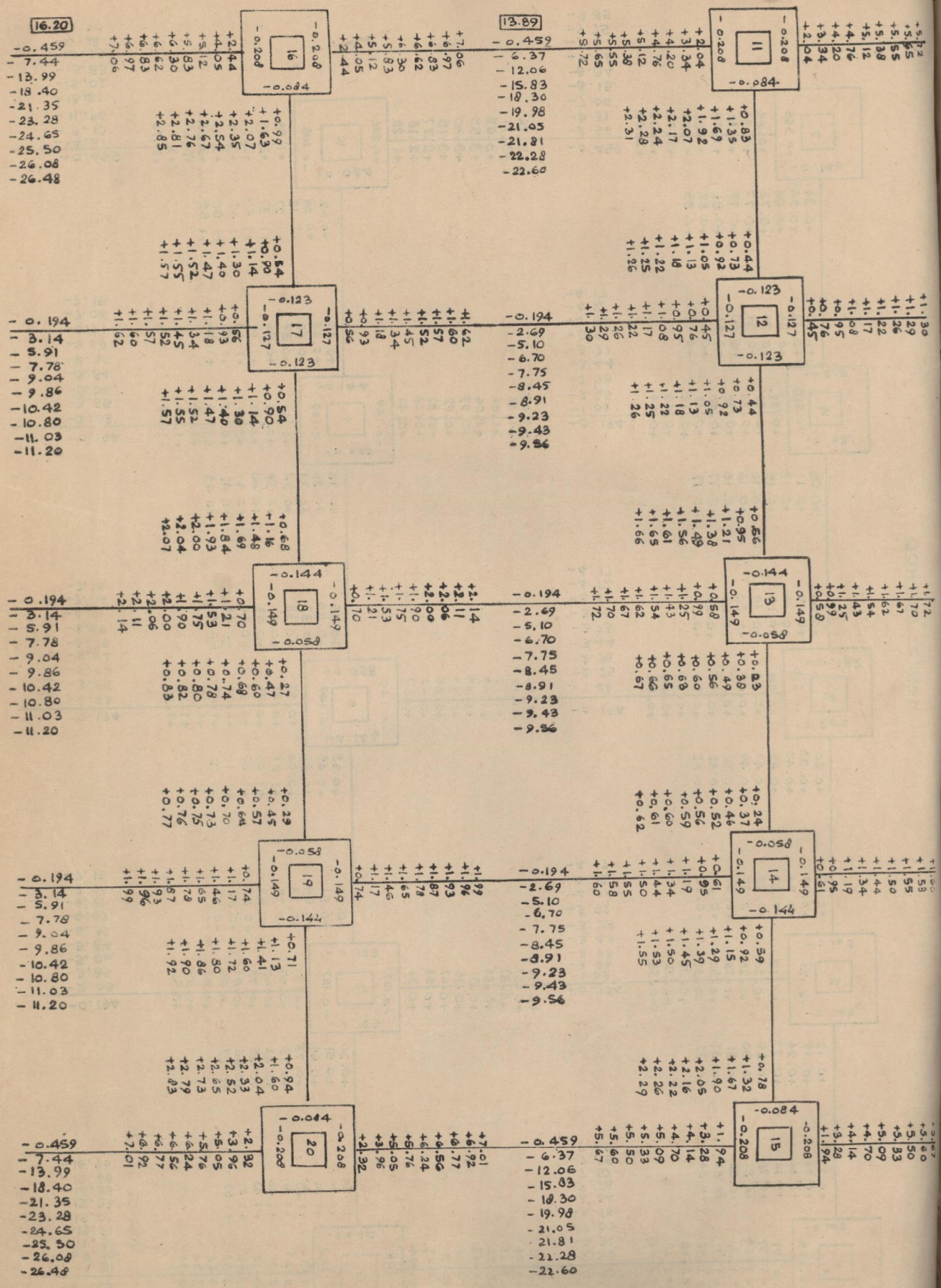
2) FIGURES SHOWN INSIDE SMALL BRACKETS INDICATE THE COLUMN MOMENT FACTORS AND GIRDER MOMENT FACTORS.

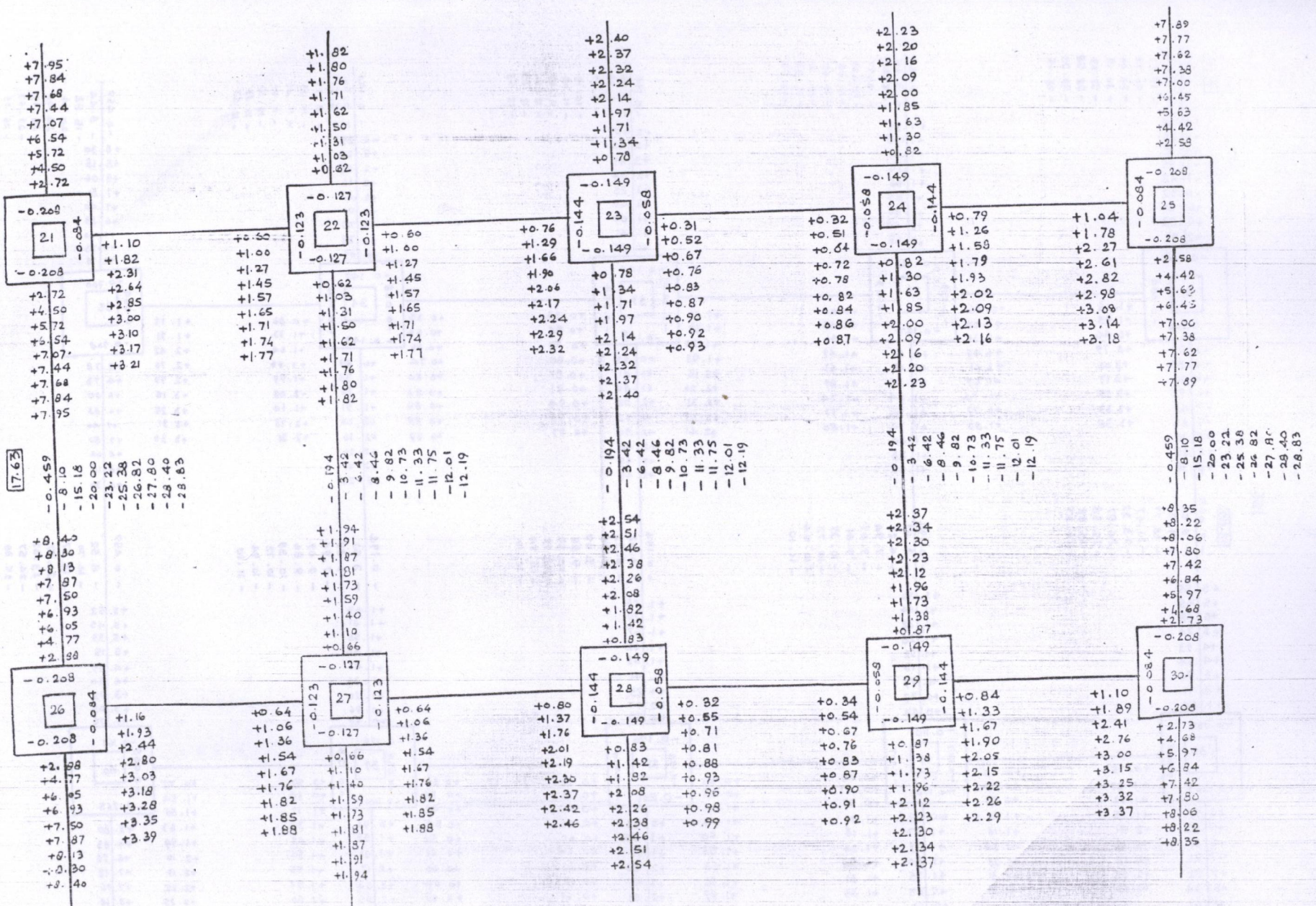
3) Q, M AND Σ C DESIGNATE TOTAL SHEAR, STOREY MOMENT AND SUM OF COLUMN MOMENT FACTORS RESPECTIVELY

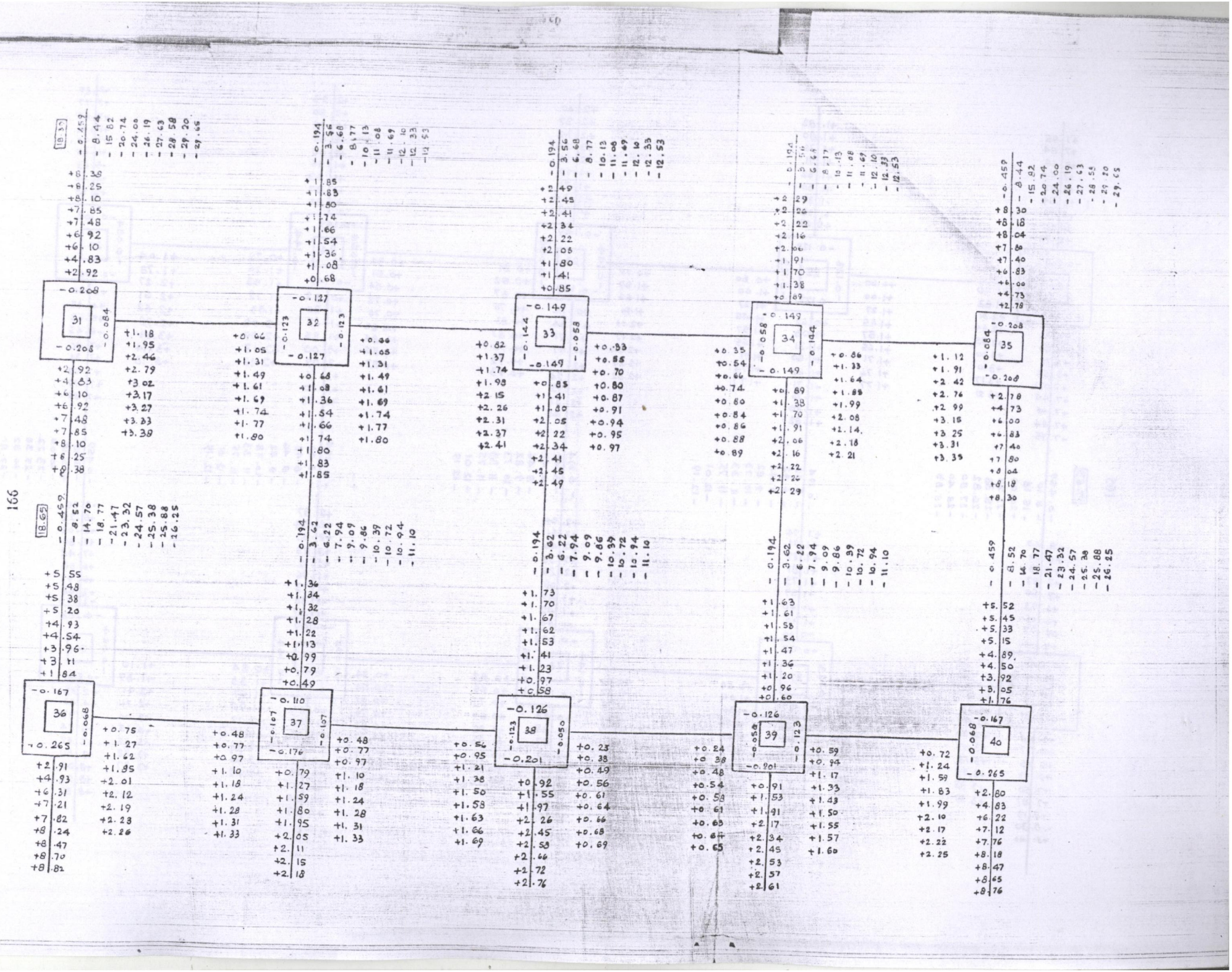
ANNEXURE 5.5(A)

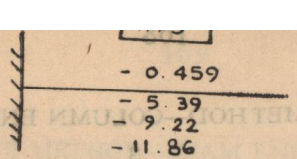
KANI'S METHOD



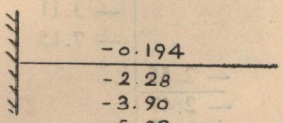

16.20
13.89
11
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17.63

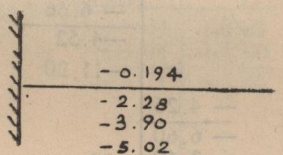

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40



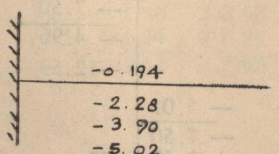
- 0.459
- 5.39
- 9.22
- 11.86
- 13.63
- 14.82
- 15.63
- 16.17
- 16.48
- 16.73



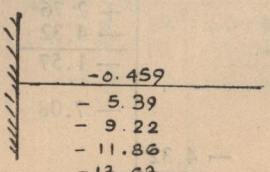
-0.194
-2.28
-3.90
-5.02
-5.76
-6.26
-6.61
-6.84
-6.97
-7.08



-0.194
-2.28
-3.90
-5.02
-5.76
-6.26
-6.61
-6.84
-6.97
-7.08



-0.194
-2.28
-3.90
-5.02
-5.76
-6.26
-6.61
-6.84
-6.97
-7.08



-0.459
-5.39
-9.22
-11.86
-13.63
-14.82
-15.63
-16.17
-16.48
-16.73

Annexure 5.5 (B)

KANI'S METHOD—COLUMN END MOMENTS

1		2		3		4		5
	+ 2.06 — 4.01 — 1.95 — 9.85		+ 0.33 — 2.94 — 2.61 — 4.17		+ 0.54 — 2.47 — 1.93 — 4.17		+ 0.44 — 2.64 — 2.20 — 4.17		+2.03 —4.06 —2.03
— 0.23 — 4.01 + 3.78	6	—2.04 —2.94 +0.90	— 1.31 — 2.47 + 1.16	— 1.55 — 2.64 + 1.09	9	— 98.5 — 0.30 — 4.06 + 3.76	10		
	+ 3.78 — 7.40 — 3.62 —16.90		+ 0.90 — 4.95 — 4.05 — 7.15		+ 1.16 — 4.27 — 3.11 — 7.15		+ 0.10 — 4.46 — 3.37 — 7.15		+3.76 —7.47 —3.71
— 1.68 — 7.40 + 5.72	11	— 3.65 — 4.95 + 1.30	— 2.55 — 2.47 + 1·72	— 2.86 — 4.46 + 1.60	13	—16.90 — 1.80 — 7.47 + 5.67	15		
	+ 5.72 — 9.82 — 4.10 —22.60		+ 1.80 — 6.64 — 5.34 — 9.56		+ 1.72 — 5.70 — 3.98 — 9.56		+ 1.60 — 5.97 — 4.37 — 9.56		+5.67 —9.92 —4.25
— 2.76 — 9.86 + 7.66	16	— 5.02 — 6.64 + 1.62	— 3.56 — 5.70 + 2.14	— 3.98 — 5.97 + 1.99	18	—22.60 — 2.91 — 9.92 +7.01	20		
	+ 7.06 —11.47 — 4.41 —26.48		+ 1.62 — 7.76 — 6.14 —11.20		+ 2.14 — 6.66 —4.52 —11.20		+ 1.99 — 6.98 — 4.99 — 11.20		+7.01 —11.58 —4.57
— 3.52 —11.47 — 7.95	21	— 5.94 — 7.76 + 1.82	— 4.26 — 6.60 + 2.40	— 4.75 — 6.98 + 22.3	23	—26.48 — 2.91 — 99.2 + 7.01	25		
	+ 6.95 —12.48 — 4.53 —22.83		+ 1.82 — 8.43 — 6.61 —12.19		+ 2.40 — 7.25 — 4.85 —12.19		+ 2.23 — 7.59 — 5.36 —12.19		+7.89 —12.59 —4.70
— 4.08 —12.44 + 8.40	26	— 6.49 — 8.43 + 1.94	— 4.71 — 7.25 + 2.54	— 5.22 — 7.59 + 2.37	28	—28.83 — 4.24 —12.59 + 8.35	30		
	+ 8.40 —12·87 — 4.47 —29.65		+ 1.94 — 8.74 — 6.80 —12.53		+ 2.54 — 7.50 — 4.96 —12.53		+ 2.37 — 2.87 — 5.50 —12.53		+8.35 —13.00 —4.65
— 4.49 —12.87 + 8.38	31	— 6.89 — 8.74 + 1.85	— 5.01 — 7.50 + 2.49	— 5.58 — 7.87 + 2.2[illegible]	33	— 4.70 —13.00 + 8.30	35		
	+ 8.38 —12.32 — 3.94 —26.25		+ 1.85 — 7.89 — 6.04 —11.10		+ 2.49 — 6.88 — 4.39 —11.10		+ 2.29 — 7.18 — 4.80 —11.10		+8.3[illegible] —12.4[illegible] —4.1[illegible]
— 6.77 —12.32 : 5.55	36	— 6.53 — 7.89 +.1.36	— 5.15 — 6.88 + 1.73	— 5.55 — 7.18 + 1.63	38	— 6.91 —12.43 + 5.52	45		
	+ 8.82 — 7.91 + 0.91 —16.73		+ 2.18 — 4.90 + 2.72 — 7.08		+ 2.76 — 4.32 — 1.57 — 7.08		+ 2.61 — 4.47 — 1.68 — 7.08		+8.7[illegible] —7.9[illegible] +0.7[illegible]
— 7.91		— 4.90	— 4.32	— 4.47		—16.73 — 7.97			

KANI'S METHOD—BEAM END MOMENTS

1		2		3		4		5	
	+0.84 +1.16 +2·00	+0·32 +1.16 +1·48	+0.32 +0.84 +1.16	+0.52 +0.84 +1.36	+0.21 +0.38 +0.59	+0.17 +0.38 +0.55	+0.62 +1.25 +1.67	+0.8 +1.25 +2.08	
6		7		8		9		10	
	+1·53 +2.40 +3.93	+0.87 +2.40 +3·27	+0.87 +1.99 +2.86	+1.12 +1.99 +3.11	+0.43 +0.88 +1.31	+0.43 +0.88 +1·31	+9.06 +2.58 +3.64	+1·52 +2.58 +4·10	
11		12		13		14		15	
	+2·31 +3·27 +5·88	+1.26 +3.57 +4.83	+1.26 +2.92 +4.18	+1·66 +2·92 +4·58	+0.67 +1.29 +1.96	+0·62 +1.29 +1.91	+1.55 +3.84 +5.39	+2·29 +3·84 +6·13	
16		17		18		19		20	
	+2.85 +4.42 +7.27	+1.57 +4.42 +5.89	+1.57 +3·64 +5·71	+2·07 +3.64 +5·71	+0·83 +1·60 +2·43	+0·77 +1·60 +2.37	+1·92 +4·75 +6·67	+2·29 +3·84 +6·13	
21		22		23		24		25	
	+3·21 +4·98 +8.19	+1.77 +4·98 +6·75	+1.77 +4·09 +5·86	+2·32 +4·09 +6·41	+0.93 +0.80 +0.73	+0.87 +1.80 +2.67	+2·16 +5·34 +7.50	+3·18 +5.34 +8.52	
26		27		28		29		30	
	+3.39 +5.27 +8·66	+1.88 +5·27 +7·15	+1.88 +4.34 +6·22	+2.46 +4.34 +6·80	+0.99 +1.91 +2.90	+0·92 +1.91 +2.83	+2·29 +5·66 +7·95	+3·37 +5·66 +9·03	
31		32		33		34		35	
	+3.38 +5·18 +8·56	+1.80 +5·18 +6·98	+1.80 +4.21 +6.01	+2·41 +4.21 +6.21	+0.97 +1.86 +2.83	+0.89 +1·86 +2·75	+2·21 +5.66 +6·77	+3. 5 +5.56 +8·91	
36		37		38		39		40	
	+2.26 +3·59 +5.85	+1.33 +3.59 +4·92	+1·33 +3.92 +4.92	+1.69 +3·02 +4·71	+0·69 +1·34 +2·03	+0·65 +1.34 +1.99	+1.60 +3.85 +5.45	+2.25 +3.85 +6.10	

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